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SEISMIC RETROFITTING OF A 56-STOREY RC TALL BUILDING BASED ON A NONLINEAR DYNAMIC PERFORMANCE EVALUATION

S. Epackachi[1], O. Esmaili[2], M. Samadzad[2], M. Ghalibafian[3], A.A. Taheri[4], S.R. Mirghaderi[5] and S.Vahdani[5]

ABSTRACT

Tehran tower is a 56 storey reinforced concrete building consisting of three wings with identical plan dimensions, each approximately 48 meters by 22 meters. The three wings are at 120 degree from each other and have no expansions/seismic joints. This paper considers the retrofitting of the Tehran tower based on the findings of an exhaustive investigation of the nonlinear performance evaluation efforts. It has attempted to show the procedure followed, methodologies utilized, and the results obtained for life-safety and collapse-prevention evaluation of the building. More over, the weak zones of the structure due to analyses results are introduced and appropriate retrofit techniques for satisfaction related life-safety and collapse-prevention criteria are also presented.

Introduction

Generally, a new 56 storey building located in a zone of high seismicity is designed to derive its earthquake resistance from ductile moment resisting frames or a combination of them and shear walls or braced frames.

A 56 storey building designed entirely to rely on shear walls in a seismic zone, such as the Tehran, is highly unusual and does not satisfy the current code requirements of Iran or the United States. The design of the building is also unusual because it uses heavy spine walls with a lot of concrete and reinforcements per square meter of construction. In contrast, the transverse walls rising 56 stories high are rather thin and at most are 25 to 30 centimeters thick.

Since the designed building did not comply with the Iranian Seismic Code provision, which has a limitation that only special moment frame or dual system are allowed for use in buildings with more than fifteen stories, some alternative evaluations were necessary to establish whether it satisfies the life safety and collapse prevention requirements implied by the seismic code provision. Accordingly, to evaluate the seismic performance of the building, FEMA-356, which is the performance-based design document, was selected as the principal guideline.

[1]Graduate Student of Structural Engineering, College of Civil Engineering, Tehran University
epackachi.siamak@gmail.com
[2] Graduate Student of Structural Engineering, College of Civil Engineering, Tehran University
[3]Associate Professor, College of Civil Engineering, Tehran University
[4]Structural Engineer, Professional Engineer
[5]Assistant Professor, College of Civil Engineering, Tehran University

Description of the building

Tehran Tower is a 56 storey reinforced concrete structure. It consists of three wings with identical plan dimensions each approximately 48 meters by 22 meters. The three wings referred to as wing A, B and C at 120 degree from each other and have no seismic joints. Fig. 1 shows a typical floor plan for the building. The wall layouts for wings A and B are exactly identical. Due to different apartment arrangement at wing C, however, the layout of walls in this wing is different in a minor way from those on wings A and B.

The structural system consists of solid floor and roof slabs spanning to transverse and longitudinal walls. These walls provide gravity as well as wind and seismic resistance for the building. Due to the unique geometry and design of the building, lateral translational resistance is provided by the longitudinal walls (spine walls) and torsion is resisted solely by the cross walls (transverse walls).

The walls do not have any seismic boundary elements along either exterior or interior openings. Reinforcement detailing is typically non-ductile except possibly for coupling beams of the transverse walls. These coupling beams, which are located in transverse walls near to spine walls to connect these perpendicular walls together, were originally detailed with typical non-ductile wall reinforced. However some later drawings indicate strengthening design of these beams with ductile ties and additional flexural reinforcement.

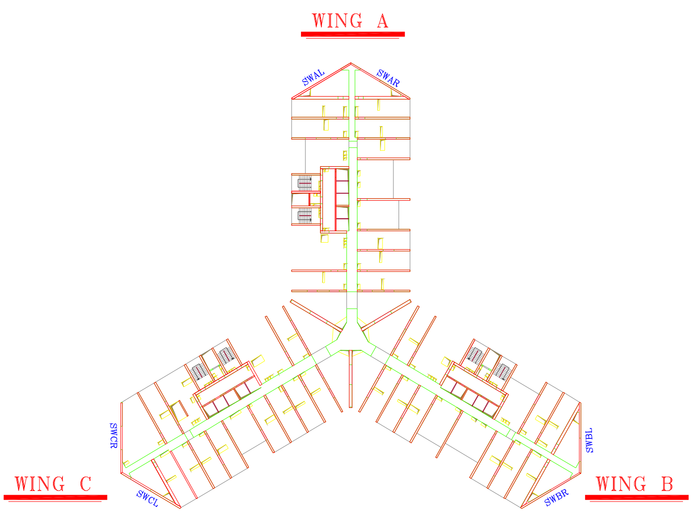


Figure 1. The view of the structural plan.

Figure 2. The view of the structural foundation.

Foundation consists of a reinforced concrete mat with variable and substantial thickness. As a result, our computer model assumes a fixed base for building shear walls (Fig. 2).

To our knowledge, this is the only building of this height in a zone of high seismicity which relies solely on shear walls with typical non-ductile wall reinforced as its lateral load resisting system. Most, if not all, building codes developed with seismic regions in mind prohibit use of shear wall systems as the only lateral force resisting system for building taller than 50 meters.

From the engineering point of view, it is understood that the design engineers relied solely on the longitudinal walls (spine walls) for lateral load resistance and did not count on transverse walls to participate in resisting lateral forces. In addition since the three spine walls concur at the center of the building, they cannot provide the building with any torsional resistance. Then the torsional resistance must come from the transverse walls.

Linear Evaluation

To investigate the linear behavior of the structure, it is generated two detailed linear models, one using ETABS and the other using SAP2000 computer program.

The ETABS model (Fig. 3) used a rigid diaphragm assumption while SAP model included the floor slabs in order to include the effect of slab deformations, compute stress concentration and ringing effect in floor diaphragm. Also FEMA-356 was used as the basis for the building evaluation. Moreover linear dynamic spectral analysis using site specific spectrum was employed considering all significant vibration modes of the building.

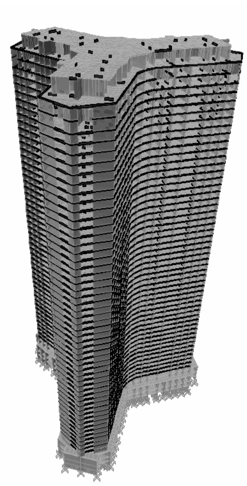


Figure 3. The 3D view of the model generated in ETABS.

The general assumptions and results of these analyses are as follows:

1- Diaphragm stresses are not significant and assumption of rigid diaphragm is generally valid. In fact there are some stress concentrations at the re-entrant edges of floor slabs. These stresses, however, are expected because of sharp edges at the points of wing junctures. However these stresses will be re-distributed over larger areas after initiation of some nominal and expected cracks in the slab concrete (T.R.B 2002).

2- The global behavior of spine walls is within the elastic range, even for the unreduced forces resulting from spectral analysis (T.R.B 2002).

3- The coupling beams of the transverse walls were found to be overstressed, in some cases by a substantial amount (T.R.B 2002).

4- The transverse walls were found to be overstressed under axial compression and tension. This, of course, is at least in part the result of high shear present in coupling beams, caused by the assumption of elastic performance of these elements (T.R.B 2002).

Nonlinear Evaluation

General properties

The PERFORM-3D computer model (Fig. 4) is composed of nonlinear walls and nonlinear beams. Whole walls were modeled as fiber elements with concrete and steel layers constituting different fibers. Also Flexural nonlinearity of walls is captured via the use of nonlinear fiber elements, modeling fibers of vertical concrete and steel layers. All spine walls were considered as cracked walls due to their performance against lateral loads and in contrast the transverse walls were considered uncracked elements based on their main role to carry gravity loads. Flexural nonlinearity of the lintels was found to be no controlling as all these beams invariably fail in shear.

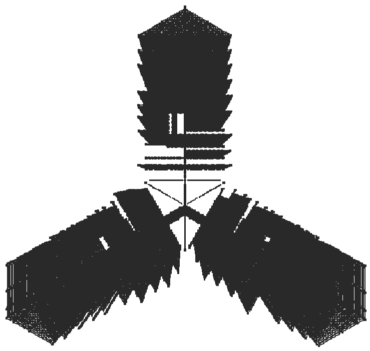


Figure 4. The 3D view of the model plan generated in PERFORM-3D.

Shear behavior of walls and beams was modeled as elastic-plastic shear hinges. The effect of stiffness degradation in reducing the energy dissipation capacity of members was also included in this model.

The PERFORM-3D computer model totally has 82180 degree of freedom and is consisted of 17120 structural joints, 13368 shear wall elements, and 3079 beam elements.

To consider nonlinear behavior of the material, PERFORM-3D model uses a trilinear backbone with an optional strength drop as the basic nonlinear material model for steel reinforcement in tension and for concrete in compression. This advantage is fully consistent with the nonlinear material models of FEMA-356.

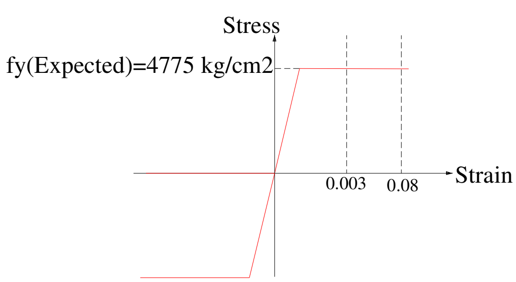


Figure 5. The steel stress –strain curve as an elastic-perfectly-plastic curve.

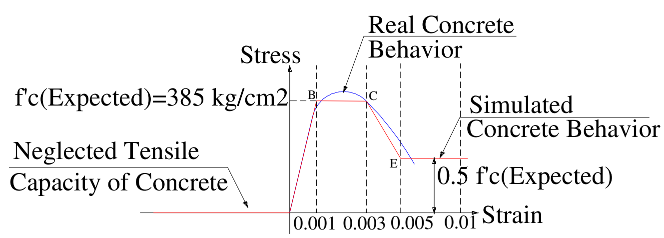


Figure 6. The concrete stress –strain curve as a tri-linear backbone curve.

To identify the performance of each component, five limit states have been used.

1- The onset of nonlinear behavior identified by steel reaching expected yield strength or concrete in compression reaching expected compressive strength or brittle shear cracking of concrete.

2- Life safety (LS) for primary members.

3- Life safety (LS) for secondary members.

4- Collapse Prevention (CP) for primary members.

5- Collapse Prevention (CP) for secondary members.

In full conformity with section 3.2.8 FEMA-356 requirements, the load combinations with seismic loads considered as follows:

$$Q_g = 1.1(Q_D+Q_L+Q_S) \quad (1)$$

$$Q_g = 0.9(Q_D) \quad (2)$$

Q_D: Dead-load (action)

Q_L: Effective live load (action), equal to 25% of the unreduced design live load, but not less than the actual live load

Q_S: Effective snow load (action)

For life safety evaluations it has been used the site specific design spectrum and ground motion time histories which had been selected from a data base of some 2500 recorded motions and represent the most suitable records considering earthquake magnitude, fault distance, fault type and site soil conditions. The seven pairs of earthquake time history records were selected to provide the best average approximating the site spectrum for earthquake probability of 10 percent in 50 years (Rahnama and Berberian 2002).

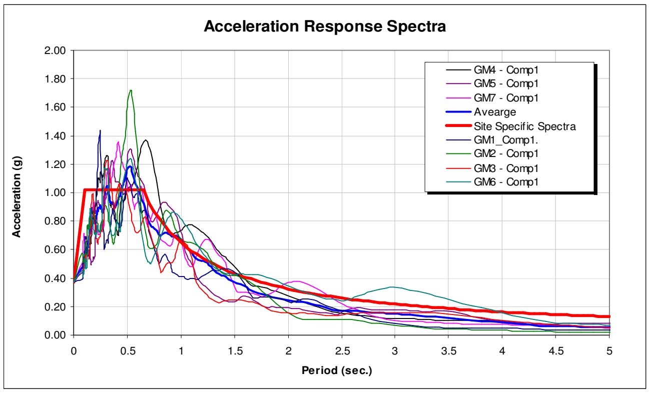


Figure 7. 5%-damped site specific spectrum, 5%-damped spectrum of ground motion records, average of spectrums of records.

Analysis Results

Evaluation of mode shapes

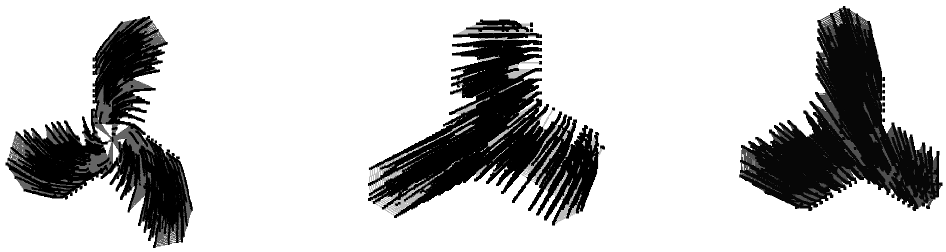


Figure 8. The First mode shape withT_1 = 3.761 s (left), the Second mode shape with T_2 = 1.313 s (center), the Third mode shape withT_3 = 1.294 s (right).

The fundamental period of other mode shapes are as follows:

T_4 = 0.798 s
T_5 = 0.368 s
T_6 = 0.365 s

T_7 = 0.345 s
T_8 = 0.208 s

Evaluation of time-dependent effects with consideration of construction sequence loading

To consider these effects (ACI-209 1997), a FORTRAN code was developed to analyze the spine wall and transverse wall separately under theirs self-weight considering creep and construction sequence loading effects.

In the next part, the geometric properties of model and the analysis results will be presented.

Geometric properties

Table 1. Properties of walls for creep effect analysis.

Wall Type	Length(m)	Thickness(m)	Height(m)
Spine Wall	50	1	170(56 stories)
Transverse Wall	12	0.3	170(56 stories)

The results are presented in Fig. 9, and significant differences are shown between transverse wall and spine wall displacements due to creep and construction effects.

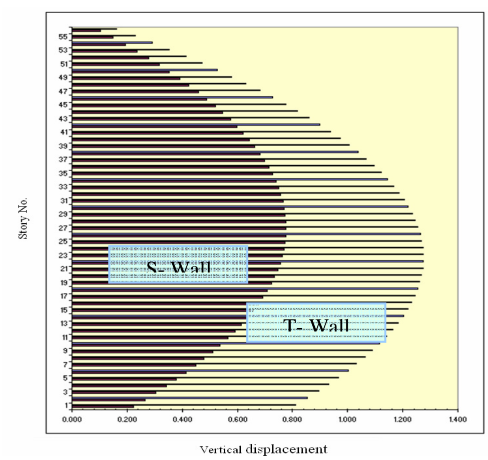


Figure 9. Results with consideration of construction sequence loading plus creep effects for S-Wall (Spine Wall) and T-Wall (Transverse Wall).

Conclusions of time analysis are followings:

1- Provided that the structure analyzed traditionally, not considering these facts, the critical demands due to cumulative differential displacements would be occurred in upper structural elements. If time dependency of concrete and construction sequence loading are coupled in analyses, the critical demands would be descended to middle height of the structure (here is somewhere between 25~35th storey).

2- Increased differential movement of coupling beams which connect spine wall to transverse walls due to time dependent effects, has caused these elements considered as cracked elements before occurring any earthquake.

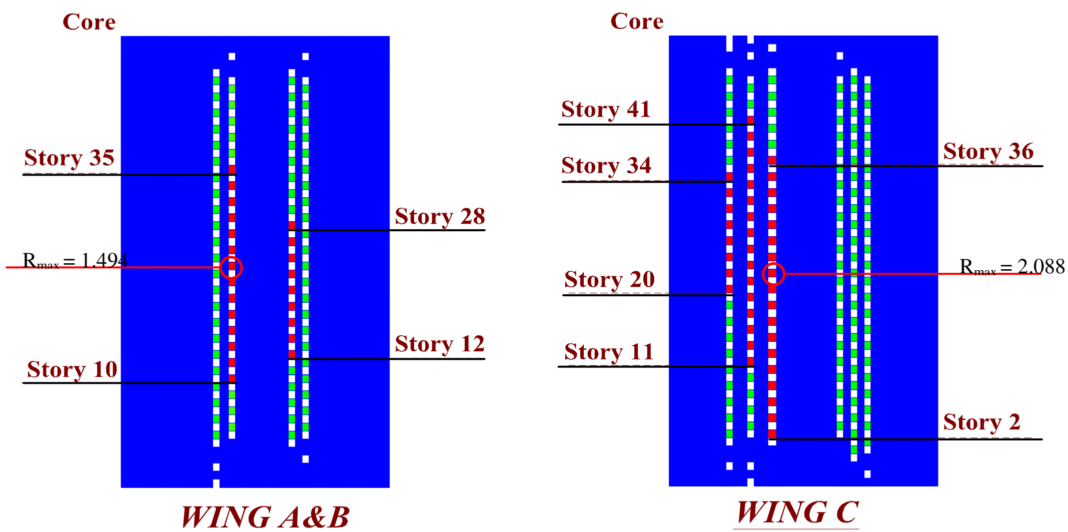
3- Redistribution of loads according to creep and sequential loading will intensely change the primitive assumptions on gravity load tributaries and consequently the level of ductility.

Evaluation of Structural Drifts

Considering the dominant shear behavior of spine walls and based on item C2.4.4.3.1 FEMA -356, which defines the shear behavior of wall as a displacement controlled behavior, drift evaluation is performed to control wall displacements on the basis of FEMA-356 requirements. Drift quantities in this building are smaller than maximum drifts, which are defined in the table 6-19 FEMA-356, in LS level (0.65%) and CP level (0.75%) due to the rigidity of the spine walls (T.R.B 2005).

Evaluation of coupling panel elements in spine walls

Coupling panels of the spine walls represent the most seismically vulnerable components of this building. In addition they provide primary resistance to translation and consequently their performance determines lateral performance of the entire building. All of these components have a short span and they are strong in bending, shear controlled, and non-ductile elements. Because of the fact that this is not true to attribute the drift story to locally relative displacements in coupling elements, their shear strain was considered as a main item to control coupling panel displacements on the basis of FEMA-356 requirements.



Coupling panel shear strain demand/capacity

Coupling panel shear strain demand/capacity

Figure 10. Inelastic shear strain demand-capacity ratios on coupling panels.

Inelastic shear strain demand-capacity ratios on coupling panels are obtained for both the LS level and CP level and in all coupling panels the CP ratios exceeded the LS ratios (T.R.B 2005).

Evaluation of transverse walls

Because of the large height-to-width ratio of the transverse walls, these components should be controlled by bending moment and on the basis of section C2.4.4.3.1 FEMA-356 the moment behavior of shear walls is the one of displacement-controlled behavior types. Then plastic hinge rotation of shear walls should be estimated in order to control the displacement of shear walls. Since all of the transverse walls demonstrated elastic behavior roughly above the fifth storey, there is not any potential to generate plastic hinges in upper parts of transverse walls and as a result it has been limited to estimate plastic hinge rotations only in the first storey of the building.

Acceptable plastic hinge rotations were calculated based on section 6.8.2.2 FEMA-356 for seven pairs of time histories. Considering the average plastic hinge demand-capacity ratios, it has been gained that all ratios were less than one and as a result there is not any critical condition at the base of transverse walls (T.R.B 2005).

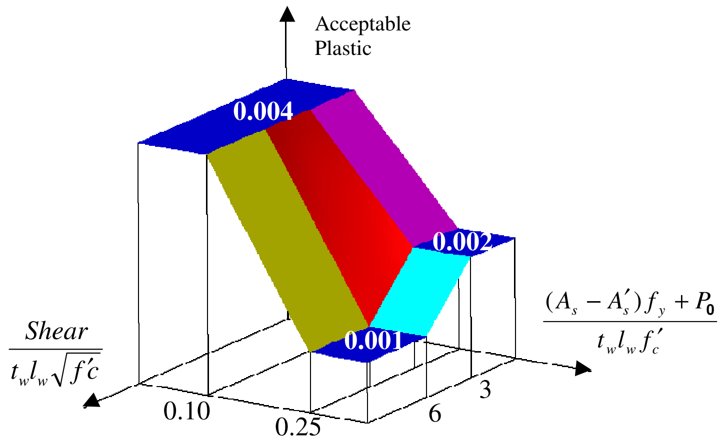


Figure 11. Allowable plastic hinge rotation according to section 6.82.2. FEMA-356.

By the fact that the torsional resistance must come from the transverse walls, these elements should have sufficient ductility and flexibility to participate in resisting torsional forces. Consequently it was necessary to control their forces demand-capacity ratios. According to section 6.8.1.1 FEMA-356 Shear walls or wall segments with axial loads greater than 0.35 P_0 shall not be considered effective in resisting seismic forces (FEMA-356 2000). Unfortunately there is not any clear statement in this part that what is the source of axial force which is not allowed to exceed from 0.35 P_0. By the way to consider any probable condition, three kind of axial forces were calculated for each transverse wall. The first kind was the maximum value of axial forces due to each pairs of time history analysis demand-capacity (0.35 P_0) ratio and the second kind was the average value of axial forces due to each pairs of time history analysis demand-capacity(0.35 P_0) ratio and the third kind was the gravity axial load demand- capacity (0.35 P_0) ratio. By examination all of these values, it is more advisable and reasonable to choose the third one. In fact a level of ductility for seismic bracing systems, conceptually, should be provided for energy absorption but axial loads have an adverse effect on their acceptable performance and this fact should be considered exactly by limiting the axial force due to gravity loads in order to hold enough ductility in seismic bracing system. According to this fact for all transverse walls it was been obtained that the third ratio is less than one and on the basis of this conclusion the transverse walls will not have any problem to behave as a seismic bracing system.

Moreover for an exact evaluation of transverse wall strength, it was necessary to plot all interaction diagrams representing true resisting behavior of walls. Next axial forces and bending moment of transverse walls due to seven pairs of time histories for LS and CP level were put in related interaction diagrams and by this way it has been calculated the maximum strength demand-capacity ratio for all transverse walls. Fortunately for most of runs, as expected, the maximum D/C ratios occurred at the lower levels of the strength of transverse walls and it means that there is not any critical condition on the strength of these walls (T.R.B 2005).

Evaluation of lintels (coupling beams)

Despite the fact that coupling beams are assumed to be cracked prematurely in earthquake, this event might take place under permanent gravity loads as a result of concrete time dependency. According to this fact, some coupling beams, connecting spine wall to transverse wall, were found to be cracked by visual observation and by considering this fact it can be concluded that coupling beams are plastified under their fixed end moments due to non-uniform vertical displacements. Level of axial stresses associated with floor loads on transverse walls and spine wall were the same and only probable cause might be time-dependent effects based on self weight of walls. Also all of the walls have at least 0.7% of reinforcement and accordingly the shrinkage effects will be negligible.

To consider the time dependent effects on coupling beams, two kinds of numerical models, *Fixed Model* and *Released Model*, were established. In other words, because of increased differential movements in coupling beam tips, their bending stiffness decreases severely and consequently it will form a kind of moderate flexibility in the coupling beam ends. To contemplate of this condition, in *Released Model* all coupling beams were modeled as simply-supported beams and in *Fixed Model* all coupling beams were modeled as fixed-end beams. Hence for all evaluation it was considered the critical value between two groups of results.

By the way according to FEMA-356 requirements for coupling beams, it has been calculated the chord rotation demand-capacity ratios for all coupling beams (*Fixed model*) without any consideration of time-dependent effects and their results was satisfactory, as the maximum chord rotation was within the safety level of FEMA-356 (T.R.B 2005).

Retrofit Options

During the construction of tower, nonlinear dynamic performance evaluation of tower specified the most critical weak zones of structure, coupling panels, and consequently coupling panels in the top 15 floors of the building were retrofitted by ductile detailing using sufficient number of X bars and seismic hooks (T.R.B 2005) and as a result the retrofitted coupling panels, having the same strength and thickness as before the retrofit, achieved more ductile performance. Then In all recent investigations the existence of X bars and special seismic details in the top 15 floors of the building were considered.

As illustrated and debated in previous parts, the most vulnerable zone of structure against earthquake loads, were coupling panels in spine walls. In fact the former retrofit project in the top of structure has caused to modify the element behavior at this zone but now the results of nonlinear study on this project depict some local weaknesses in coupling panels which are located at the mid height of the structure.

Through the various methods to retrofit coupling panels, it has been selected to cover both sides of these elements by steel plates. In fact there is no doubt that shear resistance of steel is much more than concrete and by chance this property is much useful to increase the ductility and deformation capacity of coupling panels.

Three criteria were considered to design steel plates as retrofit elements.

1- strength criterion
2- stiffness criterion
3- buckling criterion

By considering above criteria, the thickness of steel plates was calculated between 7 to 10 millimeters related to coupling panel position and finally it was decided to choose typical thickness, 10 millimeters, for all types of retrofit.

In the first try the weakened coupling panels in numerical model were modified by increasing the shear strength and stiffness. In fact, it has been calculated extra strength and stiffness which is achieved by a 10 millimeters of steel plate. After examining the nonlinear analysis results, it was observed that in retrofitted coupling panels all ratios have changed between 0.65 and 0.97 with an average of 0.81. Despite the modification of retrofitted coupling panels by this way, in spine wall A and B the extra shear demand had moved to side panels just close to retrofitted panels and as a result shear strain demand-capacity ratio had increased in these panels relatively. Moreover another line of coupling panels close to retrofitted coupling panels depicted some weaknesses which had not been observed in previous analyses. Finally it has decided to retrofit three lines of coupling panels in spine wall C and also in spine wall A and B instead of two lines and of course the retrofit extension in each line was related to the ratio of coupling panels located in each mentioned line.

Detailed Examination of the Connections between Concrete and Steel

Make using of steel elements which are connected to concrete member by anchorage is one of the current methods to modify the local behavior of concrete components. Local improvements that can be considered include improving component connectivity, component strength, and component deformation capacity.

Definitely it is a matter of fact that proper performance of anchorages can be considered as a certain guarantee in order to coordinated behavior of connected elements be true and exactly by this reason FEMA-356 has limited the spacing of anchorages to four times of their embedded depth. In fact this limitation seems not being applicable for current cases with limited geometrical dimensions. Knowing the mentioned facts to investigate the behavior of single and group anchorages which had been embedded in concrete under shear and tension loads, some experimental tests were done in HTL Rankweil which is located in Austria considering arrangement, spacing, and depth effects. The tests were performed for 1, 2x2, 2x3, and 3x3 anchorages with spacing 150 and 200 millimeters and also embedment depth 200 millimeters. Furthermore for each test were done 3 tests exactly like together to decrease the possible experimental faults. As a result 21 tests have been performed.

In this study, we have attempted to find the behavior of single and group anchorages under shear and tension loads considering their local failure modes in concrete and steel plates. Moreover, the horizontal and vertical displacements of each anchorage and steel plate have been demonstrated for whole tests to provide a real understanding of special manner in which anchorages cooperate to carry shear and tension loads. In addition, by comparing the arrangement, spacing, and depth effects to the shear and tension capacities of group anchorages have been indicated. Moreover, this study has attempted to verify the experimental measurements by analytical and numerical investigations (Epackachi and Esmaili 2006).

Conclusions

Coupling panel of the longitudinal walls and lintels of the transverse walls represent the most seismically vulnerable components of this building. Coupling panels provide primary resistance to translation and lintels provide primary resistance to torsion. Thus their performance determines the performance of the entire building. Both these components have a short span, are strong in bending and are shear controlled.

Based on linear evaluation of the tower, the global behavior of spine walls is within the elastic range while the coupling beams of the transverse walls were found to be overstressed, in some cases by a substantial amount.

Column shortening effect due to construction sequence analysis with time dependency of concrete may be usually neglected in practice while this event in members with unbalance axial stiffnesses and closely spaced such as columns and walls is significant, especially in tall buildings and may produce additional bending moments and shear forces to beam members. Increased differential movement of coupling beams of the Tehran tower has caused these elements considered as cracked elements before any earthquake occurrence. Also, provided that the structure analyzed traditionally, not considering these facts, the critical demands due to cumulative differential displacements would have occurred in the upper structural elements. If time dependency of concrete and construction sequence loading is coupled in the analyses, the critical demands would descend to the middle height of the structure.

Considering the average plastic hinge demand-capacity ratios for transverse walls, it has been shown that all ratios were less than one and as a result, there is not any critical condition at the base of these elements. Also, since the level of axial loads have an adverse effect on ductility for seismic bracing systems, for all transverse walls it was obtained that the gravity axial load demand-capacity (0.35 P0) ratio is less than one and on the basis of this conclusion the transverse walls will not have any problem to behave as a seismic bracing system.

The former retrofit project in the top of the structure (using X bars in coupling panels) has caused the element behavior to change at this zone but now the results of nonlinear study on this project depict some local weaknesses in coupling panels, which are located at the mid height of the structure. Hence, to improve the local behavior of these elements, steel plates which are connected to concrete members by chemical anchorages has been used as the best retrofitting method for this case.

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