SEISCMIC-ISOLATION BUILDINGS UNDER LARGE-MAGNITUDE EARTHQUAKE MOTIONS

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ABSTRACT :

Most urban areas in the world have been developed on alluvial plains, where earthquake motions having long period components may be excited under distant but large magnitude earthquakes amplified by the surface geology as deep as a few thousand meters. This paper reports the response of 10-story reinforced concrete base-isolated buildings to ground motions simulated for large-magnitude earthquakes. If columns are assumed to lose vertical load carrying capacity after the design shear force is developed, the structure collapses in the upper stories after large response deformation is developed in the seismic isolation layer; i.e. even low stiffness of the seismic isolation device develops large shear after such deformation. If the pounding of the first floor of the super structure against surrounding retaining walls is considered in the analysis, the collapse occurs in the lower stories. It is essential even in the design of base isolated buildings to provide ductility on the basis of the weak-beam strong-column concept against unexpected disturbances.

KEYWORDS: Base isolation, pounding against retaining walls, brittle collapse, ductility, long-period long-duration motions

1. INTRODUCTION

Performance objects of a building, in addition to structural damageability and safety, are more emphasized in recent structural design such as the uninterrupted use of the building, the protection of non-structural elements, the prevention of furniture and mechanical facilities from overturning and the reduction of occupants' uneasiness due to excessive oscillation. Therefore, the application of seismic isolation devices gained popularity in seismic design of buildings especially in Japan after the 1995 Kobe earthquake disaster. The flexible seismic isolation devices can elongate the natural period of a structure beyond the dominant period range of an earthquake motion and reduce the acceleration response of the super structure.

Most urban areas in the world have been developed on deep alluvial plains, where long period components of earthquake motions might be amplified by the surface geology as deep as a few thousand meters during a large magnitude earthquake occurring along tectonic boundaries. Such long-period motions were indeed observed during the 2003 Tokachi-oki earthquake. Therefore, the safety of long-period structures such as base-isolated buildings, high-rise buildings, and long-span bridges needs to be examined. The Japan Society of Civil Engineers (JSCE) and Architectural Institute of Japan (AIJ) launched a cooperative research program on this issue in October 2003.

The objective of this paper is to study the response of base-isolated buildings under possible earthquake motions suggested by the JSCE-AIJ cooperative program. Large oscillation in a base-isolation layer might cause the pounding of the structural base against surrounding retaining walls, or might develop large shear in the isolation device due to even low elastic stiffness. Such large response may cause significant damage in the super structure.

2. PROTO-TYPE BASE-ISOLATED BUILDING

2.1 Super Structure

A part of a ten-story reinforced concrete moment-resisting frame building, including continuous columns and their adjacent girders cut at the mid-span, is represented by a fish-bone model as shown in Figure 1. The inter-story height is 4.00 m in the first story, and 3.50 m from the second to the top story. The column dimensions are 600x600 mm from the first to fourth story, 550x550 mm from the fifth to seventh story, and 500x500 mm from the eighth to tenth story; and the beam dimensions are 500x2000 mm in the first floor and foundation, 400x800 mm from the second to fifth floor, 400x750 mm from the sixth to eighth floor, and 350x700 mm from the ninth to roof floor. The floor mass is assumed to be 49 ton for 49 m^2 tributary floor area of a column. The beam-column connection, part common to beams and columns, and the first-floor girders are assumed rigid. The elastic modulus of concrete is 23.0 kN/mm^2 for the concrete nominal strength of 24.0 N/mm^2.

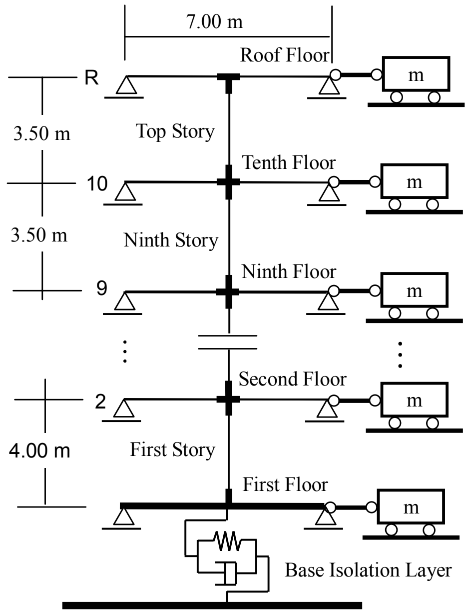

Figure 1 Fishbone structural model

2.2 Characteristics of Base Isolation Layer

The base-isolation layer consists of linearly elastic laminated natural rubber bearings and elasto-plastic lead dampers. The base isolation devices are designed in accordance with the Japanese Building Standard Law. The allowable maximum lateral deformation δ_s of the isolation layer is 416 mm based on the properties of the rubber bearings and the lead dampers. The stiffness of elastic rubber bearing per column is selected 1,740 kN/m to yield the effective period close to the dominant period of earthquake motion (approximately 3.0 sec); the lead damper behaves elastically to the yield deformation of 7.33 mm and yield resistance of 175 kN, and then shows perfect plastic behavior without additional resistance (Figure 2).

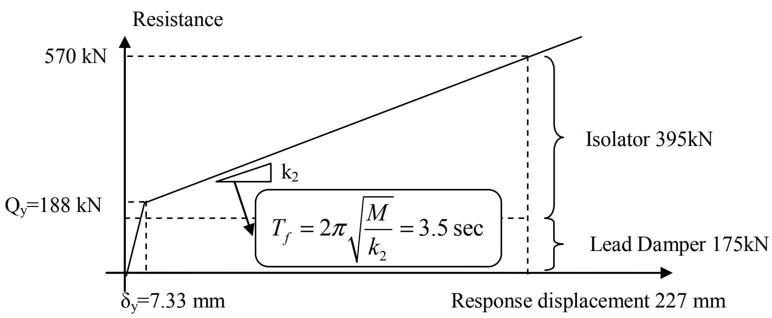

Figure 2 Resistance-displacement relation of the isolation layer per column

Design earthquake force Q (kN) in terms of lateral shear in the seismic isolation layer must be calculated by the following expression;

$$Q=S_A\, G_s\, M\, F_h\, Z \tag{1}$$

in which, S_A (=$5.12/T_{eq}$, where T_{eq} is equivalent period of the isolation layer greater than 0.64 sec) is acceleration spectrum value (m/sec^2) of design earthquake motion at the engineering bedrock (firm soil layer having shear wave velocity greater than 400 m/sec), G_s: amplification factor of earthquake motion from the engineering bedrock to the structural base (=1.0 for assuming the isolation layer placed on the engineering bedrock), M: total mass (=539 ton) of the super-structure, F_h: acceleration response reduction factor by the effect of damping, and Z: seismic zone factor (=1.0 assuming Nagoya).
The design shear and corresponding response deformation of the isolation layer must be determined by iteration (see Figure 3). A response displacement of the isolator layer is first assumed less than the allowable deformation δ_s. An equivalent stiffness K_{eq} is determined as the secant stiffness of the isolation layer. Equivalent period T_{eq} is estimated for the equivalent stiffness K_{eq} and the total mass M of the super structure. The design spectral acceleration S_A is evaluated for the equivalent period T_{eq}. The equivalent damping factor h_d of hysteretic damping devices is evaluated at the assumed response displacement, and the acceleration response reduction factor F_h is estimated by the following expression;

$$F_h = \frac{1.5}{1+10\,h_d} \quad \leq 0.4 \tag{2}$$

If the design shear Q in the isolation layer, determined by Eq. (1), is not equal to the shear of the isolation layer at the assumed response displacement, the process is iterated.

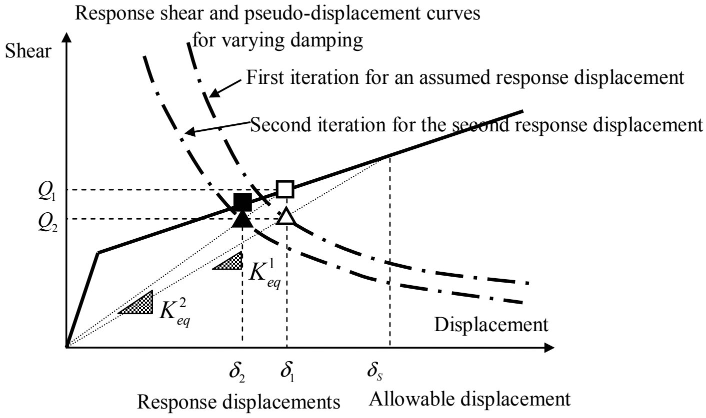

Figure 3 Determination of design shear and response deformation of the isolation layer

The response displacement of the isolation layer is evaluated to be 227 mm after the sixth iteration and the equivalent damping factor of the isolation layer is 0.15 for the displacement. The Japanese Building Standard Law requires that the clearance between the first floor and the surrounding retaining walls should be determined considering (a) possible variation in the stiffness of isolators and dampers with age, (b) scatter of material quality, (c) temperature change, (d) unexpected torsional oscillation and (e) other unstated safety margins. Therefore, the final clearance at construction should be not less than 500 mm for the expected response displacement of 227 mm. In this study, however, these safety margins are not considered in the analysis. Therefore, the clearance was chosen to be 250 mm slightly larger than the expected response displacement of 227 mm.

2.3 Design of Super Structure

The design shear of the isolation layer should be distributed along the height of the super structure. The Japanese Building Standard Law requires that the structural members of a super structure should remain elastic under the design lateral forces amplified by 1.3 to consider the uncertainty associated with material properties of isolation devices and decay in stiffness properties with age. The shear in the isolation layer at the response displacement is the sum of 175 kN of the lead damper and 395 kN developed in the elastic rubber isolator (see Figure 2).

Therefore, design base shear of 740 kN (=1.3x569 kN) is distributed along the height of the super structure. It should be noted that the shear carried by the elastic isolator at the expected response displacement is more than twice the shear resisted by the lead damper.

2.4 Pounding of Super-structure and Retaining Walls

The pounding of the first floor above the isolation layer and surrounding retaining walls is modeled by the linear visco-elastic model (Zhu, Abe and Fujino, 2002), represented by parallel linearly elastic spring and viscous damper between two masses (Figure 4). The damping coefficient is expressed by the following equation using the repulsion coefficient e;

$$c_{cnt} = 2\frac{-\ln e}{\sqrt{\pi^2 + (\ln e)^2}}\sqrt{k_{cnt}\frac{m_1 m_2}{m_1 + m_2}} \tag{3}$$

The stiffness k_{cnt} of the retaining walls is assumed to be 1.5 times the first-mode effective stiffness of the super-structure fixed at the base; i.e. k_{cnt}=31,500 kN/m. The mass m_2 of retaining walls is assumed infinite, and the mass m_1 of the super-structure is equal to the first-mode effective mass (m_1=539 ton). The repulsion coefficient e is assumed to be 0.4 and the damping coefficient c_{cnt} of the pounding model is evaluated to be 2,310 kN/sec/m by Eq. (3). The clearance is 250 mm.

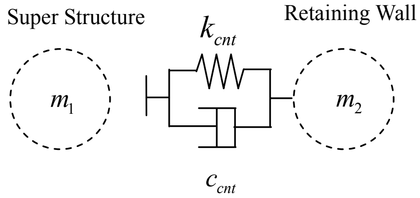

Figure 4 Linear viscoelastic pounding model

3. LONG-PERIOD LONG-DURATION EARTHQUAKE MOTION

The JSCE and AIJ program collected 13 simulated earthquake motions for Tokyo, Yokohama, Nagoya and Osaka sites under possible earthquakes of magnitude around 8 occurring in the Pacific Ocean off east coast of Japan. The simulated earthquake motions lasted 200 to 650 sec; the maximum acceleration amplitude ranged from 900 to 4,990 mm/sec^2. Out of 13 records, the C-SAN-EW motion, simulated at the Nagoya Castle under a possible combined Nankai and East Nankai trough earthquake, caused by far large response. Therefore, this record was used in the study; the maximum ground acceleration was 1,895 mm/sec^2, maximum ground velocity was 510 mm/sec and the duration was 328 sec (Figure 5).

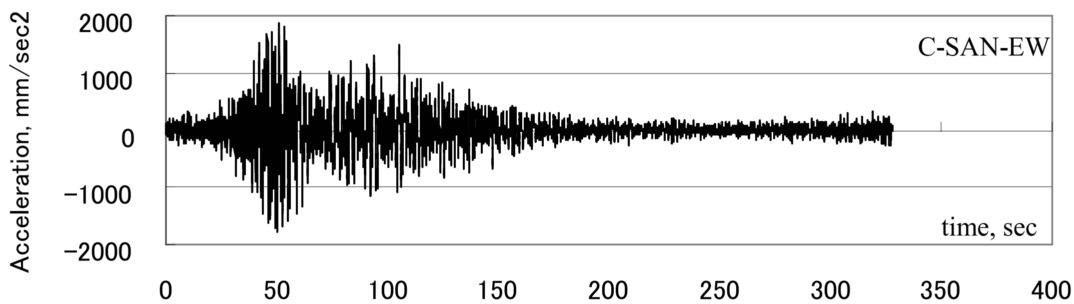

Figure 5 Acceleration waveform of the C-SAN-EW record at Nagoya Castle

The velocity response spectrum (damping factor 0.05) is shown in Figure 6. The response amplitude peaks at around 3.0 sec with velocity response of 2570 mm/sec. The design earthquake force in terms of velocity response spectrum is 815 mm/sec for effective period longer than 0.64 sec in the Japanese Building Standard Law. In other words, the C-SAN-EW motion developed structural response more than three times larger than that required by the Japanese Building Standard Law.

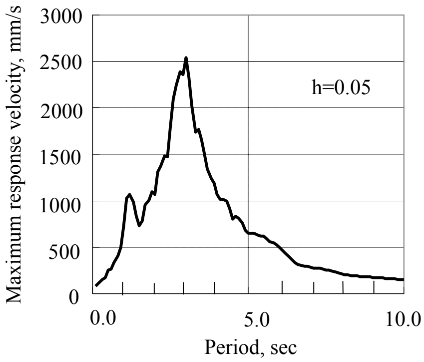

Figure 6 Velocity response spectrum of the C-SAN-EW record in Nagoya Castle (h=0.05)

The maximum response displacement of 446 mm is calculated in the base-isolation layer of the proto-type building at around 50 sec of the record. Therefore, the first floor must have collided against the surrounding retaining wall if this earthquake motion were to occur at the construction site.

4. RESPONSE OF BRITTLE BUILDINGS

4.1 Brittle Failure of Reinforced Concrete Columns

Some reinforced concrete buildings, during the 1995 Kobe Earthquake Disaster, collapsed in a story by the loss of vertical load carrying capacity of columns after their shear failure. A column of the super structure is assumed to lose lateral resistance when shear in the column reaches the shear under the design lateral force, and then the story collapsed when the gravity load resistance was lost in the column (Figure 7). The stiffness changes at flexural cracking and shear failure, then the lateral resistance decays after the shear failure and the column loses the vertical load resistance at collapse point. The girders in all floor levels are assumed to be rigid in this section.

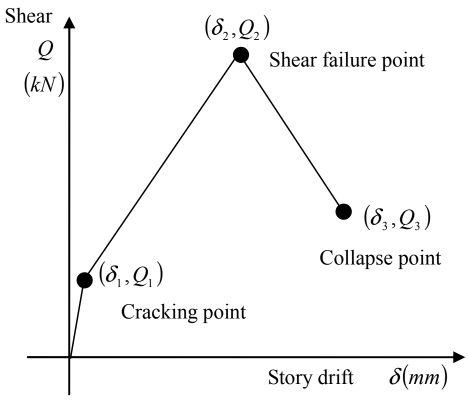

Figure 7 Story shear and drift relation of brittle columns

The initial stiffness of a story is evaluated as elastic flexural stiffness using gross section and clear height of the column. Shear at cracking is calculated for flexural cracking moments at the top and bottom of the column. The secant stiffness at the shear failure is 1/4 of the initial stiffness. Column shear at shear failure is equal to the shear calculated under design seismic force. Column shear at collapse is 0.4 times the shear at shear failure. Story drift angle (inter-story displacement divided by the column height) at collapse is assumed to be 0.012 in the first and

second stories, 0.0105 in the third and fourth stories, 0.009 in the fifth and sixth stories, 0.0075 in the seventh and eighth stories, and 0.006 in the ninth and tenth stories (Yoshimura and Takaine, 2005).
Hysteresis rules of story shear and drift relation followed the Pivot model (Dowell, Seible and Wilson, 1998) with some modifications to represent cracking behavior.

4.2 Response with and without Pounding

Response story ductility factor, defined as a ratio of maximum story drift to the story drift at shear failure, is shown in Figure 8 subjected to the C-SAN-EW motion.

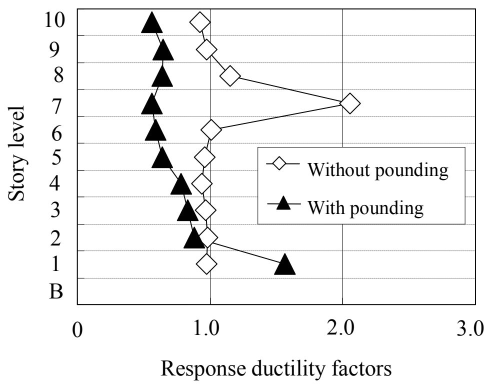

Figure 8 Story ductility factors of brittle structures under the C-SAN-EW motion

If the pounding is not considered in the first floor against the retaining walls, the ductility factor in the seventh story significantly exceeds unity, indicating the story collapse, at as early as approximately 8 sec of the motion. Ductility factors in the other stories are slightly less than but close to unity; the other stories might have collapsed even if the seventh story does not. The story drift in the isolation layer reaches 313 mm, exceeding the expected design drift. Even if the elastic stiffness is small in the elastic rubber bearing, the large displacement causes large shear in the isolation layer. Corresponding increased story shear in the super structure causes story collapse.
If the pounding is included in the analysis, the first story collapses due to the impact of pounding at as early as 6.7 sec of the earthquake motion. It is interesting to note that the inter-story drift of the super structure is slightly smaller with pounding than that without pounding. The collapse in the first story limits the input forces in the stories above.
Therefore, it is essential even in the design of a base-isolated structure that the super structure should be provided with some ductility in order to survive unexpected disturbances.

4.3 Effect of Stiffness of the Elastic Rubber Bearing

The stiffness of the elastic rubber bearing controls the dominant period of the base isolated structure. The elastic stiffness of the rubber bearing is varied to yield effective periods of 2.5, 3.5 and 4.5 sec. The damping factor in the isolation layer is maintained to be 0.15 by varying the yield resistance of lead dampers to 276, 175 and 143 kN, respectively. In other words, the design seismic force of the super structure decreases with the effective period due to the reduction in elastic stiffness of rubber bearings and reduction in yield resistance of lead dampers. The expected response displacements under design code are 162, 227 and 292 mm, respectively. Without pounding, the collapse in the upper story (first, seventh and sixth story, respectively) occurs at response displacement of 251, 311 and 324 mm, respectively, in the isolation layer (Figure 9).

4.4 Effect of Yield Resistance of the Lead Damper

The yield resistance of lead dampers is varied to 110, 175 and 215 kN to yield damping factors of 0.075, 0.15 and 0.20, respectively, at the code expected displacement in the isolation layer. The elastic stiffness of rubber bearings is set to make the effective period to be 3.5 sec. The expected response displacements of the isolation layer, 359, 227 and 174 mm, respectively, decrease with yield resistance of lead dampers. The design seismic force of the super structure decreases with increasing yield resistance of lead dampers because the shear in the isolation layer is governed by the shear in rubber bearings which decreases with reduction in the response displacement. The collapse in the upper story (eighth, seventh and ninth story, respectively) without pounding occurs in the three buildings before the end of the motion (Figure 10).

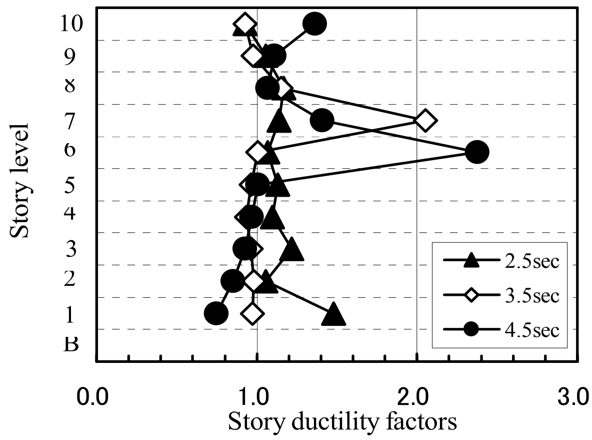

Figure 9 Effect of stiffness of rubber bearing

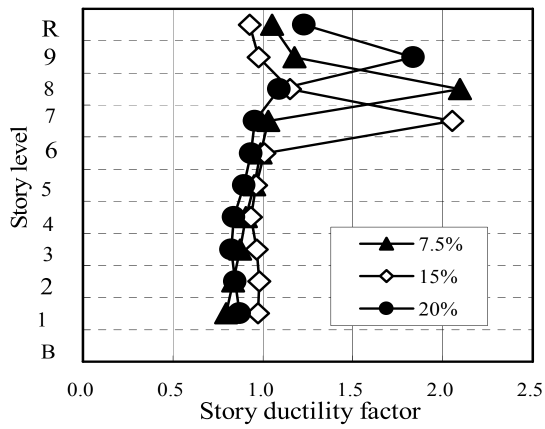

Figure 10 Effect of yield resistance of lead damper

5. RESPONSE OF WEAK-BEAM STRONG-COLUMN BUILDINGS

The proto-type structure is re-designed by the weak-beam strong-column concept. The flexural resistance of girders is set equal to the bending moment under the design lateral forces while flexural strength of columns is set equal to 1.3 times the bending moment under the design lateral forces. The member end moment-rotation relation under monotonic loading is represented by a trilinear relation with cracking moment equal to one-third yield moment and the secant stiffness at yielding equal to one-quarter of the elastic stiffness. The hysteresis relation follows the Pivot model (Dowell, Seible and Wilson, 1998). Strength sufficient to prevent brittle shear and bond failure is assumed in all members. The elastic stiffness of rubber bearings corresponds to the effective period of 3.5 sec, and equivalent damping factor of lead dampers is 0.15 in the analysis.
Ductility factor of girders without pounding against retaining wall is slightly larger than unity in the 10th floor, and less than or equal to unity in the other floors (Figure 11). No flexural yielding occurs in the columns. The drift in the isolation layer reaches 284 mm, and the weak pounding must have occurred if the pounding were included in the analysis. The girder ductility factors with pounding increase slightly, but the pounding effect is small. No flexural yielding occurs in the first story column.

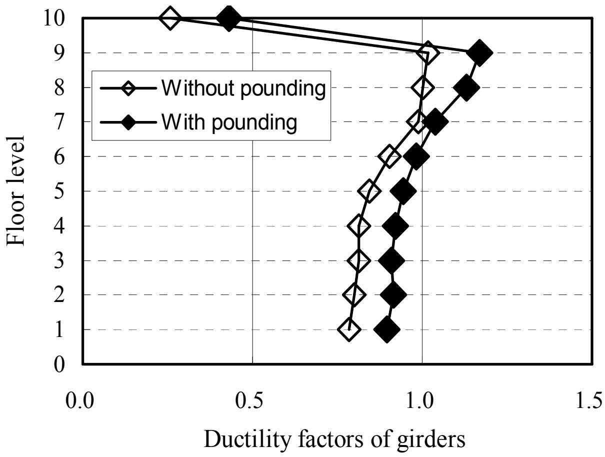

Figure 11 Girder ductility with and without pounding

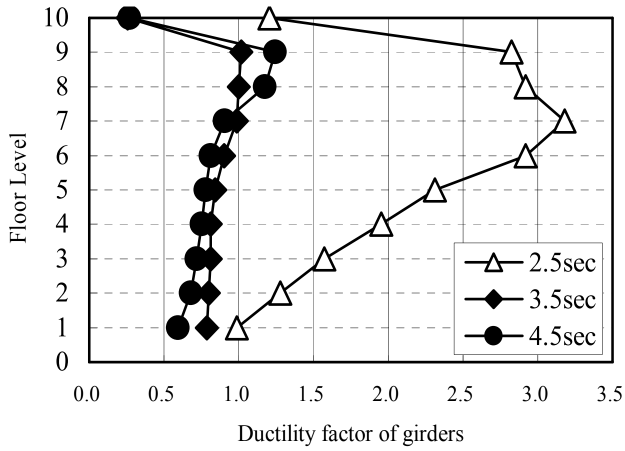

Figure 12 Effect of elastic stiffness of rubber bearings

The effect of the elastic stiffness of rubber bearing is studied by varying the effective periods from 2.5, 3.5 and 4.5 sec. If the elastic stiffness of rubber bearing becomes high, then the shear in the rubber bearing increases rapidly

with drift in the isolation layer, developing higher shear in the super structure. No pounding is considered to study the effect. The yield strength of lead damper is selected to yield damping factor of 0.15 at the expected displacement in the isolation layer.
The girder ductility factors are slightly larger than unity in eighth and nine floors if the effective period is 4.5 sec (Figure 12). The girder ductility factors, however, increase sharply if the effective period is made 2.5 sec because large lateral forces are developed in the super structure due to increased shear in the isolation layer and because the fundamental period of the structure becomes longer with inelastic deformation and approaches closer to the dominant period of the earthquake motion. The maximum girder ductility factor is slightly larger than 3, but such inelastic deformation can be easily accommodated at the end of reinforced concrete girders.

6. CONCLUSION

The response of 10-story reinforced concrete base-isolated buildings was calculated under long-period earthquake motions anticipated in the subduction zone off east coast of Japan. If reinforced concrete columns are assumed to reach shear strength at the shear corresponding to design seismic forces and then lose vertical load carrying capacity (collapse), the collapse in the upper stories occurs at early stage of earthquake motion, even without pounding of the first floor against surrounding retaining walls. The large displacement response in the isolation layer caused a significant increase in shear in the isolation layer due to the elastic stiffness of elastic rubber bearings, which in turn increased response shear acting in the super structure. On the other hand, the pounding causes large impact force in the first story and caused the collapse in the first story.
Design codes for base isolated buildings normally require clearance between the first floor of a super structure and surrounding retaining walls sufficient to avoid pounding. It is essential that the super structure should be provided with resistance and deformation capability to satisfy the response demand corresponding to large shear developed in the isolation layer at the code specified clearance. For this reason, even in the design of a base-isolated structure, the super structure should be provided with some ductility in order to survive unexpected disturbances.

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