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Seismic assessment and retrofitting of existing structure based on nonlinear static analysis

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Abstract. Seismic assessment and retrofitting of existing structure is a complicated work that typically requires more sophisticated analyses than performing a new design. Before the implementation of a Code for seismic design of buildings (GBJ 11-89), not enough attention has been paid on seismic performance of structures and a great part of the existing reinforced concrete structures built in China have been poorly designed according to the new version of the same code (GB 50011-2010). This paper presents a case study of seismic assessment of a non-seismically designed reinforced concrete building in China. The structural responses are evaluated using the nonlinear static procedure (the so-called pushover analysis), which requires its introduction within a process that allows the estimation of the demand, against which the capacity is then compared with. The capacity of all structural members can be determined following the design code. Based on the structural performance, suitable retrofitting strategies are selected and implemented to the existing system. The retrofitted structure is analyzed again to check the effectiveness of the rehabilitation. Different types of retrofitting strategy are discussed and classified according to their complexity and benefits. Finally, a proper intervention methodology is utilized to upgrade this typical low-rise non-ductile building.

Keywords: seismic assessment; nonlinear static procedure; retrofitting; capacity; demand

1. Introduction

The first unofficial seismic design code in China was established in 1964 based on the seismic code of Soviet Union. After several revisions, in 1974 the first official Code for seismic design of buildings (TJ 11-74) was issued in China. However, structures designed to earlier codes did not behave well during earthquakes such as 1975 Haicheng and 1976 Tangshan due to insufficient lateral load carrying capacity and limited ductility. Tangshan city was almost totally destroyed by the largest earthquake of the 20th century in terms of number of casualties (Butler *et al.* 1979). In 1978, TJ 11-78 code made improvement on the basis of TJ 11-74 code and serviced afterward. The implementation of Code for seismic design of buildings (GBJ 11-89) in 1989 was a sign that structural performance during earthquake excitation became more significant. The concept of seismic design based on limit displacements has been emerged over the past 20 years. It is

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generally recognized that structural damage can be directly related to strain (and hence by integration to displacement), and non-structural damage, in buildings at least, can be related to drift (Priestley *et al.* 2005). Correspondingly, the following design criteria were first appeared in design code GBJ 11-89 (1989): (a) when suffering from normal seismic damage with lower degree than the local seismic fortification intensity, the building generally should not be damaged or could be used without repairing; (b) when suffering equal degree with the local seismic fortification intensity, the building should probably be damaged but could still put into use with normal repair or without any repair at all; (c) when suffering higher degree than the local seismic fortification intensity, the building should not collapse or have so serious damage that causes death.

The existing buildings that were gravity-load designed or designed to earlier codes without giving due proper consideration to the earthquake forces possess much more potential threats. Modern structures may also behave imperfect according to the latest revised version of the seismic design code GB 50011-2010 (2010). Besides, the normal design often neglects the complex soil-structure interaction (SSI) effect conservatively. Traditionally, it is believed that such negligence is beneficial to the structure. However, the detrimental effect of SSI has been observed in certain classes of bridges when considering the non-linearity induced in sub-structure components (Ciampoli and Pinto 1995, Mylonakis and Gazetas 2000). Therefore, proper and efficient seismic assessment procedure of existing structures especially in seismic vulnerable areas is necessary, and as a result, if needed, strengthening operations should be performed. There are two categories of seismic assessment methods: linear and nonlinear procedures. Equivalent linearized method, being recognized as force-based design method, is usually used to perform a new design. In most of seismic countries, this simple method is recommended and adopted in practice for decades even if it is inappropriate according to performance-based design theory (Priestley *et al.* 2007). While, nonlinear procedure is more attractive to analyze the existing structures. Both nonlinear dynamic time history analysis and incremental dynamic analysis (Vamvatsikos and Cornell 2002) can provide the seismic demand and capacity at the same time, but require more computational time in analysis. Encouraging results of direct displacement-based assessment procedure have been obtained for Single Degree-of-Freedom (SDOF) systems, frame and structural wall buildings, and multi-span bridges (Ni 2013a, Ni *et al.* 2014, Paolucci *et al.* 2013, Priestley 1997, Şadan *et al.* 2013). The complexity of these approaches prevents the application of use in practice. Nonlinear static analysis is expected to provide information on many response characteristics that cannot be obtained from a linear elastic static analysis: (a) realistic force demands on potentially brittle elements; (b) estimates of the deformation demands for elements that have to deform inelastically in order to dissipate energy; (c) consequences of the strength deterioration of individual elements on the behavior of the structural system (Krawinkler and Seneviratna 1998).

In this paper, nonlinear static procedure (the so-called pushover analysis) is adopted to evaluate the seismic risk of a non-seismically designed reinforced concrete structure. The need for retrofitting will be determined by comparing seismic capacity and demand. For a certain structure, the demand is defined by the ground excitation and the capacity can be evaluated according to GB 50011-2010. If the demand is less than the capacity, the structure can be regarded as safe; otherwise, failures may occur and strengthening is required. For this typical low-rise non-ductile building, rehabilitation procedure should be performed to achieve a suitable performance limit state, that a defined level of damage either in ductile or brittle mode (Lima *et al.* 2012a, b) unreached during the designed level earthquake and structure uncollapsed under maximum credible level earthquake (ATC-40 1996, FEMA356 2000, Ghobarah and Abou-Elfath 2001, Maheri and Sahebi 1997). Different kinds of upgrade technique implement into the existing

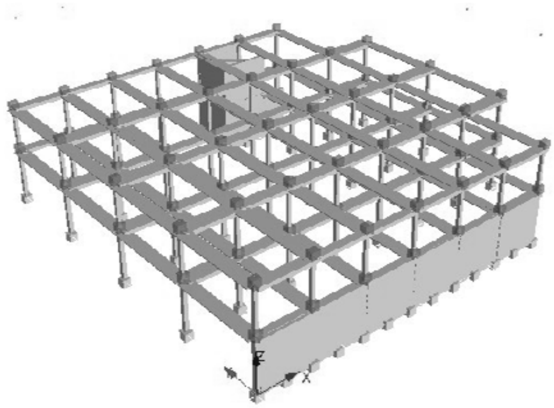

Fig. 1 3-D view of the building

structure. In principle, seismic verification of retrofitted structure should be carried out by applying the same method employed for the "as-built" frame. A major challenge for structural engineers is to ensure that the modified seismic action is less than the modified capacity. All criteria, such as engineering consideration, economics, disturbance to occupants, downtime and esthetics, should be taken into account. The relevance of criteria strictly depends on the specific application and, moreover, they often represent trade-offs. Final remedial plan is proposed based on the comparison of all considered criteria.

2. Building description

The existing structure under investigation is a typical reinforced concrete two-story building built in 1998 and located in Ningcheng, Chifeng, Inner Mongolia Autonomous Region, China. It was designed according to the design code GBJ 11-89. The local seismic fortification intensity is 8 degree, design basic acceleration of ground motion is 0.20 g, and design earthquake grouping is Group I. The building is founded on site type II and design characteristic period of ground motion is 0.35 s. Fig. 1 displays the overview of the building under consideration. The structure consists of 5 frames, 7 bays with the footprint of 30.6 × 30.65 m. The height of first floor is 3.85 m from ground level, which is reduced to 3.35 m for second floor. This structure has no underground level. In total, 76 columns have been planted to support the dead and live loads of the building and to resist the seismic actions. The cross sections of the columns are square and their dimensions vary depending on the floor they are located: 0.3 × 0.3 m and 0.25 × 0.25 m for first and second storey respectively. There are 60 beams installed along the *x* direction (Fig. 1). Rectangular cross section with various dimensions and reinforcement patterns is used for different bays and different frames. Frames are attached to each other continuously by slab. In analysis, the slab is represented by T shaped beam situated along *y* direction (Fig. 1). The total height of the representative T beams is 0.28 m, which is smaller than the threshold height of 0.4 m defined in GB 50011-2010 for rigid diaphragm. To resist seismic action, 6 interconnected shear walls are constructed laterally along *x* direction in 1st floor. At the opposite end of the building, U-shaped shear wall is introduced around the staircase to cope with the seismic action in *x* direction and partially in *y* direction. The second floor is more vulnerable to the ground excitation due to insufficient length of the U-shaped shear wall.

At first glance, it is anticipated that the structure is under-designed and may not be capable of resisting a probable moderate earthquake. The reinforcement ratio of columns can be calculated by dividing the embedded steel area to inclosing concrete area. First and second floor columns reinforcement ratios are 0.89% and 0.72% respectively. Both ratios are smaller than the prescribed reinforcement ratio by GB 50011-2010, which should be in the range of 0.9% to 5%. The hoops and cross ties are on the limit both in diameter and distant-wise for columns. Predefined maximum distance of 0.15 m for hoops is violated at the first floor. One other shortcoming related to columns is that their confinement ratio is very small (less than 1.2), thus their confinement is insufficient for strength enhancement while dealing with shear and chord rotation demands. The reinforcement pattern of beams is compact, which provides them capacities beyond the demand of parameters of interest (shear, chord rotation, and moment). This leads the inspector to the conclusion that beams may be over-designed. The flange width and slab thickness of representative T beams are below the threshold limit prescribed by GB 50011-2010. Therefore, T beams are not capable of behaving as a rigid diaphragm, which induces uneven force distribution among columns. It is detrimental to the structure and could easily end up in column failures and partial collapse. The span lengths are wide (6.8 and 7.0 m) for insufficient representative T beams and columns. Another major drawback of the existing design lies at the location of shear walls. It is far from being a symmetric-like configuration that an abrupt reduction in the number of shear walls is witnessed between floors. The deficiency of a sudden decrease in stiffness in vertical plan might cause soft storey failure.

3. Numerical modeling and assumptions

A three-dimensional finite element model of the candidate building was established via SeismoStruct software. The version 5.2.0 should be emphasized for any type of discrepancy that may be faced during the comparison of data presented here with the data obtained from other versions.

Fibre modelling approach is employed to explicitly represent the section area as shown in Fig. 2, thus material inelasticity spreads along the member length to ensure the accurate estimation of structural damage distribution. All the structural elements are modeled as force based elements with 200 fibers each having 5 integration points (D’Aniello *et al.* 2010). Beam is modeled as a single element that the change of reinforcement pattern along the beam length is not considered. It is very likely that this assumption may lead to imprecise assessment outcomes of the beams. However, it is made for the ease of calculation to avoid instability that can be confronted when modeling the beam with varying cross section along the length. Slab supported by T shaped beams cannot be modelled as a rigid diaphragm due to its small flange width and slab thickness. They are simply modeled as T beams via rigid connection with columns.

To represent steel, Menegotto-Pinto (1973) steel model with elasticity modulus of 200 GPa and yield strength of 500 MPa is used. The ultimate strain of the steel is defined as 0.06 both in materials and performance criteria. For concrete, Mander *et al.* (1988) model with a compressive strength of 42 MPa is assumed. Tensile strength of concrete is not introduced for numerical stability and for minimizing the computational efforts. A confinement factor of 1.2 is assigned for concrete core section different than the cover concrete to consider that confined concrete is more ductile (Guneyisi and Altay 2005).

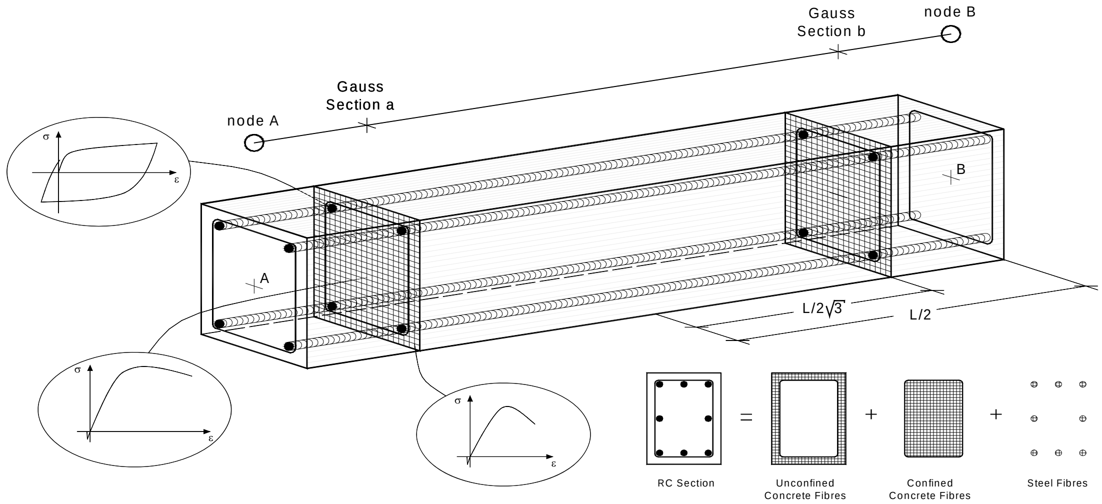

Fig. 2 Fibre element modelling for an inelastic beam-column element

Weight on structural members can be calculated by the available data of 6.5 kN/m^2 per storey and then be converted into mass. A lumped mass is assigned to the column instead of being uniformly distributed along the beam. This assumption may alter the moment experienced by the column during an earthquake, but the contribution of the additional moment is not dominant. Besides, design code GB 50011-2010 does not explicitly dictate the moment check for columns under seismic action. Concentrated mass assumption can also simplify the calculation. After the implementation of retrofitting, mass assigned at each column will be upgraded.

Soil-structure interaction may alter the seismic response of a SDOF system resting on a shallow foundation during moderate-to-strong earthquakes, with potential temporary mobilization of the bearing capacity of the foundation and development of permanent displacement and rotations (Ni 2012, 2013b). For simplicity, nonlinear soil response is neglected in this study. Fixed foundation is assumed that rigid links are appointed for structure-ground interaction. The nodes at z = 0 level are constrained in all 6 degrees of freedom (x, y, z, rx, ry, rz).

4. Nonlinear static analysis

Nonlinear analysis can determine the structural performance more accurately than a linearized approach when a prescribed seismic event strikes. All existing methods, such as nonlinear dynamic time history analysis, incremental dynamic analysis (Vamvatsikos and Cornell 2002) and direct displacement-based assessment procedure (Ni 2013a, Ni *et al.* 2014, Paolucci *et al.* 2013, Priestley 1997, Şadan *et al.* 2013), are complex and require large computational efforts. This paper is focused on the selection of the retrofitting strategies; hence nonlinear static analysis is a good approximation and more practical to indicate the rehabilitation in a short time.

The first proposed well-known Capacity Spectrum Method (CSM), as representative of nonlinear static procedure, is due to Freeman in 1975. However, the gradual recognization of the importance of displacement rather than strength of the members influencing the structural and non-structural damage became a turning point for nonlinear static procedures. In 1990s, the development of performance based seismic design (FEMA356 2000) and consequently displacement based design as ones developed by Panagiotakos and Fardis (2001) or by Priestley *et al.* (2000, 2007) lead to more and more interests regarding the improvement of CSM method. Capacity curve can be achieved by performing pushover analysis, that an incremental distributed loading is applied to the structure. One assumes that the structural response under pushover may replace the results from dynamic analysis. The performance point of structure is determined in agreement with the maximum displacement demand requested to the structure by the damped elastic spectrum. Damping is the summation of the inherent damping and substantial equivalent hysteretic damping due to the response of structure.

In this investigation, response controlled loading is used to capture the softening path of the capacity curve and soft-storey behaviour of the structure. Normalized eigenmode shape load pattern is selected since the weakness of the structure can be revealed better. A significant mode is designated to have more than 5% mass participation. The Modal Pushover Analysis (MPA) is performed in positive/negative x and y directions for each significant mode. The capacity curve is acquired and transformed into equivalent curve of a SDOF system, in the format of acceleration vs. displacement as follows

$$\Gamma_i = \frac{\sum m_i \Phi_i}{\sum m_i \Phi_i^2} \tag{1}$$

$$M_i^* = \sum m_i \Phi_i \tag{2}$$

$$d_{eq}^i = \frac{d^i}{\Gamma_i^*} \tag{3}$$

$$a_{eq}^i = \frac{F^i}{M_i^*} \tag{4}$$

where, i is mode number, m_i is the mass of i^{th} floor of the structure, Φ_i is i^{th} mode shape, Γ_i is modal participation coefficient, M_i^* is modal participation mass, d^i is top displacement, F^i is base shear, $d_{eq}{}^i$ is equivalent SDOF system displacement and $a_{eq}{}^i$ is equivalent SDOF system acceleration.

Displacement demand for each of the equivalent SDOF system is defined by using the CSM approach (ATC-40 1996):

(1) Inherent damping (e.g., 5%) is assumed to start the iteration.

(2) Damped response spectrum is generated according to Chinese Seismic Design Code (GB 50011-2010) and the site condition of the structure.

(3) Capacity curve of equivalent SDOF system and damped response spectrum are represented in one displacement-acceleration plot.

(4) Displacement demand is identified as the intersection between the two curves, which is so-called performance point.

(5) Equivalent bilinear curve crossing performance point is defined by imposing the same energy dissipation capacity (i.e., area) under both the capacity and bilinear curves.

(6) Equivalent viscous damping associated to hysteretic energy dissipation is then calculated.

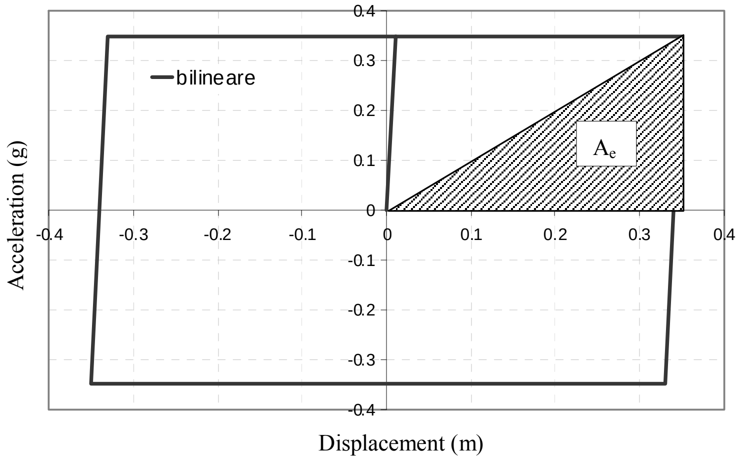

Fig. 3 Calculation of equivalent damping factor $\xi_{inherent}$

$$\xi = \xi_{inherent} + \xi_{hysteretic} = \xi_{inherent} + \frac{A_{loop}}{4\pi A_e} \tag{5}$$

where, $\xi_{inherent}$ is the inherent damping assumed in step 1, A_{loop} is the dissipated energy in a hysteresis cycle and A_e is the elastic energy stored by the system, referring to Fig. 3.

(7) Iteration from step 1 is conducted using the new damping value, until there is a convergence between assumed and computed damping values.

The converged performance point corresponds to the displacement demand of the Multiple Degree-of-Freedom (MDOF) system. It is now possible to extract, for each mode, the corresponding response quantities of interest (e.g., moments, rotations, shears, etc). Overall response of structure is computed by means of quadratic combination of the modal actions.

$$r = \sqrt{\sum r_i^2} \tag{6}$$

where, r_i is the response quantity of each mode and r is the overall response of structure.

The capacity of each structural member is estimated according to GB 50011-2010. If the calculated capacity is insufficient to handle the imposed loads, retrofitting is needed. In general, physical state of members has to be determined considering the degradation due to carbonation and steel corrosion. For this case study, the integrity of structure is assumed. Beams, columns and shear walls are checked for the limit state of collapse in terms of ultimate chord rotation and shear capacity.

5. Analysis result and retrofitting strategy

Eigenvalue analysis is carried out first to have a general ideal about the mode shapes, the frequencies and modal mass participations. This analysis provided key information about the behavior of the structure. Table 1 displays the modal mass quantities and percentage contributions of individual modes in translational x, y and z directions. Based on the effective modal mass participations in each direction, significant modes with larger than 5% participated mass that have to go through the Modal Pushover Analysis are determined. Along positive *x*-direction, 1st, 2nd, 3rd and 5th modes are depicted to resemble the structure's behavior and 2nd, 3rd, 6th and 1st modes are classified as influential for negative *y* direction. It can be anticipated that both positive and negative loading patterns along one direction have the similar results. In order to simplify the cumbersome calculation procedures, negative *x*-direction and positive *y*-direction are not considered here.

Pushover analysis for each mode in each direction is performed separately using CSM method. The left side of Fig. 4 presents incremental loads applied on the structure in positive *x* direction of 1st mode. Deformed shape of the structure corresponding to this load pattern is displayed in the right side of Fig. 4. Fig. 5 illustrates the procedure followed to attain performance point for mode 3 in positive *x* direction (left side) and mode 1 in negative *y* direction (right side) respectively.

When a convergence between assumed and computed damping is achieved, the displacement is recorded and the corresponding critical number of loading step is assigned. Attained load factor is utilized to extract base shear, moment, chord rotation and axial forces from SeismoStruct as the estimated seismic demand. The extracted results can only represent the response of structure in one direction for one mode; then quadratic combination rule should be used to obtain the overall

Table 1 Effective modal mass percentages

Mode	Period (s)	[Individual Mode]			[Cumulative Mass]		
		[*Ux*]	[*Uy*]	[*Uz*]	[*Ux*]	[*Uy*]	[*Uz*]
1	0.622	**39.03%**	**7.94%**	0.00%	39.03%	7.94%	0.00%
2	0.535	**20.44%**	**40.01%**	0.00%	59.48%	47.95%	0.00%
3	0.426	**6.06%**	**24.91%**	0.00%	65.53%	72.86%	0.00%
4	0.328	2.20%	1.04%	0.00%	67.73%	73.90%	0.00%
5	0.284	**5.88%**	0.01%	0.00%	73.61%	73.91%	0.00%
6	0.261	0.02%	**8.64%**	0.00%	73.62%	82.55%	0.00%
7	0.241	1.62%	0.28%	0.00%	75.24%	82.83%	0.00%
8	0.219	0.38%	3.33%	0.00%	75.62%	86.16%	0.00%
9	0.198	0.03%	0.34%	0.00%	75.65%	86.50%	0.00%
10	0.216	0.46%	0.36%	0.00%	76.11%	86.86%	0.00%

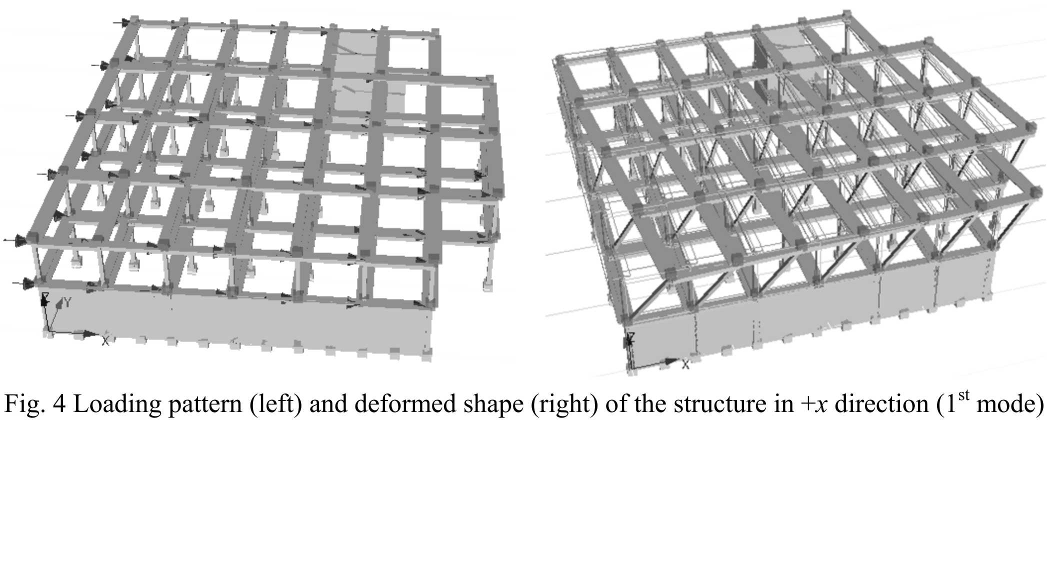

Fig. 4 Loading pattern (left) and deformed shape (right) of the structure in +*x* direction (1st mode)

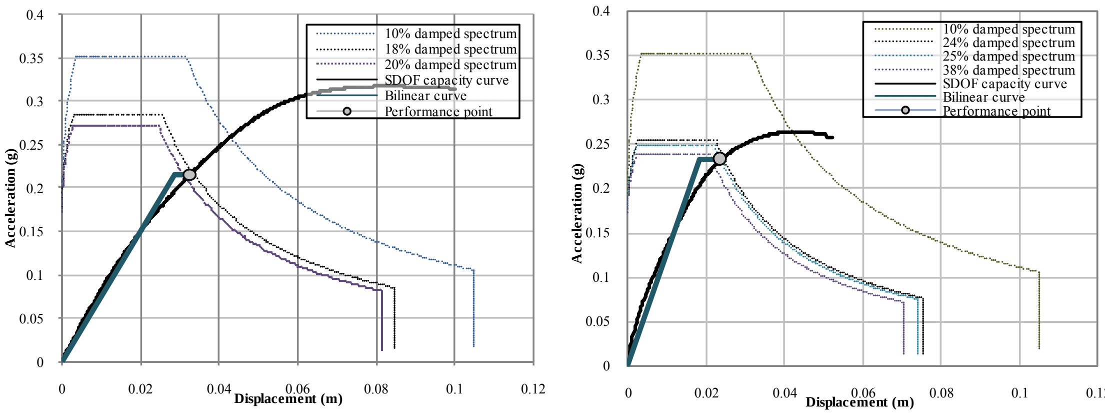

Fig. 5 Capacity Spectrum Method-mode 3 in x direction (left) and mode 1 in -y direction (right)

Table 2 Failure pattern of structure

Direction	Storey	Column	T-beam	Shear wall
x positive	1st	Middle frame columns have failed under shear.	T-beam failed along two side frames.	No failure.
	2nd	Nearly all columns failed except side frame along staircase.	All T-beam failed under shear.	No failure.
y negative	1st	Triangular shape of failed columns excluded from the secure zone of staircase core.	All T-beam failed.	All shear walls failed under rotation except the staircase core.
	2nd	Similar triangular distribution as 1st storey with smaller covering area.	All T-beam failed.	No failure.

seismic performance of structure in this direction. Except the potential risk checking of each direction (x and y), the load combination of $0.3x+1.0y$ and $1.0x+0.3y$ should also be assessed.

The capacity of each structural member is calculated and compared with demand to picture the potential seismic risk. For the “as-built” frame, column, T-beam and shear wall failures are observed. The capacities of beams are far more than the demand of the system. Excess reinforcement in the beams and short span lengths help them to survive seismic loading. No failure of beams is observed at the end of assessment. Observed structure’s failure pattern is demonstrated in the Table 2.

It is convincing that the system is not capable of handling the prescribed seismic loads and urgent retrofitting is required for the structure to ensure the long lasting safe serviceability. The selection of the most suitable retrofit strategy for the current building is not straightforward. There is no particular solution that is clearly better than others according to the whole criteria considered (i.e., cost, implementation downtime, etc.). The tradeoff between intervention philosophies and the benefit is illustrated in Table 3 (Christopoulos and Filiatrault 2006, fib-24 2003, Pinho 2001).

Two steps retrofitting scenarios are proposed based on engineering practices (Akis 2004, Bayramoglu *et al.* 2014, Bergami and Nuti 2013, Elenas and Vasiliadis 2002, Elenas *et al.* 2002, Griffith 2008, Kabir and Ghaednia 2008, Yılmaz *et al.* 2010). As displayed in the ellipse at Fig. 6, structural walls are implemented to the structure as the first step. The introduction of additional shear walls to the system is acquired by very comparatively small modifications relative to other

Table 3 Seismic retrofitting strategies

Retrofitting strategy		Ease of Implementation	Occupants Disturbance	Downtime	Foundation Intervention	Cost	Aesthetics
Global intervention	Structural walls	Easy	High	Medium	Yes	Medium	Little
	Steel bracing (added damper)	Difficult (Needs skilled labor)	Low to medium	Medium	Yes	Medium to High	Extra braces looks bad
	Base isolation	Difficult (Needs skilled labor)	Very high	Long	Maximum	Very High	Will not be affected
Member intervention	Injection of epoxy resin	Easy	Low	Less	No	Low to Medium	Will not be affected
	RC and steel jacketing	Easy	Medium to High	Less	No	Low, per member	Will not be affected
	Steel plate adhesion	Easy	Medium to High	Less	No	Medium	Will not be affected
	Shotcreting	Easy	Low	Less	No	Low	Will not be affected
	FRP composites jacketing	Easy	Low	Less	No	High	Little
	External reinforcement steel	Easy	Low	Less	No	Medium	Will not be affected

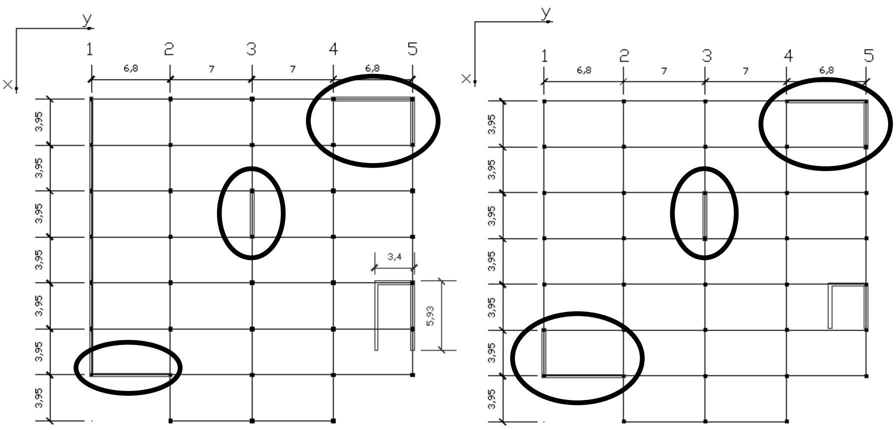

Fig. 6 Additional walls: 1st floor (left), 2nd floor (right)

possible scenarios, which keeps the cost at affordable limits. Time required for the implementation is in reasonable range since structure only goes through local retrofitting works. The initial configuration of structure leads to torsion since the center of mass does not coincide with the center of stiffness. To remedy the shortcoming mentioned, 9 shear walls are introduced to have a configuration with less eccentricity.

Only a local intervention in first step may not prevent collapse or partial failure; hence a second step remedial work should be done to the slab system. Slab plays a vital role to the overall seismic behaviour of the structure. The flexible slab is neither efficient for allocating the stiffness added to system via shear walls nor capable of redistributing the excessive loads applied to the system when a few number of columns and walls fail. Retrofitting of representative T beams is required to provide sufficient in-plane stiffness for slab. When the rigidity of the slab is increased, they will act as a horizontal rigid diaphragm to collect and transfer the inertia forces to the supporting vertical structural systems. It will ensure that vertical structural systems act together in resisting the horizontal seismic action. The proposal is to expose the reinforcement of the existing slab and beams by scratching the plaster and cover concrete at each floor level. A similar procedure is implemented in the top of columns for a height of 100 mm from the floor. Additional reinforcement is installed to connect the exposed vertical and horizontal load carrying members' skeleton. Concrete jetting is used to cover the new added unprotected reinforcement. The new RC slab will have a thickness approximately 400 mm. In-plant stiffness of the integrated and continuous slab will be sufficient to be modeled as a rigid diaphragm.

Two steps rehabilitation techniques are applied to the numerical model in SeismoStruct sequentially, and nonlinear static analysis is performed again to check the effectiveness of these strategies. Just as anticipation, without retrofitting slab system, failures of T beams are observed after first step rehabilitation. The outcomes are convincing for most of the columns and shear walls. After second stage retrofitting by strengthening the present flexible slab, a rigid diaphragm modeling is sufficient enough to provide the capacity for vertical and horizontal load carrying members of the structure. Augmented stiffness by shear walls is redistributed in the system through rigid diaphragm. Capacity of the structure in overall is upgraded and exceeds the seismic demand. Therefore, satisfactory seismic resistance of the structure is obtained by abovementioned retrofitting procedures.

6. Conclusions

The development of seismic design code in China has been reviewed. Displacement based or performance based seismic design theory appears in the Code for seismic design of buildings (GBJ 11-89) in 1989. This paper deals with the seismic assessment of a typical nonductile reinforced concrete two-storey building, which is built in 1998 by GBJ 11-89. Finite element method is used to analyze the potential risk of structure subjected to earthquake motions. Nonlinear static procedure is performed due to the consideration of relative accuracy and limited time. It is proven that the existing building possesses great risk according to the new seismic design code GB 50011-2010. Therefore, retrofitting strategy is necessitated to be implemented to the existing systems in a time and cost-effective way, without changing the character of the building too much. Different approaches are compared and the final solution is proposed by adding shear walls and strengthening slab system. Two steps of seismic assessment of structure after intervention are conducted, and satisfactory rehabilitation is attained.

Abovementioned retrofitting strategies reflect current practice of displacement based or performance based seismic design trends. Chinese Seismic Design Code (GB 50011-2010) requires that when suffering higher degree than the local seismic fortification intensity, the building should not collapse or have so serious damage that causes death. This means the global response and failure mode of structures under seismic action should be fully controlled. Additional shear walls increase the overall stiffness of the structure and their reasonable distribution diminishes the effect of eccentricity to the maximum extent. Although the increased stiffness leads the structure to attract more seismic forces, enhanced capacity of most members except T beams can handle the extra seismic demand. Less torsion effect can guarantee even force distribution in structural members, which is also a good sign of retrofitting methodology. In order to cope with T beams failure, strengthening procedures alternate the flexible slab system into rigid diaphragm, which transfers the seismic loads in more rational path. The significant improvement in the seismic performance of this building is observed after two steps retrofitting.

However, it must be emphasized that the adoption of retrofitting procedures is different case by case. Even if there are a large number of works regarding the regulation of retrofitting proceeding, special discussion and analysis of upgrading techniques should be done prior to their application.

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