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STUDIES ON SEISMIC ISOLATION FOR HOUSING IN DEVELOPING REGIONS

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ABSTRACT

This paper summarizes a time-history analysis of a base-isolated demonstration building that was built in 1994 at Pasir Badak, Indonesia. The building is a four-story reinforced concrete frame with masonry infill walls and is supported on high-damping natural rubber (HDNR) bearings. The modeling of the isolation bearings and the superstructure were based on the experimental studies carried out at the Earthquake Engineering Research Center (EERC), University of California at Berkeley, Berkeley, California.

The superstructure of the demonstration building was modeled as a three-dimensional frame with linear elastic behavior. Beam-column elements were used to model the structural members. The stiffness of the structure was obtained using cracked section properties of the beam and full cross-section of the columns. The stiffness contribution of the masonry panels was not taken into account because the panels were separated from the structural frame by a 25 mm-thick seismic gap that was filled with soft mortar to accommodate 1.5% interstory drift. The bearings were modeled as a combination of a linear spring element and a viscous damping element. The dynamic properties of the bearing elements were selected to match the force-displacement relationship from the full-scale bearing tests. A series of ground motions with two orthogonal components from past earthquakes were selected based on site-specific design spectra to represent the maximum design earthquake (MCE) and the maximum probable earthquake (MPE) seismic risk levels.

The results show that the HDNR bearings used in the demonstration building were very effective in isolating the superstructure from the ground motions. The floor maximum accelerations were smaller than the ground maximum acceleration. The maximum displacements at the isolation level were within the design values. The interstory drifts were reduced drastically when compared to the fixed-base model. The demonstration building in Indonesia is a case study in the process of implementing seismic isolation technology using HDNR bearings, and will serve as a lesson for the implementation of isolation in other developing countries as an alternative approach to protecting structures beyond life-safety criteria.

KEYWORDS

natural rubber bearing, base-isolation, seismic isolation, low-cost isolator, low-cost housing, earthquake protection, demonstration building.

INTRODUCTION

The construction of a four-story reinforced concrete building, supported on high-damping natural rubber (HDNR) isolators, in the south of Java, Indonesia was completed in 1994. The construction of this demonstration building was sponsored by the United Nations Industrial Development Organization (UNIDO) to introduce base-isolation technology to developing countries. For any innovative system to be widely adopted in developing nations, it must be cost-effective and technically efficient, thus, the design and construction of the superstructure of the isolated building should not deviate substantially from common practice and should use building codes for fixed-base buildings and vernacular building materials.

In base-isolated buildings, the earthquake forces transmitted to the superstructure or the floor accelerations are usually smaller than the peak ground acceleration (PGA) because the majority of the earthquake energy is not transmitted to the superstructure, but is transformed into large displacements at the seismic isolation level. Well-designed seismic isolation devices, such as HDNR bearings, should be able to accommodate very large horizontal displacements resulting from an earthquake without losing the ability to carry vertical and horizontal loads. Typically, two levels of seismic risk are defined for isolated structures. The Maximum Design Earthquake (MDE) represents the level of shaking expected during the lifetime of the building and it is used to design the isolator and the superstructure. The Maximum Credible Earthquake (MCE) is the largest input possible given the geological framework of the building site. The isolation system as a whole should be designed and tested to remain stable for the MCE input level.

Description of the Demonstration Building

The demonstration building in Indonesia is located in the southern part of West Java, about one kilometer southwest of Pelabuhan Ratu. It is a four-story moment-resisting reinforced concrete structure with masonry infill walls that are supported by sixteen high-damping natural rubber elastomeric bearings. The isolation bearings are located on the ground level and employ a recessed-endplate connection detail. This configuration is very easy to install and is cost-effective, thus lending itself as a viable solution for isolation in low-cost housing applications. The building is 7.2 m by 18.0 m in plan, and the height to the roof above the isolators is 12.8 m. The building accommodates eight apartment units. The walls that enclosed each unit are made out of unreinforced masonry with special seismic gaps filled with soft mortar. This seismic gap structurally separates the walls from the main frame.

The site-specific seismicity for this project was assessed by Beca Carter Holling and Ferner (BCHF) of New Zealand. They had earlier produced the response spectra incorporated into the Indonesian Code for the earthquake-resistant design of buildings (BCHF, 1979). The site-specific response spectra were developed for various levels of earthquake risk. Figure 1 shows five curves of acceleration spectra representing respectively a 20-year return period (serviceability check), a 200-year return period (recommended design - MDE), a 500-year return period, a 1000-year return period (Maximum Probable Earthquake - MPE), and a Maximum Credible Earthquake (MCE).

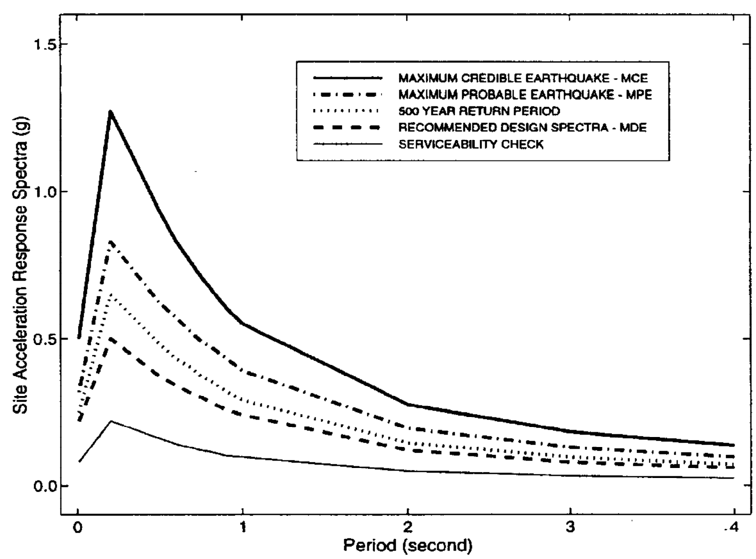

Fig. 1. BCHF site response spectra

The isolation system was designed so that the period of the isolated building is 2 seconds with 10% of critical damping during a MDE, and no catastrophic failure will take place in the event of a MPE. The design displacement of the isolator is 101 mm corresponding to 104% shear strain, and the displacement at the MPE input level is 184 mm equivalent to 189% shear strain. The bearings were designed and fabricated by the Malaysian Rubber Producers' Research Association (MRPRA) in Hertford, UK. Two different HDNR compounds and a single bearing size were used to achieve overall economy on fabrication, installation, and maintenance of the isolation system. The bearings were connected to the superstructure and foundation using recessed endplates which is simpler and more cost-effective than a standard bolted connection. The dynamic properties of the bearings were obtained from full-size bearing tests conducted at the Earthquake Engineering Research Center (EERC), University of California at Berkeley (Taniwangsa *et al.*, 1995).

There are currently no guidelines for seismically-isolated buildings in the Indonesian Codes. The superstructure of the building was designed according to the Indonesian Building Code by the Institute of Human Settlements (IHS) of Indonesia. It was designed as a fixed-base building, expected to behave elastically for a base shear of 0.07 g, which is about one third of the spectral value of 0.2 g corresponding to a 20-year return period. The Indonesian code allows the development of plastic hinges in a significant number of primary structural members at the full spectral value corresponding to a 20-year return period, and the structure has to satisfy life safety requirements for a spectral acceleration of 0.55 g (corresponding to the MDE). Minteck Exploration and Drilling Services, under the supervision of BCHF, determined the geological conditions underneath the building by drilling a borehole 65 m deep. There is a thin layer (about 0.5 m) of sandy silt with gravel covering unweathered rock layers that support the foundation of the building.

ANALYTICAL MODEL

A three-dimensional time history analysis of the building was carried out using the computer program 3D-Basis-Tabs (Nagarajaiah *et al.*, 1993). This program is a combination of the 3D general purpose program, ETABS, and a specific program to model isolation systems, 3D-BASIS. A mathematical model encompassing both the superstructure and the seismic isolation system was first developed. This model was then subjected to a series of recorded ground motions representative of the site response spectra for the MDE and the MPE seismic risk levels, as well as the serviceability checks.

Superstructure Model

The superstructure of the demonstration building was modeled as a three-dimensional frame with linear elastic behavior. The beams and columns were modeled using beam-column elements in ETABS. The stiffness of the structure was computed using cracked section properties of the beams and full cross-sections of the columns. The stiffness contribution of the unreinforced masonry walls was not taken into account because the interstory drifts of the base-isolated model were much less than the drifts of the fixed-based model. The seismic gaps, which were filled with the soft mortar that separated the walls and the structural frame, were large enough to accommodate the anticipated maximum interstory drifts due to the earthquake ground motions and isolating the masonry panels from the reinforced concrete frames.

The distribution of the structural elements at each floor was symmetrical in both directions, and the distribution of the mass was slightly asymmetrical in the longitudinal direction; however, the degree of eccentricity was very small, less than 3% of the length of the building. The story masses were lumped at the center of mass of each floor. The mass values included gravity loads and one-half of the live-load. The first nine modes of the fixed-base superstructure were calculated. Figure 2 shows the first three periods and their corresponding mode shapes. The first period is 0.62 seconds, corresponding to the longitudinal direction; the second

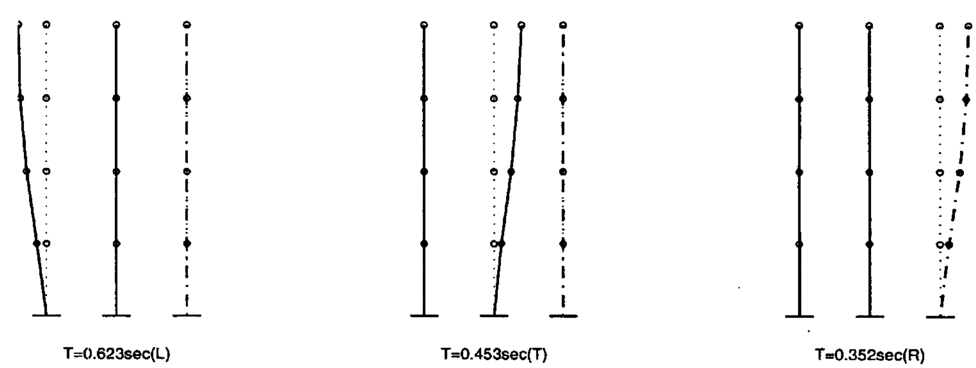

Fig. 2. First three mode shapes and periods of the demonstration building

mode is 0.45 seconds, corresponding to the transversal direction; and the third mode is 0.35 seconds, corresponding to the first rotational mode of vibration. These periods of the superstructure were sufficiently separated from the 2 second horizontal period of the isolation system. Five percent modal damping was assumed for all modes.

The model of the superstructure was verified by comparing the analytical studies and experimental results on a one-third scaled model of the lower portion of the frames that was built and tested at EERC (Taniwangsa *et al.*, 1996). The analytical model accurately predicted the response of the building under the set of ground motions representing various levels of seismic risk in the BCHF site spectra.

Isolation System Model

Two rubber compounds, to be designated as "soft" and "hard", were used to isolate the building. This allows for the use of one size of bearing to accommodate uneven column loads. Six soft bearings were located under the columns of the exterior frames in the transverse direction, while ten hard bearings were located under the remaining columns.

Each bearing was modeled as a combination of a linear spring element and a viscous damping element. The spring constant and the damping value were appropriately adjusted to account for input intensity. The damping value was also tuned to include the influence of the axial load on the bearing so that an accurate representation of bearing response was obtained over the range of inputs. The dynamic properties of the bearing elements were selected to match the force-displacement relationship obtained from the full-scale bearing tests. A typical shear force-displacement hysteresis curve of a bearing test (Fig. 3(a)) shows that the secant stiffness is linear and similar behavior is observed for all shear strain levels under different axial pressures. Figure 3(b) shows the relation between the shear stiffness and shear strain for the soft bearings under the design pressure. The assumptions in the modeling of these bearings were validated by real-time tests (Taniwangsa *et al.*, 1995).

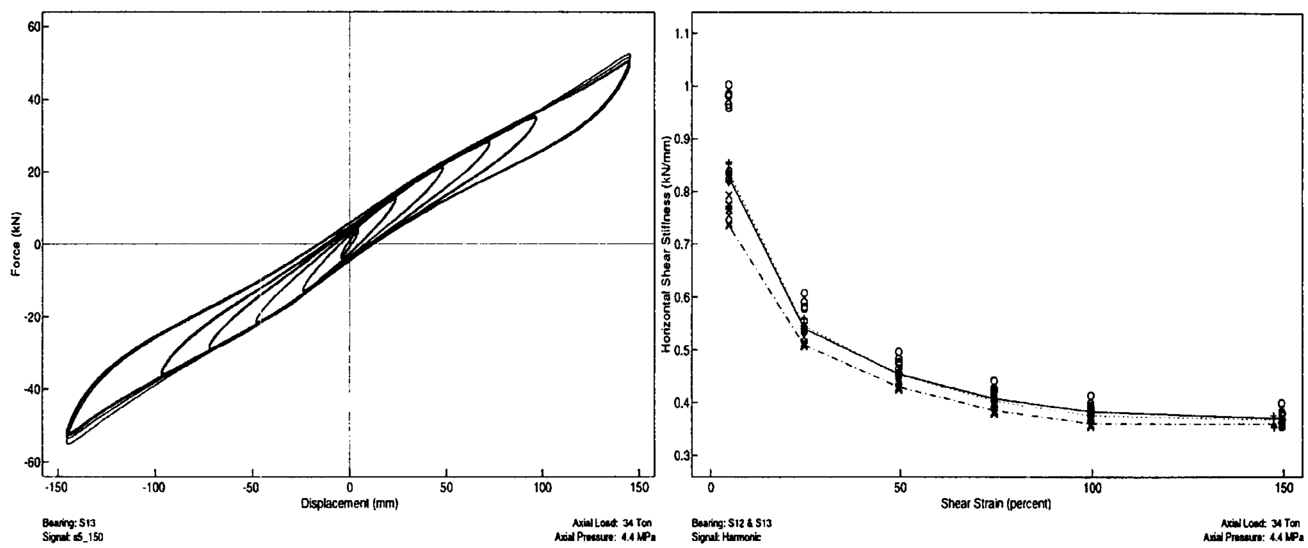

Fig. 3. (a) Bearing force vs. displacement and (b) Shear stiffness vs. shear strain

Selection of Input Ground Motions

The site response spectra for various seismic risk levels were developed by BCHF using a probabilistic approach because no previous strong motion records were available in the vicinity. These site-specific spectra were used as target spectra in selecting the input motions from the past earthquake record data base. In order to perform a three-dimensional time history analysis, the records should have two horizontal components and the elastic response spectra of those records should match the target spectra reasonably well, so that it is not necessary to conduct acceleration and/or duration scaling (Clark P., *et al.*, 1993). Table 1 shows the seven sets of ground motion used in the analysis. The 1985 Chilean and Michoacan earthquakes were selected because they were large earthquakes with a similar subduction zone mechanism to that of the Java trench earthquake source. The 1994 Northridge, the 1989 Loma Prieta, the 1986 Palm Springs, and the 1990 Uplands earthquakes, earthquakes with a wide range of sources and magnitudes, represent the possibility for moderate intensity strike-slip earthquake mechanism within the highly-faulted region of the building site.

In the analysis, the building was subjected simultaneously to the pair of ground motions, with the strongest component applied in the weakest direction of the structure, in order to get the results from the most rigorous case. A serviceability check is critical in assessing the response of a base-isolated structure because during small earthquakes, the horizontal force is often not large enough to bring the isolation system to the low stiffness levels anticipated at the design shear strain levels (see Fig. 3(b)), the isolation system is therefore not effective and at that point the building will behave more like a fixed-base structure.

Table 1: Ground motions used in the analysis

Earthquake	Date	Magnitude	Station	Epicentral Distance	Comp.	PGA(g)
Chile	3-Mar-85	Ms=7.8	Llolleo	4.5 km	N10E	0.66
					S80E	0.41
Michoacan	19-Sep-85	Ms=8.1	Caleta de	22 km	N00E	0.14
			Campos		N90E	0.14
Palm Springs	8-Jul-86	Ml=5.9	Desert Hot	12 km	N00E	0.30
			Spring		N90E	0.27
Loma Prieta	17-Oct-89	Ms=7.1	Corralitos	7 km	N00E	0.63
					N90E	0.48
Loma Prieta	17-Oct-89	Ms=7.1	Capitola	9 km	N00E	0.48
					N90E	0.42
Uplands	28-Feb-90	Ml=5.5	Rancho	12 km	N00E	0.14
			Cucamonga		N90E	0.10
Northridge	17-Jan-94	Ms=6.8	Pacoima dam	19 km	N175E	0.42
			Down-stream		N265E	0.44

ANALYTICAL RESULTS

Because the model used for the isolation bearings is strain dependent, the time-history analysis of the isolated building is an iterative procedure. First the expected strain in the isolators is assumed based on the intensity of the input ground motion. The values of elastic springs and viscous dampers representing the rubber bearings are chosen according to that estimated strain level. After the initial analysis is performed, the horizontal deformation at the isolation level obtained from time-history analysis is compared to the estimated strain values. If the assigned and calculated deformation levels at the isolation level are substantially different, the calculated deformation level is used to determine new parameters of the bearing model; the process is repeated until the assumed and calculated deformation levels are of the same order. The results of the analyses were presented in terms of the combined peak floor accelerations and the normalized peak accelerations. The combined peak floor displacement and the inter-story drifts are also discussed. The variation of the column loads during the earthquake should be verified to assure that the column loads remained in compression, because the recessed connection of the isolators would not be able to transfer the shear force to the superstructure under tensile loads.

Prediction of Maximum Accelerations

The time-history analysis was carried out for all selected ground motions shown in Table 1. The combined peak floor accelerations were obtained by combining the maximum floor acceleration of each direction using the square root of the sum square method. The peak ground accelerations of both direction were also combined by the same method to obtain combined peak ground acceleration. Figure 4(a) shows the distribution of the combined peak acceleration along the height of the building. The maximum acceleration responses on the superstructure corresponding to the input ground motion at the MPE risk level (an earthquake with return period of 1000 years) are around 0.2 g. According to the Indonesian Earthquake Code, spectral accelerations of 0.2 g correspond to the full spectral value of an earthquake with a 20-year return period, and the structure may develop plastic hinges in a number of primary members.

The Uplands earthquake recorded at the Rancho Cucamonga was selected to represent the serviceability level earthquake, which is a more frequent earthquake with a smaller spectral acceleration values. Usually under this level of earthquake, the isolation system is not effective because the earthquake forces are not large enough to pass the threshold of the other serviceability horizontal loads, i.e., wind. The maximum floor acceleration of the building due to these ground motions was around 0.05 g. For this level of response, the demonstration building should behave elastically because the superstructure was designed as a fixed-base building with elastic behavior under 0.07 g spectral acceleration.

Figure 4(b) show the normalized peak acceleration to the corresponding peak ground acceleration. The floor acceleration responses were always smaller than the PGA, including that for the smallest earthquake in this study (with a PGA of 0.10 g). The distribution of the normalized maximum accelerations due to the Uplands earthquake (serviceability check), and the Palm Springs and Caleta earthquakes (MDE) has an inverted triangle shape, while the distribution due to earthquakes selected to represent the MPE was more uniform.

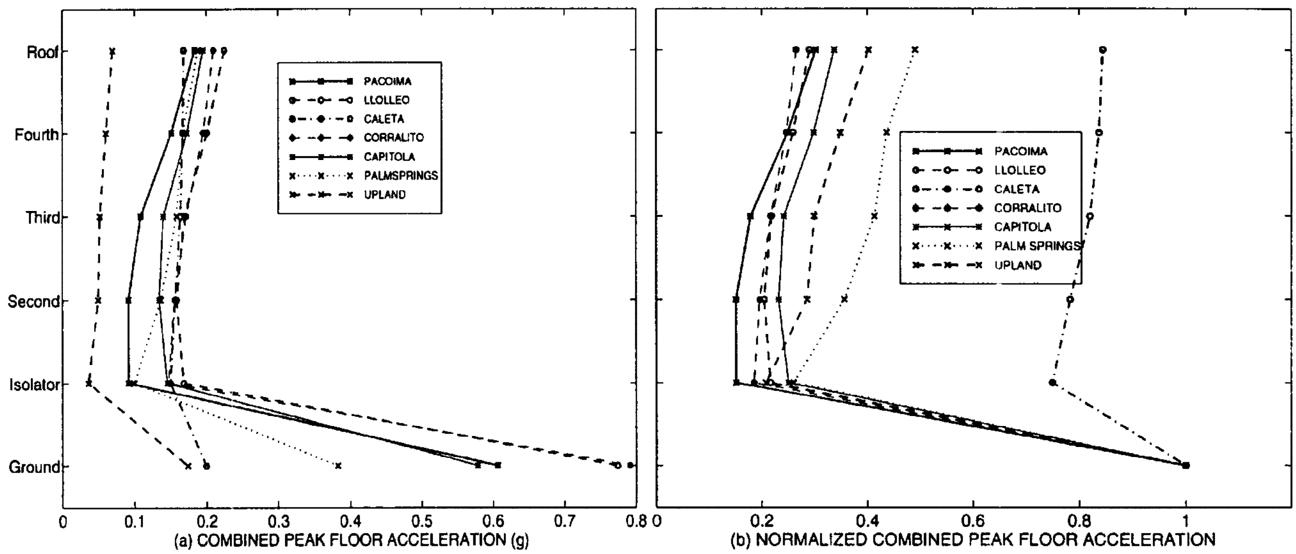

Fig. 4. (a) Combined peak floor acceleration and (b) Normalized peak acceleration

Prediction of Maximum Displacements

The combined peak floor displacements along the height of the building are shown in Fig. 5(a). The shapes of the maximum displacement distribution are similar to the theoretical first mode shape of the isolated building (Kelly, 1992) where a large portion of the building displacement is concentrated at the isolation level.

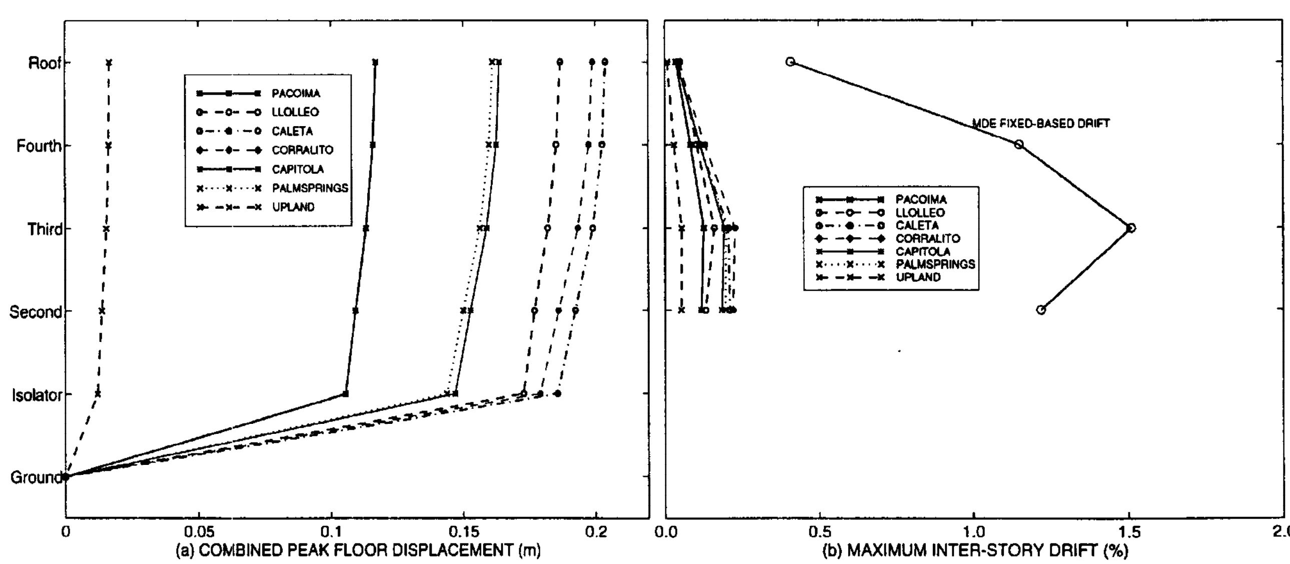

Fig. 5. Distribution of the maximum displacement

The maximum displacement of the isolation system due to all records in this study was less than the deflection at MPE level of 0.184 m. The isolator displacements due to the Caleta, Corralitos and Llolleo ground motions were very closed to this value. The displacement due to the Pacoima down-stream ground motion corresponded to the isolator design deflection, which was 0.104 m.

Figure 5(b) shows the distribution of the maximum interstory drifts. The drift of the base-isolated model is almost negligible, with a maximum value of less than 0.3% for all of the records. If the building were fixed-base instead of base isolated, the interstory drift due to a MDE ground motion could reach a value of 1.5%, which is five times larger than the drift of the base-isolated building. The interstory drift is a very important response parameter in the prediction of the performance of the non-structural elements and building contents, and, therefore, in the overall performance of the building.

Variation of Axial Loads on the Isolation Bearings

Because the connections between the isolation bearings and the building foundation were not traditional bolted but recessed, the contact area of the isolation bearings with the pedestal plates and the column end-plates became a very important issue. It was necessary to determine whether the axial load of a column changed sign during the earthquake, implying local column uplift. If the columns are in tension, the isolation bearings under these columns will not participate in carrying the horizontal shear force; and hence the earth-quake force will be redistributed to the other isolation bearings, remaining under compression.

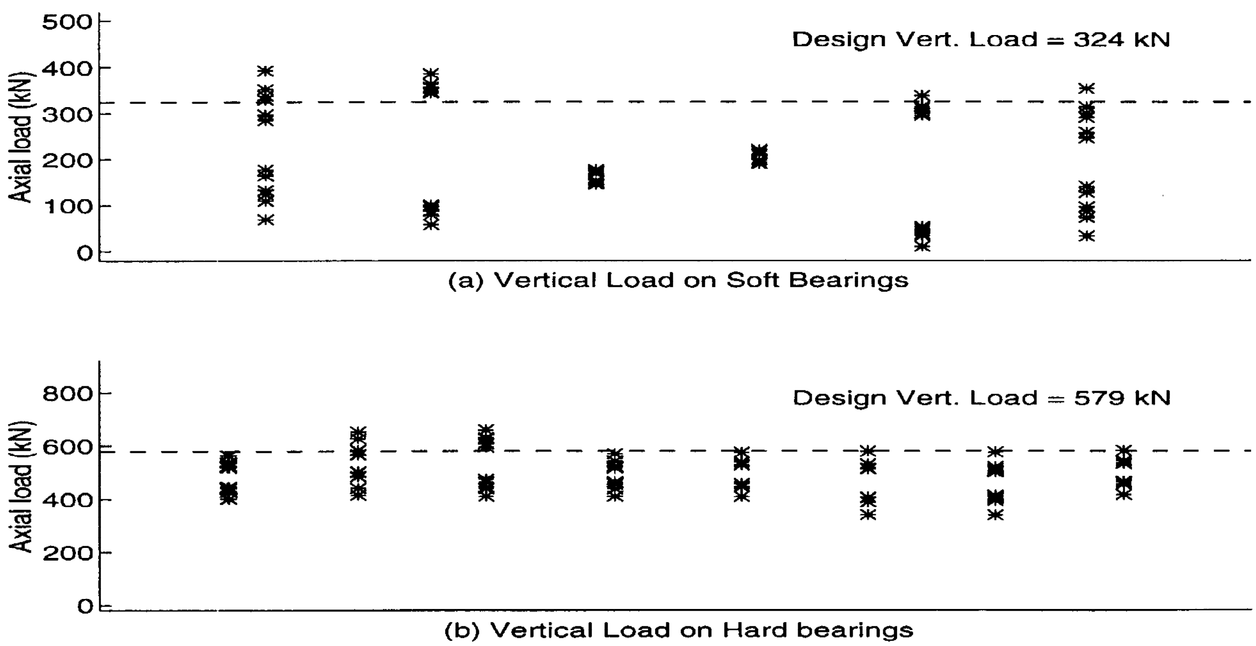

Fig. 6. Maximum and minimum vertical loads on the isolation bearings

In this study, the maximum and minimum axial loads in each column computed from the dynamic analysis were compared to the design axial load for each bearing type. Figure 6(a) shows the scatter plot of the maximum and minimum axial loads for the soft compound bearings. The axial loads in each column were always in compression. These bearings were located under the external transverse frames. The fluctuation of the axial load of these soft compound bearings was in the range of 20 to 392 kN. Figure 6(b) is a scatter plot of the maximum and minimum axial loads for the hard compound bearings. The axial loads remained under compression during the earthquakes, fluctuating +35% and -40% around the design axial load of 579 kN. At no time does any bearing go into tension, thereby eliminating the concern of column uplift and loss of bearing force-carrying capacity.

CONCLUSIONS

The results of the time-history analyses of an analytical model of the base-isolated demonstration building in Indonesia show that the HDNR bearings used in this building are very effective in isolating the superstructure from the ground motions with high PGA (above 0.6 g), usually for the ground motions representing the MPE. For this level of ground shaking, the combined peak roof accelerations were about 25 percent of the PGA. The effectiveness of the isolation system in reducing the shear force transmitted to the superstructure tended to decrease as the PGA decreased. When the PGA of the selected ground motions were in the range of 0.14 g to 0.30 g (a MDE risk level), the maximum roof accelerations were in the range of 60% to 90% of the PGA. The ground motion representing the serviceability check earthquake had a PGA of 0.14 g, and the maximum roof acceleration was only 40% of PGA. The distribution of the maximum floor acceleration over the height of the building due to the ground motions with high PGA tended to be more uniform, while for lower PGA the distribution was an inverted triangle shape.

In all cases there was no amplification of the PGA to the superstructure. The maximum floor acceleration on the superstructure corresponding to the input ground motion at the MPE risk level (an earthquake with return period of 1000 years) were around 0.2 g, which is comparable to the full spectral value of an earthquake with a 20-year return period for fixed-base design criteria.

The distribution of the maximum displacement is similar to the first mode shape of the isolated building, and in all cases, the displacements in the isolation bearings were smaller than the MPE deflection of 184 mm. The maximum interstory drifts were less than 0.3%, one fifth the interstory drift of the fixed-base building under a MDE earthquake. This study also verified that all the isolation bearings remained in compression under the all selected ground motions.

A well designed, fabricated, installed, and maintained HDNR isolation system can be an excellent choice in providing protection from strong earthquakes for public buildings, such as housing, schools, and hospitals. Isolation reduces the floor accelerations and interstory drifts to ensure earthquake protection beyond life safety, and provides better performance in terms of building contents, non-structural elements, and serviceability after major earthquakes.

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