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EFFECT OF SEMI-RIGID "KHORJINEE" CONNECTIONS IN DYNAMIC RESPONSE OF STEEL STRUCTURES

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ABSTRACT

For studying and identification of dynamic behavior of the semi-rigid beam-to-column "Khorjinee" connection which is commonly used in low rise and typical steel structure in Iran, a comprehensive forced vibration test on half scale model of a typical 4-storey steel structure with Khorjinee connection has been performed at IIEES. In this paper the comparison of forced vibration test with the dynamic analysis of mathematical model using rigid and hinge connection have been presented. Dynamic characteristic of structure (frequencies, mode shapes and damping ratios) based on the assumed connection stiffness have been obtained. As a result of these studies, a method is presented for dynamic modeling of the "Khorjinee connection".

KEYWORDS

Khorjinee connection; Semi-rigid connections; Steel structure; Dynamic; Earthquake; I.R.Iran.

INTRODUCTION

Semi-rigid connections in steel structures due to simple details and possibility of tuning the connection's stiffness which can optimize the distribution of moment between connected elements has taken considerable attention in recent years. An special type of semi-rigid beam-to-column connections named as "Khorjinee connection" has been developed in past fifty years by practicing engineers in Iran because of its simplicity and economic advantages. Also most of the existing steel structure were not designed to resist lateral loads. However the experiences of recent earthquakes in Iran, especially Manjil earthquake of 20 June 1990, shows the poor behavior of Khorjinee connection and that most of the common steel structures fail due to its joint failure(IIEES reconn. report,1991). Several theoretical and experimental research has been performed to study the static behavior of this connection as well as its workability, stiffness and strength using different models (Karami *et al.*,1994, Ghafory-Ashtiany *et al.*, 1995, Tiv *et al.*, 1995). It was found that the behavior of this widely used connection can not be modeled by classical semi-rigid connection and that special model has to be assumed that satisfy its dynamic behavior as well. For purpose of studying the

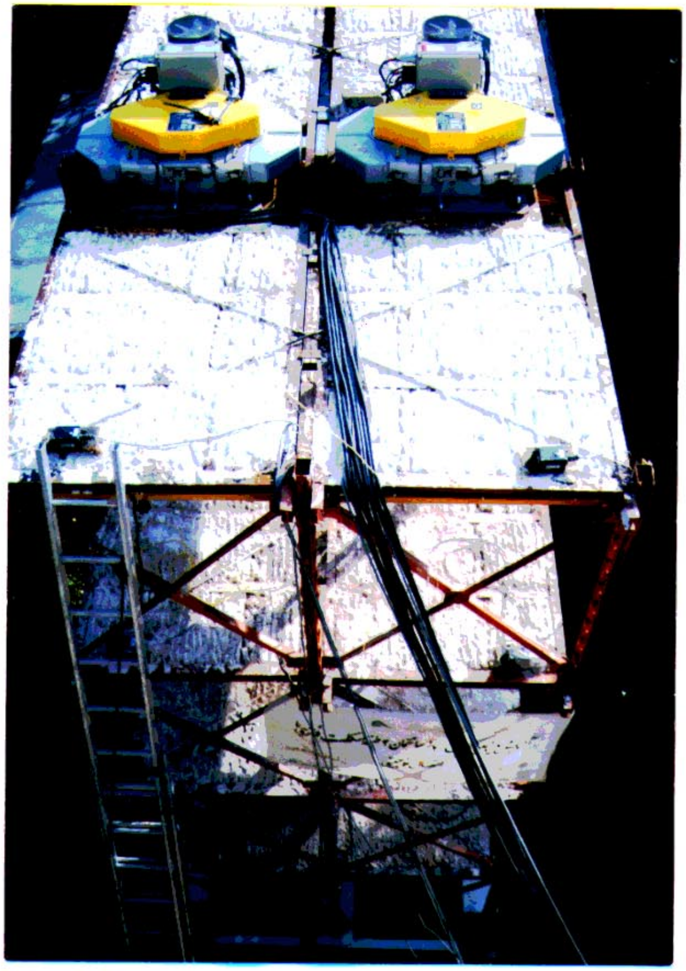

Fig. 1: The 1/2 scaled model of steel structure

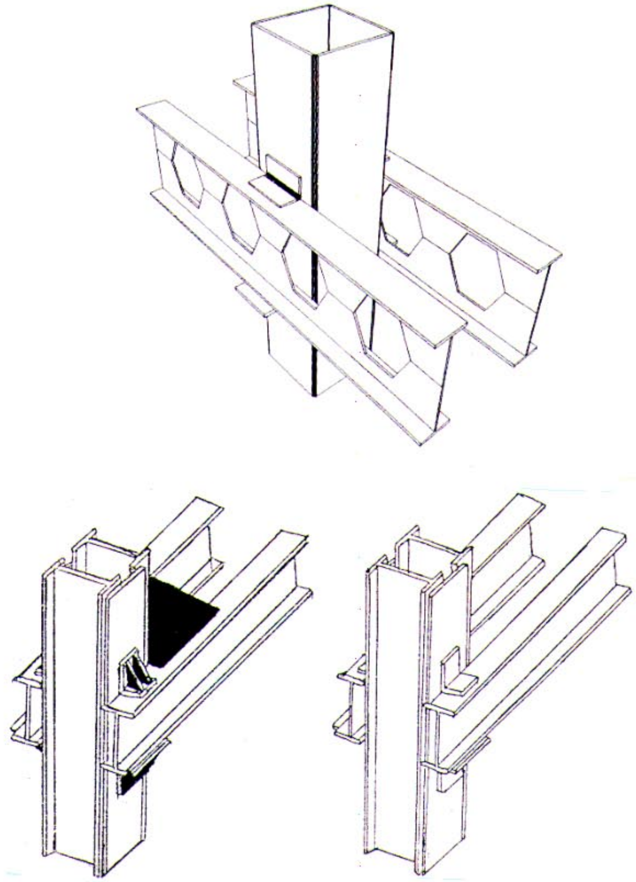

Fig. 2: Typical shape of a Khorjinee connection

dynamic behavior the typical steel structure a 1/2 scaled model of 4 storey steel structure with Khorjinee connection and jack-arch masonry floors has been designed, constructed and tested by a pair of harmonic force vibration exciters at International Institute of Earthquake Engineering and Seismology (IIEES) structural lab as shown in Fig. 1. For purpose of vulnerability analysis and proposing the retrofitting scheme, the model was constructed based on the common practice of Iranian steel workers. The structural response is being monitored and measured simultaneously by a 8 forced balance accelerometer and recorded by 8-channel digital acquisition system. In this paper the dynamic characteristic of the structure and its joint (frequencies, mode shapes, damping ratios) as well as the connection stiffness have been obtained. Finally a method with the ability of modeling the behavior of the Khorjinee connection has been presented.

KHORJINEE CONNECTION'S CHARACTERISTICS

Fig. 2 shows the detail of typical Khorjinee connection. In this connection contrary to common semi-rigid connections (Kishi *et al.*, 1987, Davison *et al.*, 1987) a pair of continuos beams, which cross several columns, connects to the sides of columns by means of angle sections. The beam and column are welded to the angle section. Sometimes for the purpose of supporting the gravity loads in the braced frame, one or two stiffeners will be added to the angle section under the beam (seat-angle). To improve the rigidity of the connection in case of no bracing, plates at different parts of connection as shown in Fig. 2 are added. This connection is widely used by contractors for following important reasons:

1, Its detail and deisgn is easier than other connections.
2, Due to continuity of the beams in this connection, the negative moment at supports decrease the positive moment at midspan and causes the reduction of beam size and the steel weight.
3, The installation of one piece beam is easier and faster which reduce the erection time and welding cost.

One of the major difficulties of khorjinee connections is that in the minor direction (the direction vertical to connections) is very difficult to improve the rigidity of the connection since the crossed beams connect to web of khorjinee beams. Thus in the weak direction of the frames the connections are considered as hinge and the bracing are used to resist seismic loads. However, in the khorjinee direction since the possibility of using the bracing is very little, the frame is taken as rigid. Also out of plane partial beam-to-column transfer of bending moment and early onset of failure in the angles are most likely the cause of failure under the lateral load. Therefore, studying the moment distribution in the beams and columns, the behavior of connections during earthquake as well as the structural stability is of great importance for purpose of strengthening of these structures.

ANALYTICAL METHOD

The common static and dynamic analysis of these types of structures is approximate since they can not be modeled by any of classical methods and any of the commercial computer programs. However engineers use following methods of analysis:

1, The structure is modeled as rigid frame by assuming that the connection's stiffness is infinitely large. The difficulties of this method is the large moment at columns which is unreal and causes error in beam's moment distribution.

2, The two ends of columns in this model is taken as hinge (in braced frames). In this method the flexural stiffness of columns and connection's stiffness is regarded as zero.

Eventhough these methods may be acceptable for static analysis of the structure under gravity loads, but in dynamic condition, the connection's stiffness greatly influences the natural periods of vibration of structure and consequently its dynamic response.

FORCED VIBRATION TESTS

For forced vibration test of the model, a pair of harmonic force exciters was utilized with capability of inducing translational and torsional dynamic force at the top floor of the structure. The exciters can be adjusted to produce maximum force $16.1f^2$ kgf, where f is the operating frequency that can vary from 1 to 20 Hz. The responses were measured by 8-channel on-line data acquisition system with force balance accelerometers. The 1/2 scaled model of the 4-storey structure based on Iranian Building Code load requirement and modeling technique requires 7.0 ton of added mass on each floor. Throughout this project in order to develop a new algorithm for system identification, the added mass were added at three different stage (1., 3. and 7.0 ton). Fig. 3 shows a sample of measured acceleration response spectrum at forth-storey. For removing noises from the recorded response, the cross correlation method is used. Three modes of structure with frequencies of 2.24, 7.94 and 16.1 Hz. can be easily seen in Fig. 3. The 4'th mode natural frequency is more than 20Hz. which could not be tested. Table 1 shows the result for various added mass on the floor.

The displacement spectrum of the first mode at various levels of the structure is shown in Fig.4. It can be seen that displacement of all stories reaches to its maximum amount at frequency near 1'st mode (2.24 Hz.). By use of this information and the phase angles, the vibration mode shapes have been obtained.

Table 1-Modal Natural Frequencies For Different Storey Added Masses

Added mass	0	1000	3000	7000
1'st Mode	2.24	1.99	1.34	1.06
2'nd Mode	7.94	6.44	4.52	3.45
3'rd Mode	16.1	12.16	8.57	6.28
4'th Mode	------	17.54	12.24	8.75

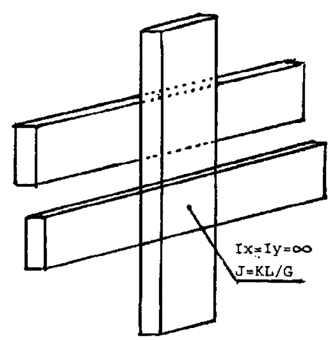

Fig.6: The mathematical model of beam-column connection

Table 2: Results of analytical and experimental frequencies and their error

Story Added mass (Kg.)	Connection stiffness (t-m/rad)	1'st mode			2'nd mode			3'rd mode			4'th mode		
		Ana.	Exp.	%err	Ana.	Exp.	%err.	Ana.	Exp.	%err.	Ana.	Exp.	%err.
0.	50.	2.24	2.24	0	8.31	7.94	4.6	18.2	16.1	13.0	29.5	--	--
1000.	150.	1.99	1.99	0.	6.89	6.44	7.0	14.0	12.2	14.9	21.6	17.5	23.0
3000.	80.	1.34	1.34	0.	4.71	4.52	4.2	9.78	8.57	14.1	15.4	12.2	25.6

THE STUDY OF DAMPING RATIOS

The damping ratio was calculated with different approach using the result of the free vibration test of the model structure as well as from measuring the free vibration part of the record of the forced vibration test. Fig. 5 shows the story displacements during the free vibration test. Using the Logarithmic decrement method, the average damping ratio becomes: ε=0.006. By other methods such as half-amplitude or half power method the damping ratio of the first mode of vibration is: ε=0.00647 that approximately is equal to the result of logarithmic decrement method. Using the bandwidth method the damping ratios of first three modes are 618%, .519%, and .509%. when there no added mass on each floors. As story added mass increases to 7.0ton, the damping ratio increases to 3.0%. It should be noted that these values are less than the actual damping ratio of the structures. This difference are due to the existence of infill walls in the frame of real structure which has not included in that stage of the test.

ANALYSIS OF THE MATHEMATICAL MODEL

The semi-rigid connections can be divided in two groups of continuous and discrete connections. To model the continuous semi-rigid, Khorjinee, connection a spring element at the connection of beam and column have been used. This element in the ETABS program is in the form of beam element and in the 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 Fig. 6.

The model structure with assumed connection stiffness has been analyzed. Fig. 7 shows the variation of modal frequencies versus the connection stiffness in the case of zero added mass. It can be seen that the modal frequency changes when connection stiffness are between 1. to 1000 t-m/rad and become insensitive as the stiffness becomes larger than 1000. t-m/rad. In other word, the connection with stiffness larger than 1000 t-m/rad can be considered as a rigid connection. Fig. 7 shows that for the first three modal frequencies of the structure the connection stiffnesses are 50., 150. and 80. t-m/rad, respectively. Using these connection stiffness and analyze the structure, we found that the calculated frequencies are different from the forced vibration test. These results are shown in Table 2. these differences are due to incomplete conformity of exact structure and computer model. Studying the result of Table 2, one can conclude a linear relation between Logarithm of connection's stiffness (K) and natural vibration frequency (ω) in different modes as:

$$\omega_i = A_i \log(K) + B_i \qquad \text{(i is related to different modes)} \qquad (1)$$

Where coefficients Ai and Bi are functions of average storey masses and equal to: $A_i = a_i M + b_i$; $B_i = c_i M + d_i$
Using (1) And the result of experimental test, the approximate value of connection's stiffness can be estimated. It must be noted that this problem is with the assumption that most of connections have the same shape.

NUMERICAL RESULTS

To study the dynamic characteristic of steel structures, seven different structures have been analyzed. Natural frequencies and various response quantities are given in Table 3 for different model of the connection (Rigid, Khorjinee and Hinged). The results show that in braced structures, the difference between stories' displacement in both Khorjinee and Hinge connections is 2 to 10 percent. However, in the unbraced structure the difference is approximately 55 percent that is very large. The midspan moment of side beam under gravity loads and the support moment of side beam under gravity or under earthquake loads for both type of Khorjini and rigid connection have been compared. It can be conclude that, the midspan moment at side beams decreases from 5% to 25% (depending on beam's length) in comparison with rigid connection. On the contrary, in this case support moment of beam under gravity and earthquake loads increase in 10 to 50 percent. Therefore, for design of side column of structure with Khorjinee connection, the connection should be analyzed exactly, and if is not possible, an extra moment beside the column axial force should be considered. The magnitude of the moment depends on beam span and number of its bays and loading types.

CONCLUSIONS

The result of this part of the study shows that in braced structure the earthquake has no considerable effect on connections and thus the Khorjinee connection can be assumed rigid. In unbraced steel frame, the effect of assuming the Khorjinee connections as rigid or hinge has significant effect on dynamic response of structure and thus the Khorjinee stiffness should be used in the model based on relation (1). In the case of gravity loads whether the bracing is exist or not, the midspan moment of side beams, support moment of side beams, side column's moment and also torsional moment of connections in the side columns must be studied and calculated exactly. If exact analysis Khorjinee connection is not done, one can use results of analysis of 6 examples discussed in this paper and design the beam, column, and connections with good safety factor.

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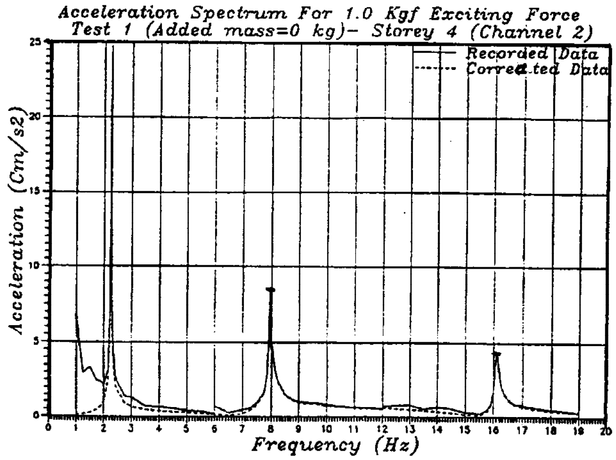

Fig. 3: Acceleration Response Spectrum for Recorded and Corrected Data

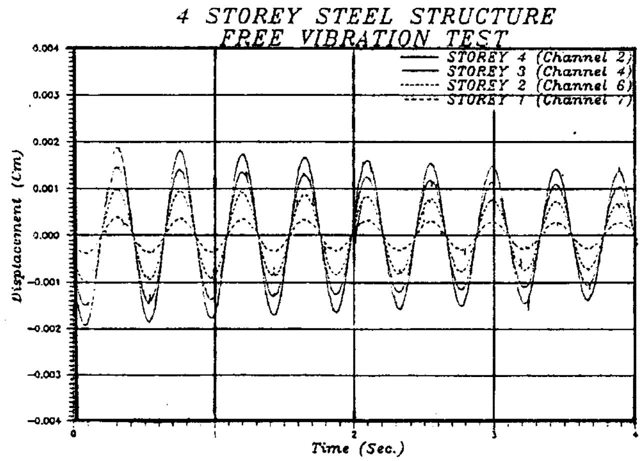

Fig. 5: Free Vibration Test Results

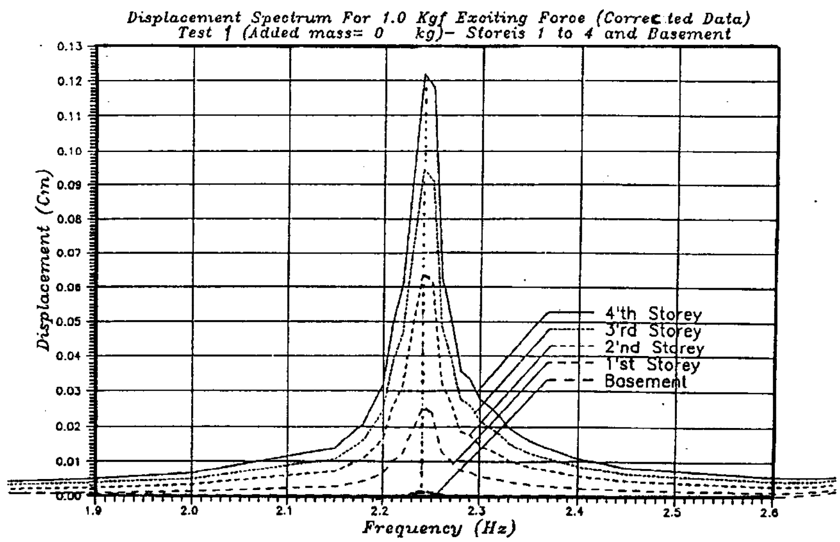

Fig.4: First Mode Displacement Spectrum

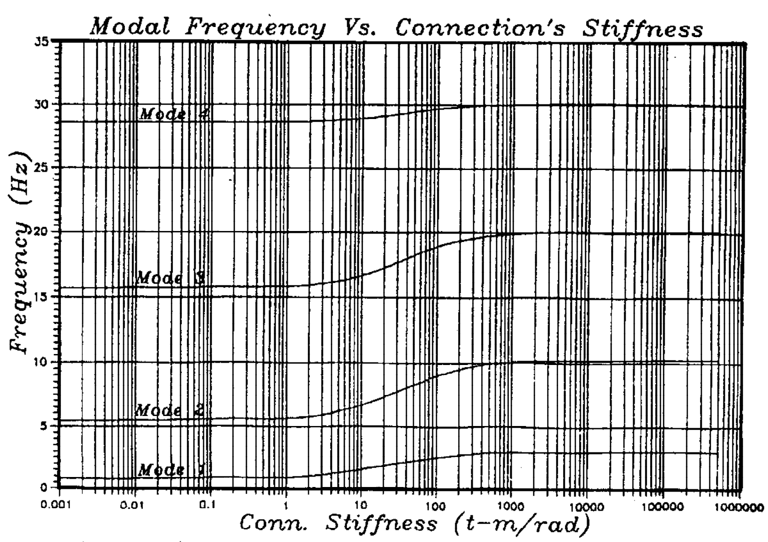

Fig. 7: Computer Analysis Result of Structure for Different Connection's Stiffness

Table.3-Results of static and dynamic analysis of 6 examples with Rigid, Khorjini, and Hinge connections

	E1 (unbraced)			E1 (braced)			E2			E3			E4			E5			E6		
	Rig.	Kho.	Hin.	Rig.	Kho.	Hin.	Rig.	Kho.	Hin.	Rig.	Kho.	Hin.	Rig.	Kho.	Hin.	Rig.	Kho.	Hin.	Rig.	Kho.	Hin.
1'st mode period	--	2.43	1.76	0.47	0.46	0.45	0.96	0.91	0.90	0.64	0.61	0.60	0.67	0.62	0.60	0.44	0.43	0.43	1.03	0.92	0.85
2'nd mode period	--	0.75	0.58	0.13	0.13	0.13	0.24	0.23	0.23	0.20	0.19	0.19	0.21	0.20	0.19	0.12	0.11	0.11	0.27	0.25	0.24
3'rd mode period	--	0.40	0.33	0.07	0.07	0.07	0.13	0.13	0.13	0.11	0.11	0.10	0.12	0.11	0.11	0.07	0.06	0.06	0.13	0.12	0.12
4'th mode period	--	0.25	0.23	0.05	0.05	0.05	0.10	0.09	0.09	0.09	0.08	0.08	0.10	0.09	0.08	0.05	0.05	0.05	0.09	0.09	0.08
Disp. of 4'th storey(mm)	--	56.6	36.8	6.90	6.60	6.50	17.5	16.2	15.9	9.90	9.10	8.90	10.3	9.30	8.80	6.10	5.80	5.60	19.5	16.6	15.1
Midspan mom. of beam in G.L.(t.m)	--	4.50	3.80	6.20	4.50	3.80	2.40	2.30	1.90	2.30	1.9	1.80	2.20	1.70	1.50	3.70	3.40	3.10	2.20	1.90	1.50
Support mom. of beam in G.L.(t.m)	--	3.50	5.10	0.00	3.50	5.10	0.00	2.90	3.30	0.00	1.4	1.80	0.00	1.00	1.0	0.00	2.10	3.00	0.00	1.10	1.60
Support mom. of beam in E.L.(t.m)	--	1.60	2.00	0.00	0.10	0.90	0.00	0.20	0.20	0.00	0.20	0.30	0.00	0.40	0.50	0.00	0.20	0.30	0.00	0.40	0.80
Side columns mom.in G.L.(t.m)	--	1.70	2.50	0.00	1.70	2.50	0.00	0.40	0.60	0.00	0.50	0.60	0.00	0.20	0.40	0.00	0.80	1.10	0.00	0.50	0.90
Side columns mom.in E.L.(t.m)	--	0.20	0.60	0.00	0.10	0.10	0.00	0.00	0.00	0.00	0.10	0.20	0.00	0.00	0.30	0.00	0.10	0.10	0.00	0.10	0.30
Torsional mom. in mid con. G.L.(t.m)	--	1.52	--	--	1.45	--	--	1.98	--	--	0.40	--	--	0.60	--	--	0.40	--	--	0.00	--
Torsional mom. in side con. G.L.(t.m)	--	2.89	--	--	2.88	--	--	2.86	--	--	1.40	--	--	1.00	--	--	2.10	--	--	1.10	--
Torsional mom. in mid con. E.L.(t.m)	--	2.37	--	--	0.20	--	--	0.20	--	--	0.30	--	--	0.60	--	--	0.20	--	--	0.60	--
Torsional mom. in side con. E.L.(t.m)	--	1.50	--	--	0.18	--	--	0.19	--	--	0.40	--	--	0.40	--	--	0.20	--	--	0.40	--