DAMAGE-TOLERANT DESIGN IN SHANGHAI JINMAO-TYPE BUILDINGS

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SUMMARY

In this study, conceptual design of mega-sub structure and damage-tolerant design are implemented into the Shanghai Jinmao-type buildings. Numerical studies via ETABS indicate that the proposed implementation could effectively reduce various dynamic responses of such type of buildings, consequently enhancing their structural safety under severe earthquakes. In addition, the proposed implementation is not only be cost-effective (e.g., no extra devices and maintenance are needed in comparison with other solutions such as installation of various control devices), but also easy to be used in practice.

INTRODUCTION

One of the major issues in design and construction of medium-height to ultra-tall structures is the problem of ensuring the structural safety under seismic and wind loads. In addition to meeting the requirements for structural safety, wind-induced vibration must be constrained below the threshold of human comfort particularly during a sustained strong wind environment. Some high-rise structures are designed with installation of control devices, which is certainly one of the solutions for suppressing structural vibrations and hence for improving structural safety as well as human comfort. Both hybrid- and tuned-mass-damper systems have demonstrated their feasibility for high-rise building applications. However, as a building gets taller and more massive, a heavier additional mass is required and a larger stroke of this mass is anticipated in case of passive systems, and reliable and powerful actuators together with advanced optimal control algorithms are needed for active systems. In either case, these requirements raise serious safety and maintenance concerns and they also impose a high cost of the implementation of the tuned-mass-damper system.

Instead of using extra mass and mechanical devices for high-rise structures, the priority for practical building designers is the cost-effective and performance-based design with fully utilizing structural configuration and materials themselves. The design of this kind of structure, namely the composite structural systems (CSS), strongly relies on the emerging advances of contemporary vibration control theory as well as its applications to high-rise structures.

Recently, outrigger and mega-sub systems find the most creative application in newly-constructed (1998) Shanghai 88-story Jinmao building, among others (e.g., Xiamen 88-story building, Amin and Qi, 1999). In particular, the outrigger supports serve to link the central reinforced concrete core to the exterior composite mega-columns. It has been shown by the designers (Korista et al., 1996 and 1997) that the CSS of the Jinmao building represents a very efficient and unique structural solution to complex site conditions such as typhoon winds, moderate seismicity, and poor soil conditions. In addition, all the major structural elements will remain elastic under moderate earthquake (since Shanghai is classified as the seismic zone with a degree 7 intensity, which is slightly less intense than Zone 2A per the Uniform Building Code).

While the outrigger and mega-sub systems for high-rise buildings in general and the CSS of Jinmao building in particular could be used worldwide in future, an immediate question is raised: how is this type of CSS applied to a broad spectrum of building structures to resist large-magnitude near-field earthquake ? This will inevitably

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face the challenges related to structural inelastic responses and structural damage. Another follow-up question is what kind of damage is tolerant in order for the key-structures to not suffer damage under moderate near-field motion and not collapse under severe motion.

In this study, conceptual design of mega-sub building (Feng and Mita, 1995; Zhang, 1997; Shinozuka, 1998) and damage-tolerant design (e.g., Reina and Normile, 1997) are implemented into the Jinmao-type building. The seismic performance of the proposed design is investigated.

IMPLEMENTATION OF DAMAGE-TOLERANT DESIGN

The essence of damage-tolerant design is to sacrifice non-key structural components to leave the whole structure in general and key components in particular with little or no damage under a severe earthquake. It is known that the damage starts when structures exceed their elastic limit, beyond which the behavior of the structure becomes highly nonlinear, characterized by a hysteresis-type, force-displacement relationship. Therefore, if the materials of those non-key components could reach non-linear plastic range under a severe earthquake before all the key components, part of seismic energy exerted on the structure can then be absorbed by the non-key components, and consequently the seismic hazards on the structure mitigated. As long as the damaged non-key components could be easily and economically fixed or replaceable after the earthquake, such a damage is tolerant from an engineering viewpoint.

The aforementioned DTD was implemented into the CSS for a high-rise building that is based on the prototype of Shanghai 88-story Jinmao Building (Zhang et al., 1998).

Specifically, an ETBAS model for the high-rise building was constructed, as shown in Fig. 1. It consists of 88 stories above and 5 stories under the ground. The total height above the ground is 402 m. The outrigger frames are located in the stories 21, 51 and 86, consisting of braces that go through three local stories, and connecting mega-columns and central core structure. The central core structure is composed of four sub-cores with rectangular shape at four corners. The cross-section of each sub-core has major portion of shear walls with minor portion open, which is connected by short link beams in each floor. The four sub-cores are then connected by long link beams to form stiff central mega-core, resisting lateral loads.

In order to apply DTD concept to this structure, long link beams between sub-cores are chosen as non-key components. In that case, the stiffness of each sub-core including those short link beams and walls will still be designed normally, i.e., to be relatively strong to form a stiff sub-core and thus to resist the lateral loads. However, the selected non-key link beams will be designed to be relatively soft, in comparison with other gravity load-resisting elements such as mega-columns and walls. Consequently, under a severe earthquake, these selected non-key link beams will yield first before all the other key components and thus take the seismic damage to protect the main structure in general and key structural components in particular. From the vibration control point of view, these link beams function as dampers, absorbing part of seismic energy input to the structure. Since all the long link beams in each and every floor are selected as non-key components, it is thus expected that the total energy absorbed and consequent damage taken by these beams are not negligible, compared with other energy dissipation mechanisms (e.g., inherent damping). Even if the link beams are damaged, while not collapsed locally, under a severe earthquake, they could always be easily replaced after the earthquake and have thus no influence on the later use of the building. In contrast, the structure will no longer be reliable and functional if key structural components such as mega-columns are damaged.

More specifically, the selected non-key link beams will play the role of uni-axial plasticity elements in the whole structure, which have the hysteretic behavior. The nonlinear properties of these selected link beams are controlled by the parameters for the post yield stiffness, the shape, and the width of the hysteresis loops, which follows the hysteretic model developed by Wen (1976).

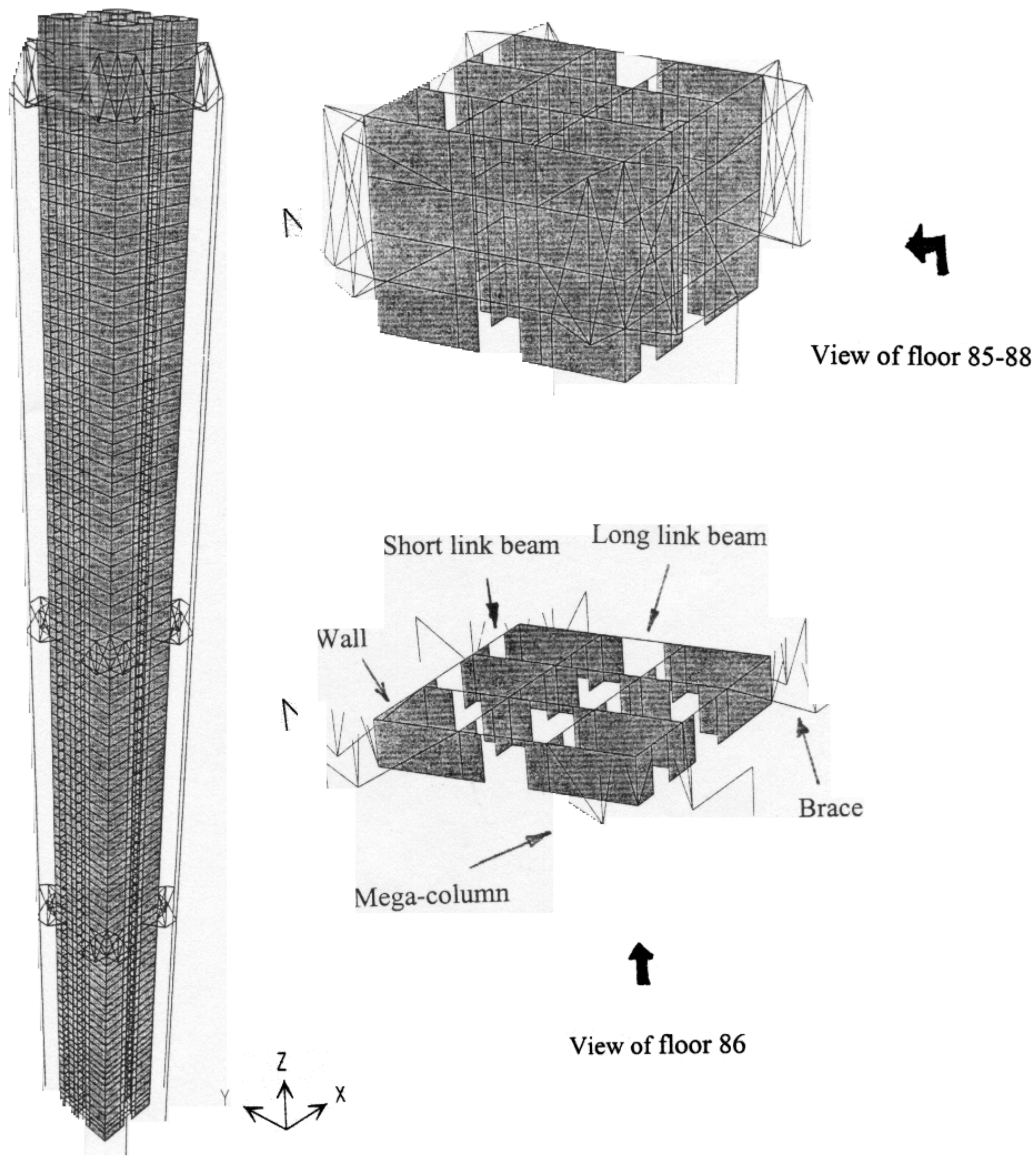

Fig. 1 ETABS model for 88-story building structure and its typical cross sections

NUMERICAL EXAMPLE

As a numerical example, seismic performance of two buildings with and without use of the aforementioned nonlinear link beams is investigated. The nonlinear beams are designed on purpose with relatively-soft stiffness, compared with the corresponding normal elastic link beams. In order for the two buildings to have similar dynamic behavior and thus be comparable in dynamic responses to the same input, the fundamental periods of the two buildings are selected to be almost equivalent. This requires that the mass of the nonlinear link beams be accordingly designed to be slightly less than that of the normal elastic link beams, which is possible through such a design of reduced beam section moment connections (e.g., Jiang, 1998, and Engelhardt, 1999). With such a dynamic equivalent criterion, the first six fundamental periods of the structures without and with nonlinear link beams are listed in Table 1. They are very close each other, indicating that the two buildings are comparable in dynamic responses to the same input.

Table 1: Fundamental periods of two buildings

Period (sec)	1st	2nd	3rd	4th	5th	6th
Building without nonlinear beams	8.9428	8.6721	4.5085	2.1342	1.9136	1.4848
Building with nonlinear beams	8.9445	8.6738	4.5140	2.1353	1.9147	1.4869

Re-scaled (i.e., peak-enlarged) Loma Prieta earthquake records are used as the input, as shown in Fig. 2. The total input energy received by the buildings from the earthquake ground motion is plotted in Fig. 3, which is function of time. The input energy is then distributed in the form of kinetic energy, potential energy, and energy loss due to viscous damping (i.e., inherent damping in the materials used in the structure such as reinforced concrete) and to nonlinear hysteresis. It can be seen from Fig. 3 that with the use of nonlinear link beams, part of total input energy will be dissipated via nonlinear hysteresis, which is obviously null in the building with the elastic linear link beams. Consequently, the kinetic and potential energy of the building with nonlinear link beams will be reduced to a certain extent. Since the kinetic and potential energies are explicitly related to the displacement and velocity responses of the building and implicitly to the acceleration and internal force responses, it is thus expected that the seismic responses will be smaller for the building with the nonlinear link beams than that with the elastic link beams. This point is confirmed by Figs. 4 and 5, which show the base shear time histories in the x- and y-directions. It is of interest to note that the reduction of base shear in the y-direction is much greater than that in the x-direction. Observing that the peak input acceleration in the y-direction is larger than that in the x-direction as shown in Fig. 2, Figs. 4 and 5 imply that the effectiveness of the proposed implementation will be significant to the large-magnitude earthquake event. This is generally true since the larger the ground motion intensity, the more the input energy received by the structure and thus the more the energy dissipated by the nonlinear hysteresis in addition to other types of energy such as damping energy. Fig. 6 depicts the trajectories of the base shear of two buildings, which provides a clear picture of the effectiveness of the proposed implementation in terms of base shear. By carefully examining Fig. 6, one might also conclude that the large reduction of base shear in the y-direction for the building with nonlinear beams is probably the compromise of small reduction of base shear in the x-direction. Fig. 7 displays the maximum x- and y-displacement responses in each floor, while Fig. 8 presents the maximum response drift ratios in each floor. These figures evidently indicate that the building with the use of nonlinear link beams can reduce the peak displacement and drift ratio responses to a certain extent, in comparison with the building without implementation of the nonlinear link beams.

While the aforementioned example demonsrtrates that the implementation could efficiently reduce various seismic dynamic responses, an equally important and usually dominant issue in the high-rise building design is the wind-resistant problem such as human comfort. Therefore, the proposed damage-tolerant design must be verified and/or validated by the wind-resistant design. Since the wind loads have usually much lower dominant frequency and less intensity than near-field earthquake motion, wind-induced structural responses of the proposed CSS with DTD in Fig. 1 will remain in elastic range and be contributed primarily by the first three modes. Therefore, the building with the nonlinear link beams will have almost the same wind-resistant performance (e.g., in terms of peak responses) as the building with elastic link beams. In other words, the proposed implementation will not significantly affect (neither decrease nor increase) the wind-resistant capability of the original structural system.

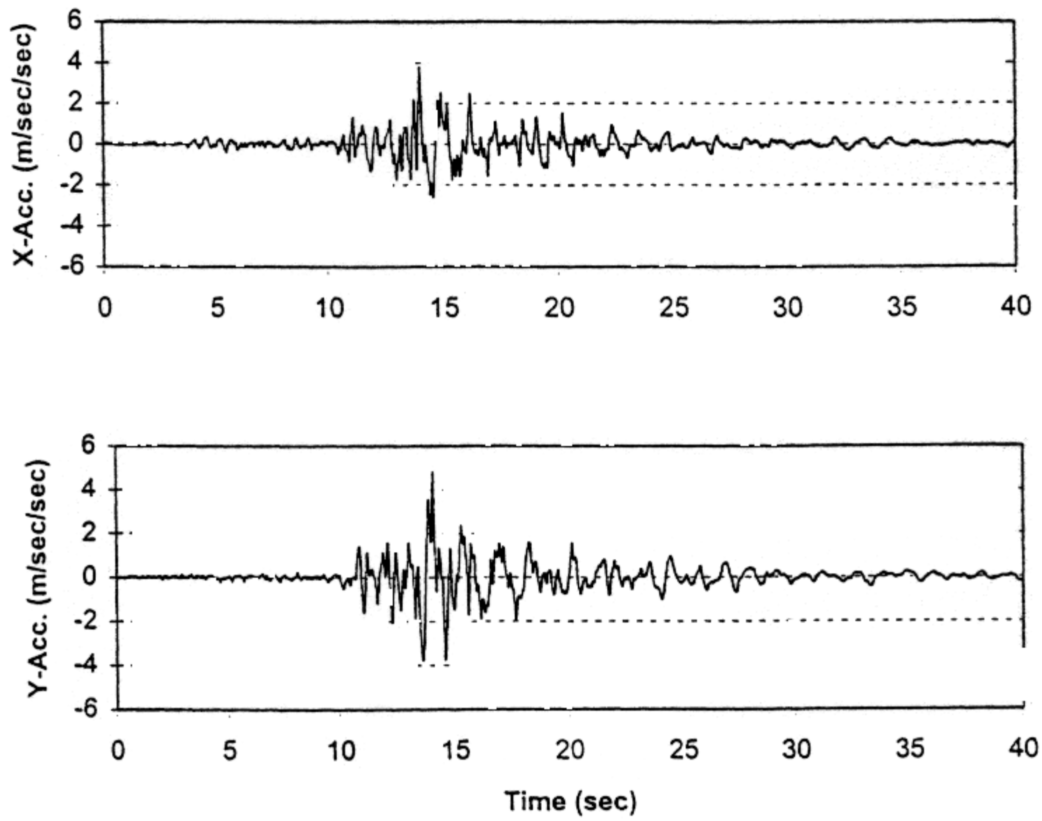

Fig. 2 Re-scaled Loma Prieta Earthquake Accelerations

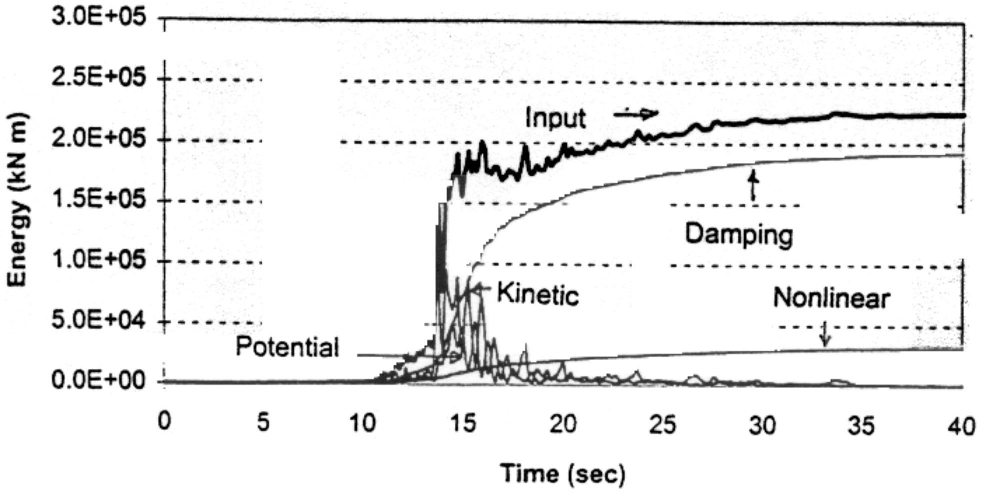

Fig. 3 Energy distribution and variations of building with use of nonlinear link beams

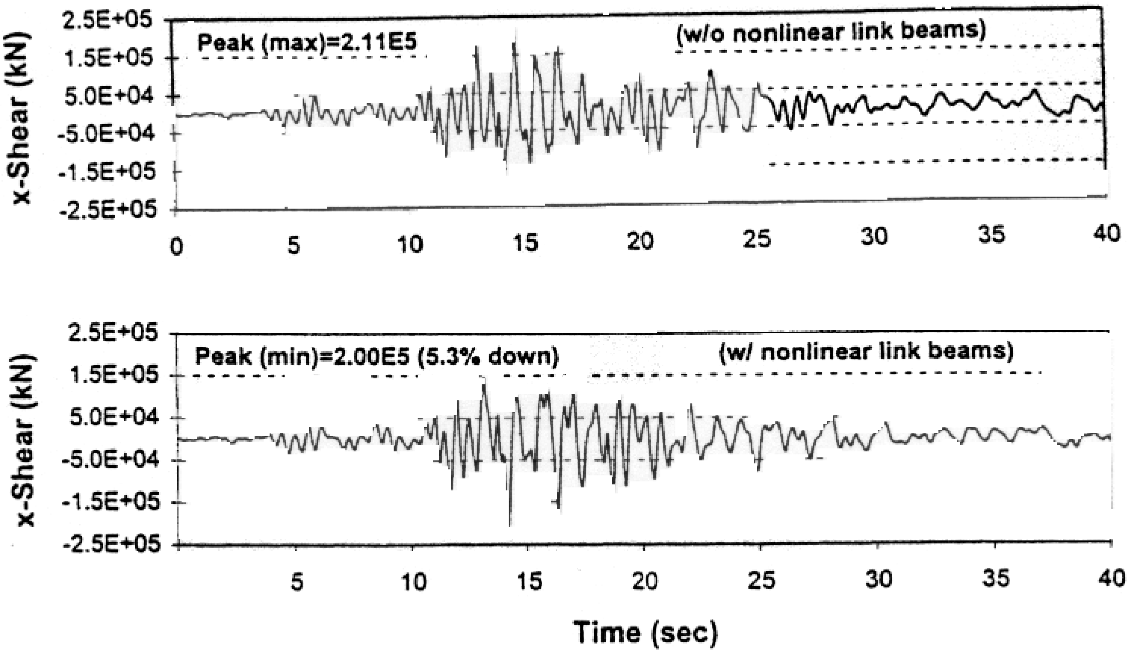

Fig. 4 Time histories of base shear in x-direction

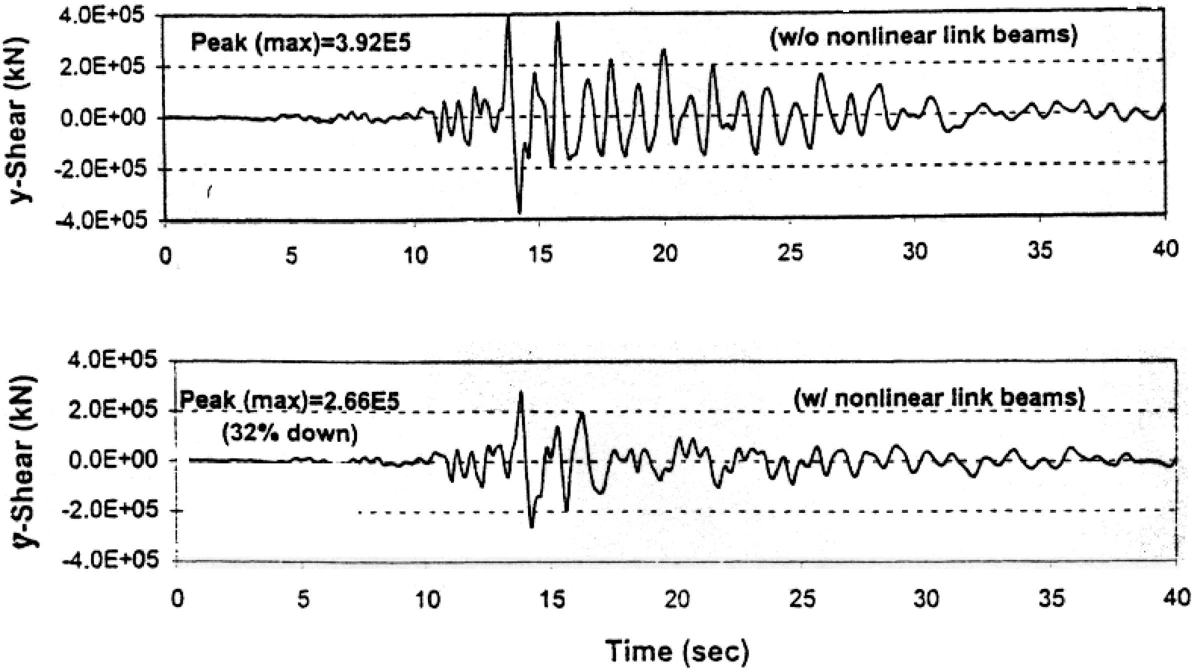

Fig. 5 Time histories of base shear in y-direction

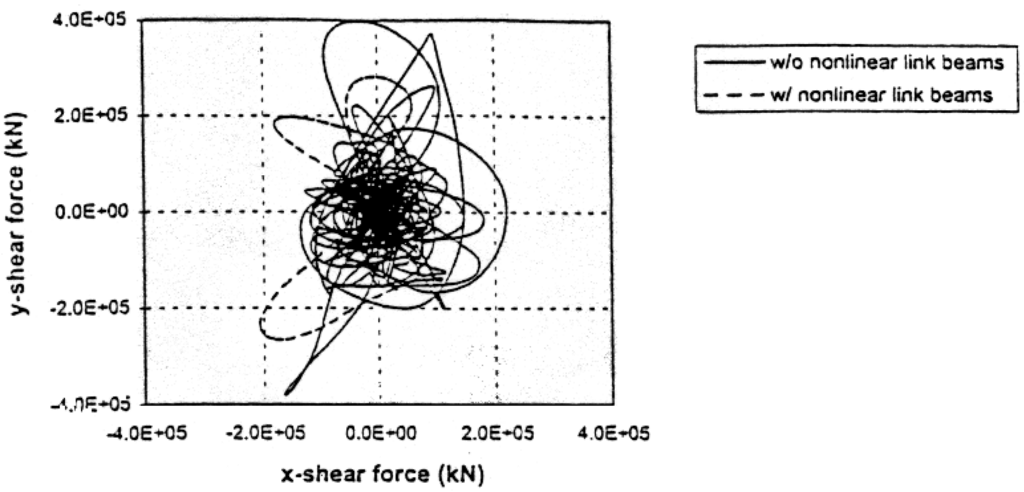

Fig. 6 Trajectories of base shear of building with and without use of nonlinear link beams

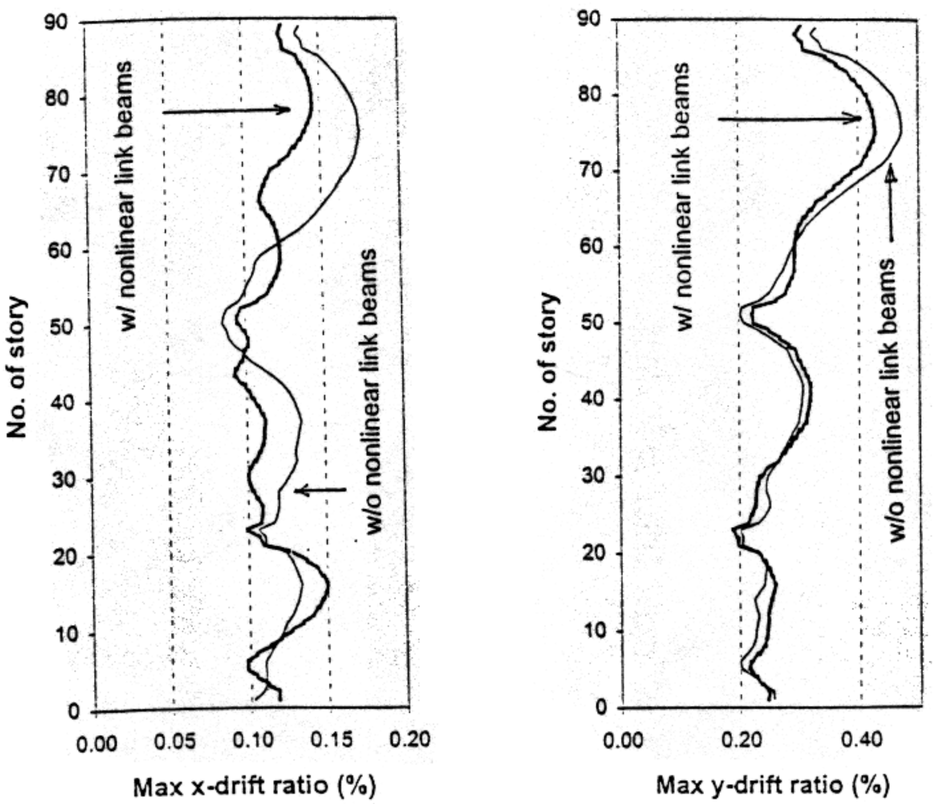

Fig. 7 Maximum response drift ratios at each floor

CONCLUSIONS AND REMARKS

This study proposes the implementation of damage-tolerant design into one of the efficient composite structural systems for a high-rise building, Shanghai Jinmao-type building. As an attempt, some link beams are selected and designed to be relatively soft in stiffness on purpose so that they could take seismic damage to protect the whole structure under severe earthquakes. Numerical example demonstrates that such an implementation could efficiently reduce various dynamic responses, consequently enhancing the structural safety under severe earthquake. In addition, the implementation is not only cost-effective (no extra devices and maintenance are needed in comparison with other solutions such as implementation of various control devices), but also easy to do in practice.

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