DYNAMIC CHARACTERISTICS OF KHORJINI SEMI – RIGID CONNECTIONS, USING FORCED VIBRATION TEST OF $^1/_2$ – SCALED MODEL OF A 4 – STORY STEEL STRUCTURE

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ABSTRACT

Khorjini (semi- rigid) connection which are used in more than 90% of steel structures and in approximately 78% of buildings in Iran is a connection in which continuous beams crosses several columns by means of two angles on each side of column (or one angle on one side of column) . In this connection bending moment is partially transferred to the column, and moments overpasses from one span to other span. Khorjini connection is neither rigid nor simple and its rotational stiffness is not known. This connection is highly vulnerable to earthquake as it was observed in Manjil earthquake, of June 21, 1990 . In this earthquake most of failures to the steel structure were due to the failure of Khorjini connections. Up to this point in order to recognize the dynamic behaviour of these types of structures, many component tests has been done. For better understanding the dynamic behaviour of these types of structures, forced vibration test of $^1/_2$ – scaled model of a 4- story steel structure with Khorjini connection is used. In this test natural frequencies, mode shapes, and damping values for all modes of vibrations determined. Comparing the dynamic characteristics of the mathematical model with the physical model, the actual stiffness of the connection have been determined. Also using the results, seismic capacity of these kinds of structures have been found and their vulnerability to earthquake and methodology of their strengthening will be proposed. Effect of bracings on dynamic characteristics and modal natural frequencies of Khorjini semi – rigid connection is also determined, by using various bracings in eastern and western sides of the structure.

1. INTRODUCTION

Khorjini connection, which is used in approximately 78% of buildings in Iran , is a connection in which continuous beams crosses several columns by means of two angles on each side of column (or one angle on one side of column) as it can be seen in Figure 1. Following the June 21, 1990 Manjil earthquake of Iran and the collapse of these type of steel structures, especially due to connection and arch roof system (Fig2,3) a series of research has been focused on these problems. ranging from mathematical modeling and analysis to the experimental studies of typical connections (Karami et al. 1987 ; Kouhian Moghdam, 1995). Jack- arch flooring systems (Maheri, 1994; Ghafory – Ashtiany & Tiv, 1995; Tehranizadeh & Alavi, 1997;Ghafory – Ashtiany & kazem, 1997).

Most of the low – rise(1- 6 stories)steel structures in Iran have two distinct differences with the "classical " steel – framed structures. First, for the beam – to – column connections as is shown in Figure 4 . a pair of continuous beams, which crosses several columns. connect to the sides of columns by means of angle sections. This type of connection, which is known as "Khorjini".

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saves on erection time , and labor cost. In addition, because of the limitations on the availability and the cost of deep rolled sections, this technique is currently used in most cases. In these connections out of plane partial beam- to – columns transfer of bending moments and early onset of failure in the angles is most likely the cause of failure under lateral forces. The second difference is the construction of the roof system which is the Jack – arch masonry floors. As it is shown in Figure 4a, simply connected steel joints are used to bridge the main frames. The space between these joists are filled with bricks and mortar as a bonding agent in a shape of an arch with approximately 3-8 cm from the head of the arch to its toe within a one meter span (fig 4b). This type of flooring system lack of rigidity required For a well – behaved diaphragm action and hence, collapses easily during earthquake. For better evaluation of the dynamic behavior of these types of structures and propose the proper formulation for aseismic design as well as the strengthening method for existing structures, a 1/2 scale , typical 4- story model having the above two characteristics was built on a strong floor at IIEES. The design and construction of this model was based on the common practice of Iranian engineers and steel workers.

2. MODEL DESIGN

The 1/2 – scaled model of the 4-story structure designed and constructed as shown in Figure 5, based on Iranian Building Codes. In the following sections the various characteristics of the model are described.

2.1 Scaling Factor and Floor Details

In the case of experimental studies of a model structure, it is desirable to choose a scaling factor, which is as close to unity as possible. However, several constraints control the scaling factor which in this case are: 1) the size of the strong floor which in our case was only 3x3m; 2) the smallest available size of rolled section which in Iran is INP- 80 (I-beam with a section modulus of S=20cm3) ; and 3) having at least three frames each with at least two spans in order to create the " Khorjini" connection. Considering these constraints, a scaling factor of 2 and a floor system similar to that shown in Figure 4 with center – to – center distance of columns(in the y – direction) of 2m have been selected. The minimum two span requirement of each frame called for a total of 4m for the width of each frame and in order to compensate for the 1m shortening of the strong floor; an extended rigid frame were installed on strong floor. Thus the three frames of the model were erected on extended rigid frame. The center – to center distance of the columns in the x- direction became 1.5m. Nine joists with a distance of 0.5m filled with 1/2 – scaled masonry bricks (10x5x2.5cm), in the form of an arch covered the space between each frame . In order to compare the effect of the Khorjini connections on the natural frequencies and mode shapes of the physical model with those of a mathematical model with rigid connections, the effect of other parameters, especially the question about the diaphragm action had to be eliminated. Thus, in this test, horizontal X- bracings were used to add to the rigidity of each floor to test its effect as well.

2.2 Number of Floors

The number of floors have been selected based on the aforementioned constraints and that by the available shakers, enough force would be generated that the model structure reaches yield connection. Sinusoidal vibration exciters are able to produce larger forces in higher frequencies and also the higher this applied force from the base the easier is to bring the structure to yield. Consequently ; a 4 – story high structure was selected which is also very typical in Iran.

2.3 Frame Design

As a common practice , the column line that supports the largest floor area would be selected to design the columns and used for the entire structure. Considering the story height of 1.5m (3 m in prototype) and 900 kgf/m2 dead load. A pair of channels (UNP-50) with a back – to – back distance of 7cm laced together at very 10 cm intervals was selected for the model. The dynamic analysis of the model showed large enough displacements in the 3rd resonance frequency that we would definitely be able to force the model into nonlinearity. To provide the equivalent lateral resistance that is provided by brick infilled frames in the actual structures, and also to avoid undesirable effects or even accidents. The model structure was firmly X-bracings as a safety measure against in the y – direction. The effect of 4 type of fixed bracing was tested as well.

2.4 Design of Connection

The criteria used in designing angles for the Khorjini type beam – to – column connections are based on the geometry and the dimensions of the connection. As shown in figure 4c, a plate is first welded to the column on each side. The lower angles are then welded to this plate. Cross sectional dimensions of these angles are so picked that they exceed the width of the beam flanges. This would leave enough space for welding the angles to the beams. The plates and the lower angles are welled to the column when they are prepared laid flat on the ground and therefore the quality of these welds are better than the ones after erection. When columns are erected and beams are place over the lower angles and welded, smaller top angles are added mainly to keep the assembly in place .

3. FORCED VIBRATION TEST

For forced vibration test of the model, a pair of harmonic exciters was utilized with capability of inducing translational and torsional dynamic forces at the top floor of the structure. The exciters can be adjusted to produce maximum force 16.1 ϖ^2 where ϖ is the operating frequency that can vary from 1 to 20 Hz. The responses were measured by force balance accelerometers and recoded on 8 – channel data acquisition system. Throughout this project in order to develop a new algorithm for system identification, the required added mass were added at four different stages (m = 0.0, 1.0,3.0 and 7.0 ton).

3.1 Dynamic Characteristics

The recorded acceleration at various point of structure included noise due to nonstructural interactions present in the system, and electrical wiring. A process was used to remove the noise and compute the response amplitudes and phase differences with respect to the input force. The response frequency is the same as the input frequency when structural behavior is liner. In other words, if one assumes p(t) = po cos t for the input force, the observed response Z (t) will have the following forms:

$$Z(t) = Y(t) + n(t) \tag{1}$$

In which, Y(t)= po $|H(\omega)|\cos(\varpi\ t\alpha-\theta)$ is the actual response and n(t) is the noise. Cross correlating the input with output function. We obtain

$$R_{pz}(\tau)=\tfrac{1}{T}\int_0^T P(t)\quad Z(t+\tau)dt \tag{2}$$

$$R_{pz}(\tau)=\tfrac{1}{T}\int_0^T P_o\cos\varpi t\times P_O|H(\omega)|\cos(\varpi t-\theta)dt \tag{3}$$

$$R_{pz}(\tau)=\tfrac{1}{T}\int_o^T p_0^2|H(\omega)|\int_0^T\varpi\quad t\cos(\varpi t-\theta)dt+R_{pn}(\tau) \tag{4}$$

$$R_{pz}(\tau)=\frac{1}{2}P_o|H(\omega)|\cos(\varpi t-\theta)+R_{pn}(\tau) \tag{5}$$

$R_{pn}(\tau)$ in Eq.(5) tends to zero if long enough records are available. Thus, by computing $R_{pz}(\tau)$ and obtaining its amplitude and phase. Values of $P_o|H(\omega)|$ and θ can be easily evaluated.

Figure 6 shows a sample of measured acceleration response spectrum at fourth story. In this figure , the structure natural frequencies of 2.24, 7.94 and 16.1Hz can be easily identified. The 4^{th} mode natural frequency is more than 20Hz, which can not be measured in this case. Table 1, shows the result for various added masses on the floor. The displacement spectrum of the first mode at various levels of the structure is shown in figure 7 .

It can be seen that displacement of all stories reaches to its maximum amount at frequency near 1^{st} mode (2.24 Hz). By the use of this information and the phase angles, the vibration mode shapes have been obtained. The following matrices shows the vibration mode shapes for different story added masses.

$$\underset{(m=0kg)}{\phi=}\begin{bmatrix} 1.0 & 1.0 & 1.0 \\ 0.76- & 0.51- & 2.19 \\ 0.51 & 1.49 & 0.05 \\ 0.20- & 0.96 & 2.44 \end{bmatrix} \qquad \underset{(m=1000kg)}{\phi=}\begin{bmatrix} 1.0 & 1.0 & 1.0 & 1.0 \\ 0.82- & 0.09- & 1.21- & 1.31 \\ 0.55- & 0.94- & 0.01 & 2.01 \\ 0.22- & 0.69 & 1.59- & 1.59 \end{bmatrix}$$

$$\underset{(m=3000k}{\phi=}\begin{bmatrix} 1.0 & 1.0 & 1.0 & 1.0 \\ 0.81- & 0.16- & 0.32- & 1.59 \\ 0.54- & 1.0- & 1.06 & 1.98 \\ 0.22- & 0.75 & 1.50- & 1.60 \end{bmatrix} \qquad \underset{(m=7000kg)}{\phi=}\begin{bmatrix} 1.0 & 1.0 & 1.0 & 1.0 \\ 0.84- & 0.02- & 0.99- & 1.66 \\ 0.57- & 0.83- & 0.19 & 1.91 \\ 0.24- & 0.66 & 1.22- & 1.53 \end{bmatrix}$$

3.2 Torsional Mode

For the first torsional test, the mass of each floor 1000kg has been used. Figure 8 shows a sample of measured displacement response spectrum of the first mode of torsionally excitation at various levels of structures with 1000kg added mass. Table 2 shows the first mode of torsional frequency for different type of hinge, Khorjini and rigid frames for various added masses on the floor.

3.3 Damping Ratios

The damping ratio was calculated with different approaches using the result of the free vibration test of the model structure as well as the free vibration part of the record of the forced vibration test. Using the logarithmic decrement method. the average damping ratio becomes $\xi = 0.60\%$. By other more reliable

methods such as half – amplitude or half – power method, the damping ratio of the first three modes are 0.55%, 0.51% and 0.29% in case of zero added mass on each floors. In the case of complete $^{1}/_{2}$ scale model with average of 0.83% (when the added mass on each floor is 7 ton), the calculated damping ration shown in Table (3).

It should be noted that these values are less than the actual damping ratio of the structure, These differences are due to the existence of infill walls in the frames of real structure , which has not been included in that stage of the test . As shown on Table (3), the lateral damping ratio and the torsional damping ratio are very close to each other.

3. ANALYSIS OF THE MATHEMATICAL MODEL

Semi- rigid connection can be divided in two groups of continuous and discrete connections. In order to model the continuous semi- rigid , Khorjini connection a spring element at the connection of beam and column have been used (Chan & Masuoka, 1987).This element in Sap90 program is in the form of frame element that works between beam and column and has very large flexural stiffness in two directions and the torsional stiffness equal to $k = G.J/L$ as shown in Figure 9. The model structure with assumed connection's has been analyzed. Figure 10 shows the variation of model frequencies versus the connection's stiffness in the case of zero, 1000kg and 3000kg added mass. It can be seen that the model frequency changes when connection's stiffness are between 1 to 1000t-m/rad and become insensitive as the stiffness become larger than 1000t- m/rad. In other words, the connection with stiffness larger than 1000t-m/rad can be considered as a rigid connection. Figure 11 shows that for the first three model frequencies of the structure the connection stiffnesses are 50, 150 and 80 t-m/rad respectively. Using these connection stiffnesses and analyzing the structure, it was found that the calculated frequencies are different from the forced vibration test. These result are shown in Table 4.

5. EFFECT OF X- BRACING ON MODEL FREQUNCIES OF KHORJINI CONNECTION

In the course of these studies, the effect of various type of bracing systems (X- bracing , K-bracing , EBF- bracing and KNEE – bracing) on the dynamic characteristic of the system was tested. Table 5, shows the effect of various bracing systems on model frequencies in comparison with no bracing when using 7000kg masses on each floor.

6. DISCUSSION

From the experimental test and numerical analysis , one may conclude the following result:

- The result of the forced vibration test and those from the mathematical model are presented in the form of natural frequencies , mode shapes and percentage of damping ratios. Table 1 and 3 . The natural frequencies as the peak resonance values for zero added mass is shown in Figure 6 . In this figure as the modes of vibration increases. the values of acceleration spectrum of fourth story decreases.
- Table 2, shows the comparison between frequencies of hinge. Khorjini and rigid connection for first mode of torsional frequency. As it was observed the torsional frequencies for all these kinds of connection decreases as floor masses increases.
- Analysis and experimental frequencies for different story added masses and connection stiffness and their errors are shown in Table 4. In this table, the percentage of error for the first mode of vibration is zero, and as the modes of vibration increase the percentage of error increases too.
- Table 5 shows the effect of various bracing on natural frequencies of Khorjini connection. The frequency of vibration for all modes of vibration is increased in the of bracing. This increase in frequency in case of X-bracing for first mode of vibration is 272% and for 4 th mode of vibration is 105%.
- Figures 11 shows the variation of torsional frequencies versus the connection's stiffness in the case of zero, 1000kg and 3000kg added mass. It can be seen tat the torsional frequency changes when connection's stiffness are between 10 to 1150 t- m / rad and becomes insensitive on the stiffness larger than 1150t- m / rad In other words the connection with stiffness larger than 1150 t- m / rad can be considered as a rigid connection.

7. CONCLUSION

The result of this study show that a) The rotational stiffness of Khorjini semi- rigid connection is 1340t- m / rad. b) Most of failure of structures using Khorjini semi- rigid connections during earthquake is due to lack of resistance of connection to the lateral loads, and large response of the structure in comparison with rigid connections. In order to increase the resistance of the Khorjini semi – rigid connection against earthquake, the structure should be braced or the connections should be reinforced. C) The damping ratio of structure using Khorjini semi- rigid connections is between 0.5% to 1.5% and when this type of structures is braced with X-bracings, the damping ratio will further increase up to 7.5%. In real structure with Khorjini connections, the damping ratio will increase because of infill and nonstructural elements.

Table 1. Model natural frequencies (Hz). For different story added masses (kg)

Added Mass Mode	0	1000	3000	7000
1	2.24	1.99	1.34	1.06
2	7.94	6.44	4.52	3.45
3	16.1	12.16	8.57	6.28
4	-	17.54	12.24	8.75

Table 2. First mode of torsional frequency (in Hz) of hinge, Khorjini and rigid Connections model for various added mass(kg)

Tape of Model	Added Mass	Hinge Connection	Khorjini Connection	Rigid Connection
1	0	9.00	9.12	9.25
2	1000	8.09	8.22	8.30
3	3000	5.64	5.72	5.78

Table 3. The damping ratio (in percent). For different added masses(kg).

Mode No.	m=0	m=1000	m=3000	m=7000
1	0.55	1.08	0.77	1.42
2	0.51	0.44	0.99	1.02
3	0.29	0.38	0.76	0.61
4	0.42	-	1.03	0.60
5	-	0.57	1.36	0.49

Table 4. Result of analytical and experimental frequencies and their error(Hz)

Story Added Mass (kg	Connection Stiffness (t-m/ rad)	1st Mode			2nd Mode			3rd Mode			4th Mode		
		Ana.	Exp	%err	Ana	Exp.	%err	Ana.	Exp.	%err	Ana.	Exp.	%err
		2.24	2.24	0	8.31	7.94	4.6	18.2	16.1	13.0	29.5	-	-
		1.99	1.99	0	6.89	6.44	7.0	14.0	12.2	14.9	21.6	17.5	23.0
		1.34	1.34	0	4.71	4.52	4.2	9.78	8.57	14.1	15.4	12.2	25.6

Table 5. Model frequencies for different lateral resistance system (Hz)

Mode No.	No Bracing	X-Bracing	K- Bracing	EBF-Bracing	KNEE-Bracing	%Diff. X Bracing vs No Bracing
1	1.05	3.95	3.5	2.5	1.7	272
2	3.45	11.40	10.00	7.00	5.50	230
3	6.28	15.20	14.00	10.50	8.70	142
4	8.75	18.0	16.50	13.00	11.50	105

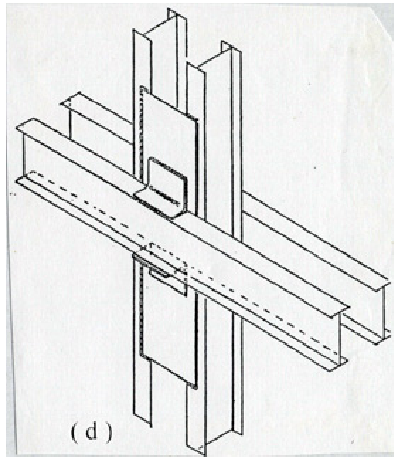

Fig. 1. Shape of a Khorjini connection

Fig. 2. A typical failure of Khorjini connection in Manjil Earthquake

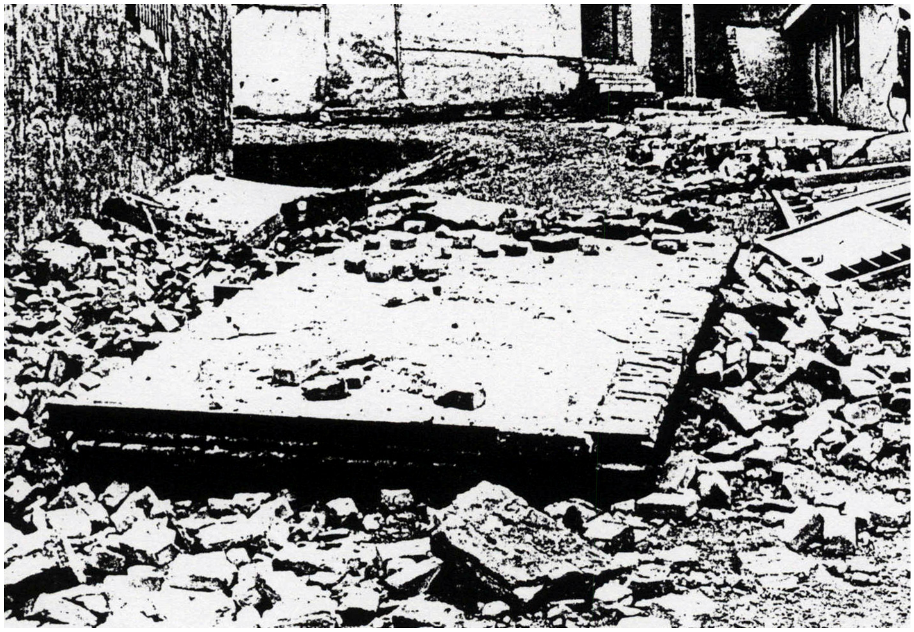

Fig. 3. A typical roof failure in Manjil earthquake

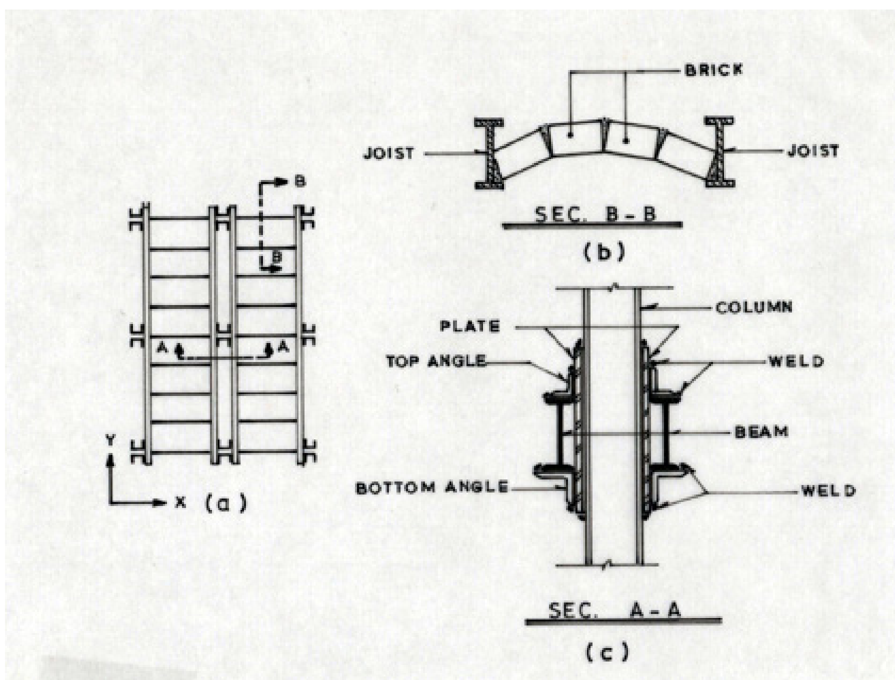

Fig. 4. A typical floor plan and connection details

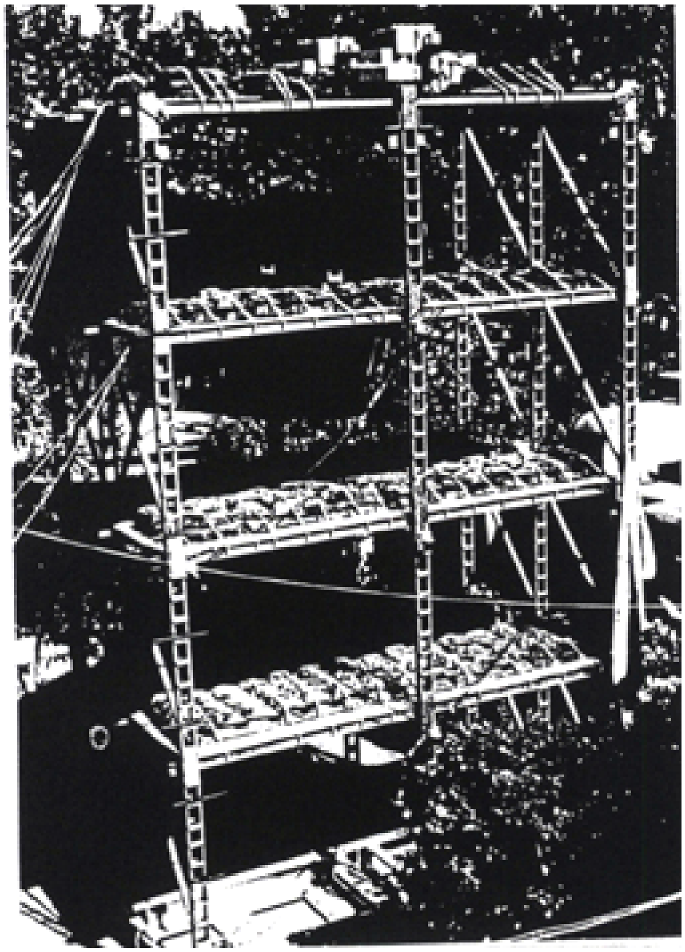

Fig. 5. The 1/ 2 scaled model of the 4- story structure

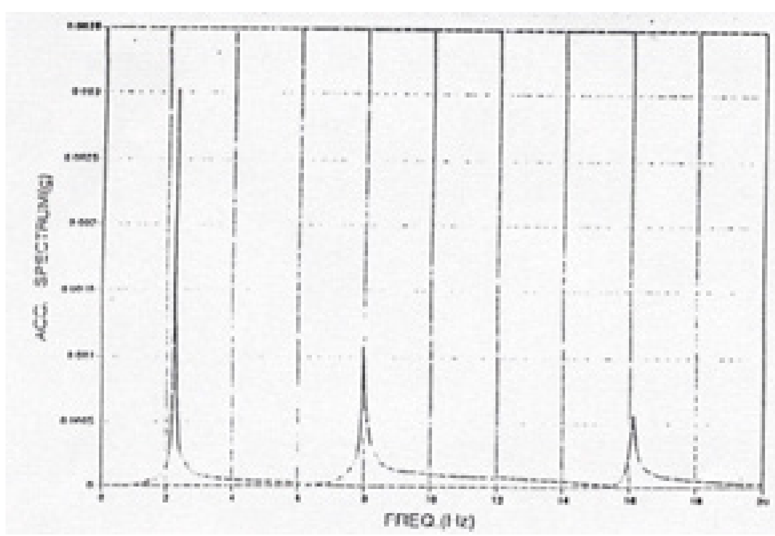

Fig. 6. Acceleration response spectrum at fourth story (channel 2)

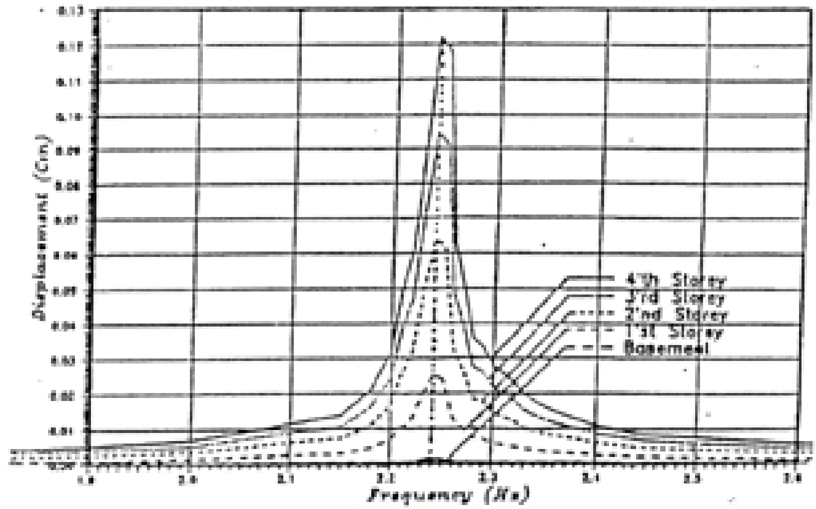

Fig. 7. First mode displacement spectrum for zero added mass

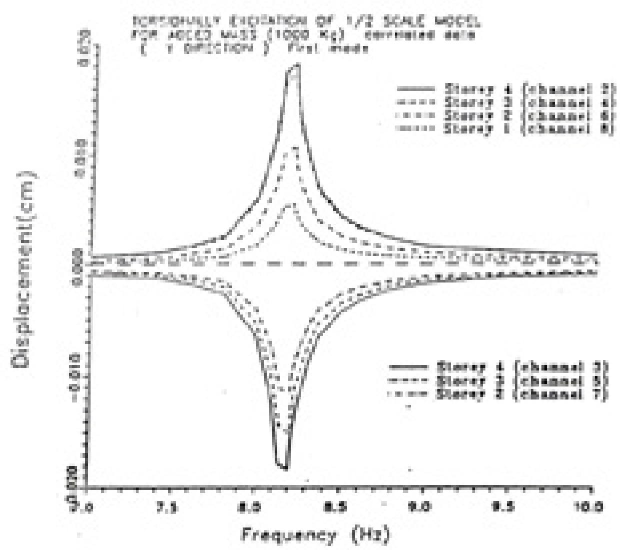

Fig. 8. Displacement response spectrum of the first mode of torsionally excitation at various levels of structure with 1000 kg added mass.

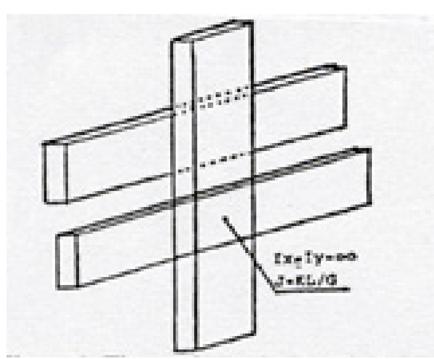

Fig. 9. The mathematical model of beam- column connection

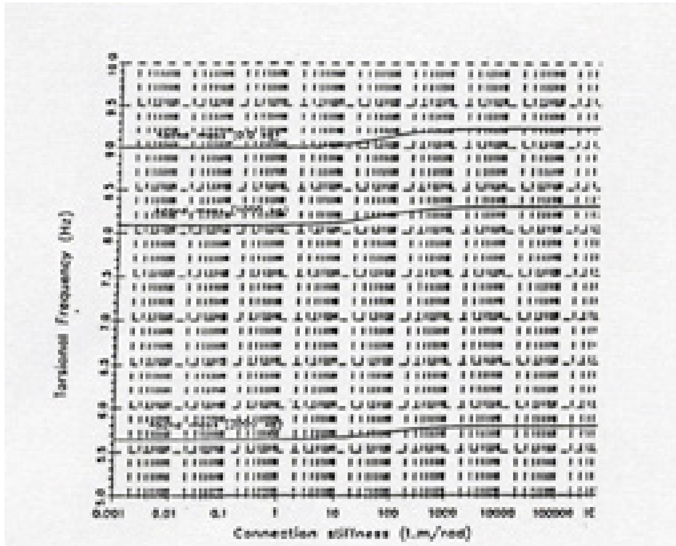

Fig. 10. Variation of torsion frequencies versus the connection stiffness for three cases of zero 1000kg and 3000kg added mass.

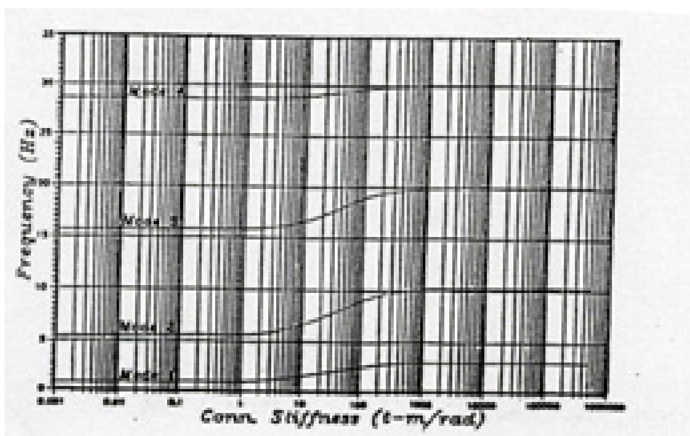

Fig. 11. Computer analysis result of structure for different connection's stiffness

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