Vulnerability evaluation and seismic rehabilitation of steel building with semi-rigid connections (Case study - tehran)

H. Shakib1; S.Dardaei2; M.Pirizadeh2; A.Moghaddasi3

1 Professor, Dept. of Civil Engineering, Tarbiat Modares University, Tehran, Iran
2 Ph.D. Student, Dept. of Civil Engineering, Tarbiat Modares University, Tehran, Iran
3 Retrofitting Division, Natural Disaster Institute, Tehran, Iran
Email: shakib@modares.ac.ir

ABSTRACT：

In this paper, the seismic rehabilitation of a steel building with semi-rigid connections in Tehran is presented. This building with 19 stories in three blocks and an asymmetric plan has been constructed 30 years ago. First step of the rehabilitation study includes qualitative vulnerability evaluation which indicates that the building has heavy seismic vulnerability potential. In the second step, quantitative vulnerability of the structure according to FEMA 356 guideline is evaluated. The results showed that the building is resistant enough against gravity loads but not safe enough for seismic loads. Finally, two seismic retrofitting plans consisting of concrete shear walls and steel shear walls are proposed. Comparison of retrofitting plans of the building from seismic performance, architectural and implementation aspects showed that the concrete shear walls plan is the superior choice.

KEYWORDS:

Semi-Rigid Connection, Vulnerability Evaluation, Rehabilitation, Concrete Shear Walls, Steel Shear Walls

1. INTRODUCTON

Steel building with Khorjini semi rigid connections has been commonly used in Iran since 50 years ago. For the construction of the steel building with this kind of connections, the continuous beams are crossed on the two sides of the column and connected to the column by top and bottom angles as shown in figure 2.
The behavior of steel buildings with this connection type in different earthquakes such as, Manjil, 1990 and Bam, 2003 showed that the connections fail before failure of the beams and columns. In 1991 the tests on physical model of Khorjini connection by Moghaddam and Karami [1] showed that this kind of connections is semi rigid without enough ductile behavior. In 1993 Tehranizadeh et al. [2] and also Barkhordari et al. [3] studied the rigidity degree of this connection. In 2000 Mirghaderi and Mazrouei[4] presented a method for rehabilitating Khorjini connections by adding top and bottom flange and triangle plates. This method usually increases the rigidity degree of these connections.

In this paper, the seismic rehabilitation of a steel building with semi-rigid Khorjini connections in Tehran is studied. By modeling the structural connection and using the nonlinear behavior, a curve representing the semi-rigid connection behavior is obtained. Then, quantitative vulnerability of the structure according to FEMA 356 guideline [5] is evaluated and two seismic retrofitting plans consisting of concrete shear walls and steel shear walls are proposed for rehabilitating the building. The renovated systems are analyzed with response spectrum procedure and the acceptance criteria of each elements is controlled according to FEMA 356 guideline [5] and the rehabilitation design for existing beams, columns, connections and foundation are presented.

2. THE EXISTING BUILDING

The existing building was designed and built between 1973 and 1978 in Tehran. According to figure 1, the building has 3 rather similar blocks. There is about 21 centimeters distance between the blocks. The building consists of 19 stories: 3 underground stories, the ground floor and 15 stories on the ground. The area of each story is approximately $1950m^2$. Floors -3 to -1 are used for car parking area and the upper floors have administrative usage and is office to 2000 people. Typical story height is 3.2 meters.
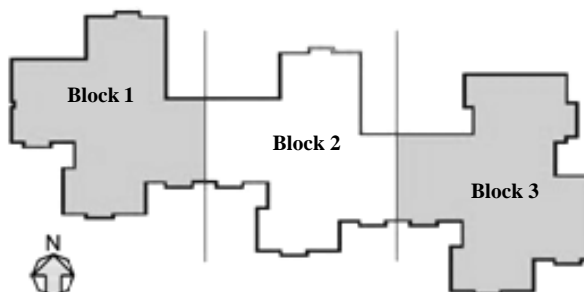

Figure 1 Position of three blocks of the building

3. QUALITATIVE VULNERABILITY EVALUATION OF THE BUILDING

Because the technical document of the building was not exist, as built architectural plan is drawn and rapid assessment of existing condition is conducted. For qualitative vulnerability evaluation of this building, a suitable method is selected [6].   In this method, construction information such as earth slope, soil kind, building height, opening dimensions, plan shape, load resisting system, floor diaphragm type and construction quality are considered. The result shows that the building will be damaged heavily at VII intensity and it will be collapsed at VIII intensity in the MSK scale [7].

4. QUANTITATIVE VULNERABILITY EVALUATION OF THE BUILDING

In the quantitative vulnerability evaluation, following steps performed. At first, the structural performance level is selected and data collection requirements level is specified. Then building configuration, in-place material properties and site characteristics are identified and seismic hazard studying is performed. After collecting information, the existing building is modeled and analyzed. In the following, these steps are described [7].

4.1. Structural Performance Level and Rehabilitation Objective

The basic safety objective is selected as based on the importance function of the building. According to FEMA356[5], the basic safety objective is a rehabilitation objective that achieves the dual rehabilitation goals of life safety building performance; level (3-C) for the BSE-1 earthquake hazard level and collapse prevention building performance; level(5-E) for the BSE-2 earthquake hazard level.

4.2. Technical Data Collection Requirements

With attention to the selected rehabilitation objective, the usual collection level of knowledge is considered and because of absence of sufficient information, a comprehensive condition assessment is conducted and the knowledge factor ,k, is selected as 1 according to FEMA 356[5].

4.3. Building Configuration

Because of absence of structural design drawings, the type and detail of structural elements includes columns, beams, connections and foundation are obtained with on-site investigation. For this reason, the similar elements from point of plan situation and load type is divided to five sets in each floor and three on-site investigations are conducted in each set.

The result showed that columns are double INP section with doubler plates. Beams in East-West direction are 2INP260 section with doubler flange plates that connected to two sides of column with top and bottom angles (Khorjini connection) according to figure 2 and in North-South direction CNP180 (castellated beam) with pin connection is used.

Also, on-site investigations of foundation showed that strip footings with link beams are used. The foundation thickness is 1.6 meter. In figure 3 foundation plan and column place for block 3 are shown. The diaphragm system is slab deck that set apart in separation distance of blocks. Also, the building is without lateral force resisting system such as steel bracing or concrete shear wall. In figure 3 the structural plan view of block 3 is presented.
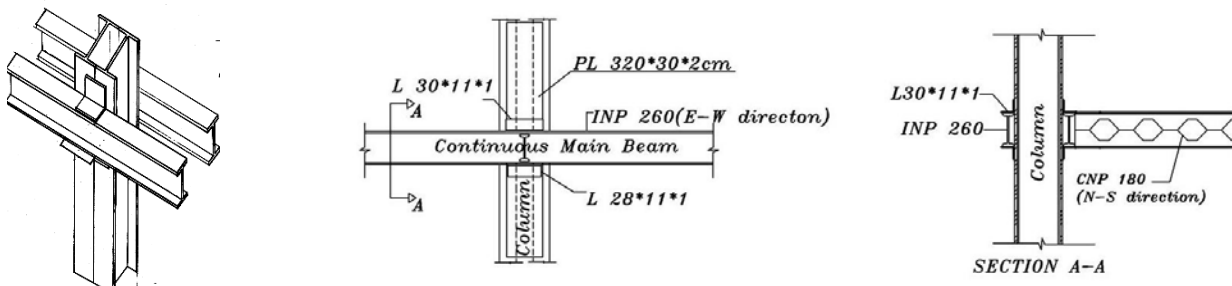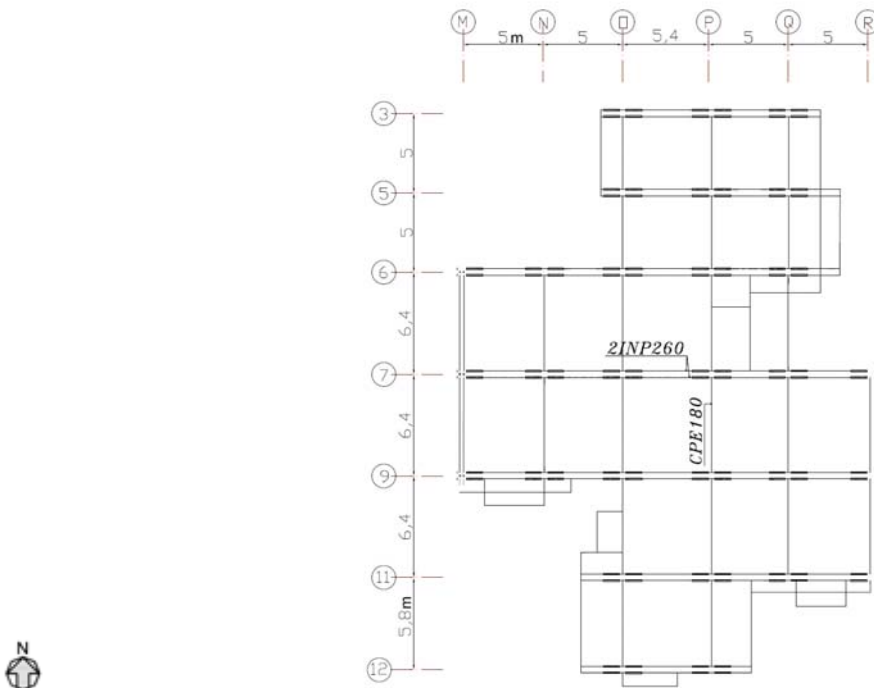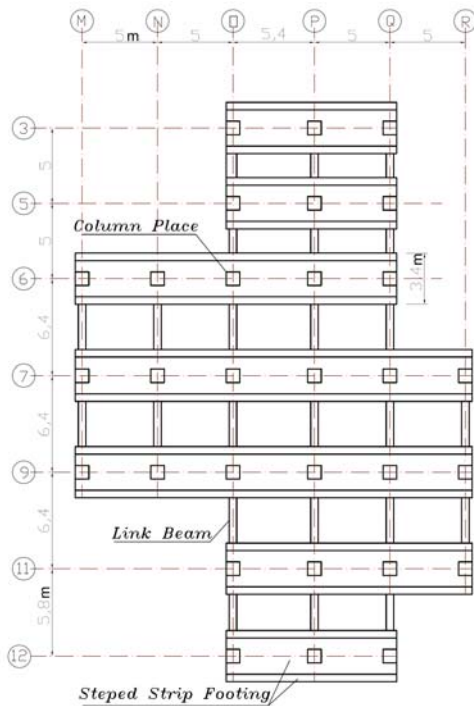

Figure 2 Detail of beams and connections

Figure 3 Foundation plan of block3 (left side), Structural plan view of block3 (right side)

4.4. Properties of In-Place Materials and Components

Properties of in-place materials and components are obtained according to FEMA356 [5] with usual tests. For this reason, sampling is taken place in regions of reduced stress such as flange tips at beam ends and external plate edges to minimize the effects of reduced area.

4.5. Site Characterization and Geotechnical Information

In order to identify the geotechnical site characteristics, four bore holes at corner sides of the building site drilled and the following results are obtained from testing of the samples:
  a) From point of geology, the building located on continental deposits. These deposits made up compact cemented sand and gravel with silts.
  b) The flow ground level water with a depth of 1.5 meter was at 15 meters below the ground surface and the site does not have liquefaction potential.
  c) The seismic bedrock surface is at 12 meters below the ground surface.
  d) The site class of soil below the foundation is C according to FEMA356 [5].

4.6. Seismic Hazard and Site-Specific Response Spectra

In seismic hazard analysis, peak ground acceleration for the specified earthquake return periods with probabilistic procedures are obtained and then based on site characterization, the value of the peak acceleration is calculated at the ground surface level as given in table 1. Then, based on seismotectonic and seismological site features, the suitable time history accelerations are selected and for damping ratio of 5%, response spectra are drawn. After statistics analysis, the mean response spectra and the mean response spectra plus standard deviation are obtained.

Table 1 Peak acceleration for earthquake return periods at the ground surface level

Return period	Peak horizontal acceleration(g)	peak vertical acceleration(g)
475years	0.39	0.3
2475years	0.66	0.51

4.7. Structural Modeling of Existing Building before Rehabilitation

Because of distance between three blocks, each of them is modeled separately. For modeling of beams and columns, frames element and for Khojini connections modeling, link elements are used. Characteristics of connection link elements obtain based on finite element analysis. For this reason, the INP beam with top and bottom angles modeled with shell element and then, the nonlinear analysis is conducted and the moment-rotation curve of semi rigid connection is drawn as shown in figure 4. The simulated two linear curve is used as link element properties.
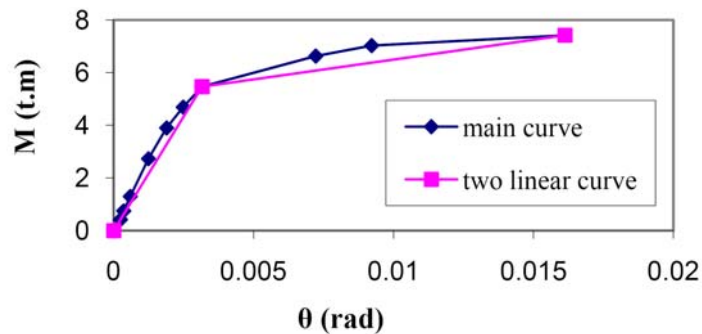

Figure 4 The moment-rotation curve of connection

4.9. Results of Quantitative Vulnerability Evaluation

After modeling of existing building, it analyzed with response spectrum procedure according to FEMA356 guideline [5]. The results shown that the structure is heavy vulnerable against the seismic lateral loads. For example the stability coefficient θ_i exceeds 0.33, so the structure shall be considered unstable (section 3-2-5-1-1 FEMA356) and the rehabilitation design must be considered to reduce the computed lateral deflections [3].

$$\theta_i = \frac{P_i \delta_i}{V_i h_i} = 0.734 > 0.33 \rightarrow unstable$$

5. THE SEISMIC RETROFITTING PLANS FOR THE BUILDING

At the next step, two seismic retrofitting plans consist of concrete shear walls and steel shear walls are proposed to supply the strengthened structure, with enough stiffness, strength and ductility according to FEMA356 guideline [5] criterions. For designing of these plans besides of obtaining lateral load resisting system, tried to solve another problems of structure such as long cantilever beams and torsion of structure caused from asymmetric plan and non-compatibility of mass center and stiffness center. In addition, construction of the renewal plans produces minimum of destruction and has minimum influence on architectural aspects. In the following the details of these two seismic retrofitting plans are explained.

5.1. The First Seismic Retrofitting Plan: Concrete shear Walls

Using concrete shear walls may be one of the best solution for retrofitting the existing seismic unstable building. In this plan, a core wall, U shape wall and one concrete shear wall are used as shown in figure 5. Core wall is located at escape stair box and U-shape wall covers 3 meters length cantilever beams. Concrete shear walls position is selected so that to optimize the technical aspects and construction features of the plan. The first choice of structural retrofitting plan is analyzed with response spectrum procedure and acceptance criteria of each element are controlled according to FEMA356 guideline [5]. The summary results of structural elements acceptance criteria are shown in table 2.
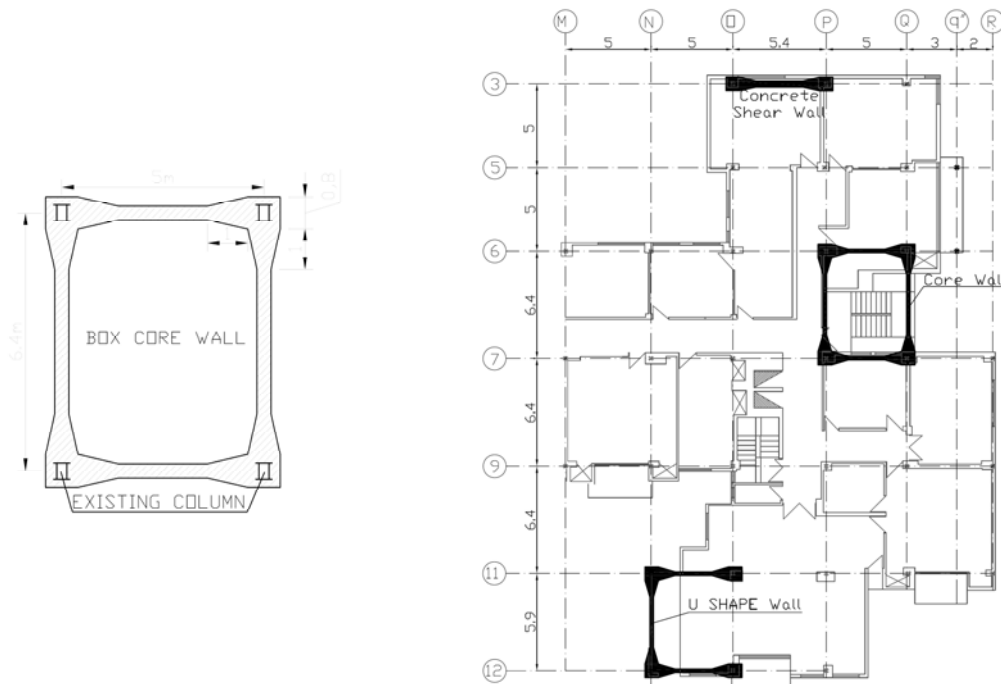

Figure 5 The first retrofitting plan (adding concrete core wall)

Table 3 the summary results of structural elements acceptance criteria (the first plan)

Structural elements	BSE-1 Earthquake Hazard Level	BSE-2 Earthquake Hazard Level
columns	All answered	10% of them require retrofitting with max ratio=1.25^{**}
beams	All answered	All answered
connections	12% of them require retrofitting	25% of them require retrofitting
foundations	10 piles required	16 piles required

**The max ratio is value of equations (5-10) to (5-11) of FEMA356 [5] for acceptance criteria of columns

The rehabilitation design for existing weak columns is conducted with welding doubler plates to web and flanges of columns section. In figure 7, a sample of retrofitted column is shown (The adding plates hatched).
For rehabilitation design of semi-rigid Khorjini connections which did not satisfy the acceptance criteria (table 2), because the strength of the connection is low compare to stiffness, it is decided to reduce the stiffness of connections. In order to reduce the stiffness, top angle of required connections cut and the semi rigid connections converted to the pin connections, then for supplying out of plane stability of the connections, the new angles added to the connections as shown in figure 6. The stiffness of converted connections is modified in the structural model and the structure redesigned.

For foundation rehabilitation design, the angled belled piles placed between the existing footing strips and by adding top and bottom bars which are fully cover the foundation surface, the strip footing converted to mat foundation. In figure 7, the retrofitted foundation plan is shown.
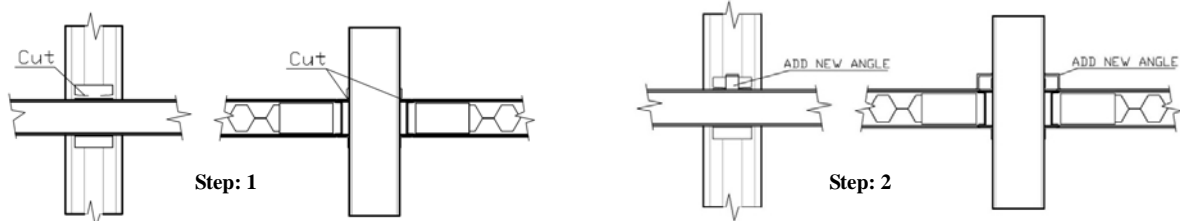

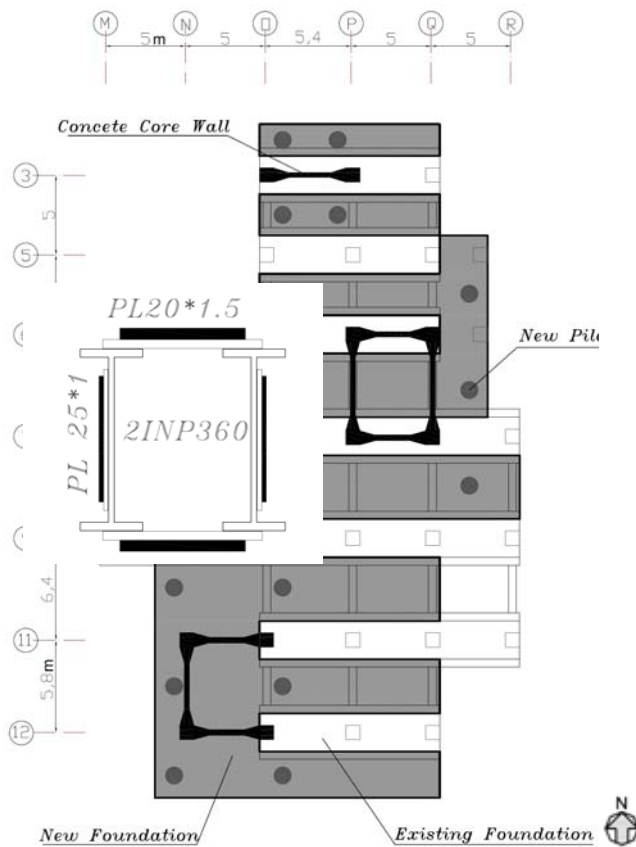Figure 6 The used method of connection pinning

Figure 7 The retrofitted plan of foundation (right side), Sample of retrofitted column (left side)

5.2. The Second Seismic Retrofitting Plan: Steel Shear Wall

In the past three decades, steel shear walls widely are used as lateral resisting force systems. The location of these walls selected such as the previous retrofitting plan which is shown in figure 8. The second choice of structural retrofitting plan is analyzed also with response spectrum procedure and acceptance criteria of each element are controlled according to FEMA356 guideline [5]. The summary results of structural elements acceptance criteria are shown in table 3. The steel shear walls as shown in figure 9 have horizontal stiffeners in each third of height and vertical stiffeners in each third of frame length which are designed based on procedures

presented by Astaneh-Asl [8]. The rehabilitation design method for existing weak semi-rigid connection and foundation is such as the first seismic retrofitting plan in section5.1. But for high demand ratio of columns which are connected to steel shear walls, adding doubler plates is not sufficient for retrofitting them and it is necessary to use other retrofitting methods such as spreading the steel shear walls in more frames.

Table 3 The summary results of structural elements acceptance criteria (second plan)

Structural elements	BSE-1 Earthquake Hazard Level	BSE-2 Earthquake Hazard Level
columns	16% of them require retrofitting with max ratio[**]=2.4	40% of them require retrofitting with max ratio[**]=4.12
beams	All answered	All answered
connections	18% of them require retrofitting	30% of them require retrofitting
foundations	9 piles required	14 piles required

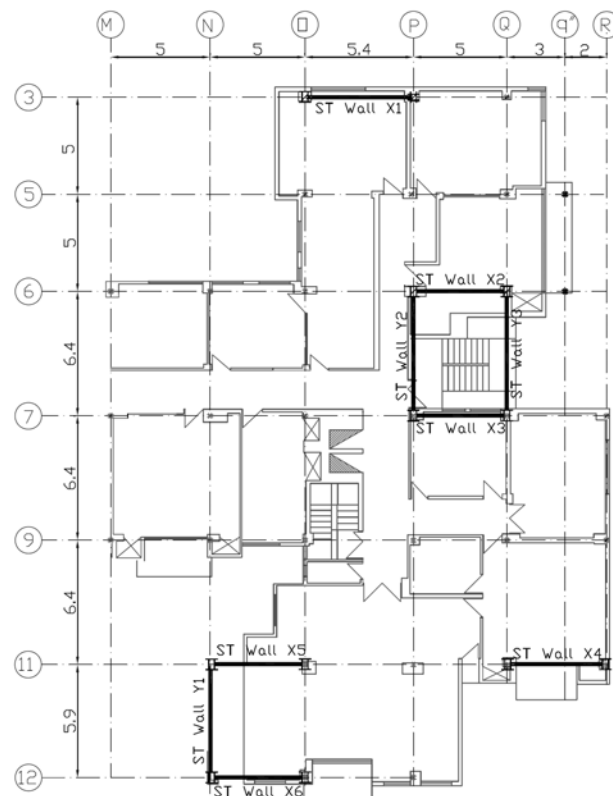**The max ratio is value of equations (5-10) to (5-11) of FEMA356 for acceptance criteria of columns

Figure 8 The second retrofitting plan (adding steel shear walls)

6. THE COMPARISON BETWEEN THE SEISMIC RETROFITTING PLANS

At the previous sections, two following seismic rehabilitation plans proposed for this vulnerable building:
- Adding concrete shear walls
- Adding steel shear walls

From point of architectural aspects, both plans have the same effects on the existing architectural plan of the building. From point of seismic performance and implementation aspects, behavior of building with concrete shear

walls is overally better than building with steel shear walls, For example the top maximum displacement of building with concrete shear wall is about 30% less than building with steel shear walls and the connections and columns required to retrofit in the building with concrete shear walls are much lesser in comparison with the building retrofitted by steel shear walls.

Therefore, the concrete shear wall selected as the superior choice for the rehabilitation of this building.
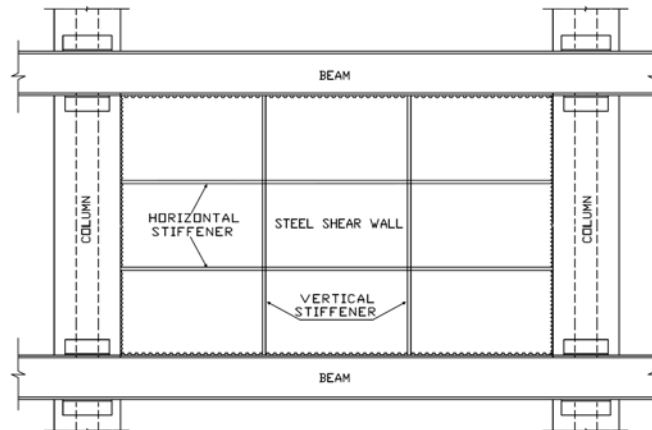

Figure 9 The steel shear wall stiffeners

7. CONCLUSION

The following results obtained from this study:

- The comparison of the seismic retrofitting plans of this building showed that concrete shear walls plan is the best choice from point of seismic performance and implementation aspects.
- For rehabilitation design of semi-rigid Khorjini connections which unsatisfied the acceptance criteria, as the strength of the connections is low compare to their stiffness, conversion of the semi rigid connections to the pin connections is provided better performance.
- The comparison of the response spectrum analysis procedure results for two seismic hazard levels showed that collapse prevention performance level controlling of this building caused 10 percent addition in column section retrofitting and 20 percent addition in design of concrete shear walls sections.
- Using steel shear walls for rehabilitation of high-rise buildings needed strong column sections or spreading the steel shear walls in more frames which may affect the existing architectural plan.

REFRENCES

[1]Moghaddam, H., Karami, R. (1991). Mechanical Properties of Khorjini Connection, M.Sc. Thesis of Sharif University, Iran (In Persian).

[2]Tehranizadeh, M. and Maleki, M. (1993). Performance Behavior of Khorjini Connection under Dynamical Loads, MS Thesis of Amirkabir University, Iran (In Persian).

[3]Barkhordari, M.A., Foroghi,M. and Zahedi, M. (1993). Studing of Khorjini Connection and Presenting a Method for Retrofitting them, MS Thesis of Elmosanat University, Iran (In Persian).

[4] Mazruei, A. and Mirghaderi, M. Dehghani, H. (2000). Experimental and Theorization Studying of Khorjini Connection and Presenting a New Rigid Connection, MS Thesis of Tehran University, Iran.(In Persian)

[5] FEMA356, (2000). Prestandard and Commentary for the Seismic Rehabilitation of Buildings, Federal Emergency Management Agency. Washington, D.C.

[6] Shakib, H. and Dardaei, S., Moghaddasi, A. (2007). Instruction for Seismic Rehabilitation of Existing Unreinforced Masonry Buildings, Management and Planning Organization, Report No.376, Iran (In Persian).

[7] Shakib, H and Moghaddasi A., Dardaei, S., M. Pirizadeh (2006). Vulnerability Evaluation Seismic Rehabilitation

Study Report of Shahed Building, Natural Disaster Research Institute of Iran (In Persian, Report).

[8] Astaneh-Asl, A. (2001). Seismic Behavior and Design of Steel Shear Walls, Steel Tips Report, Structural Steel Educational Council.