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EXPERIMENTAL STUDY AND NUMERICAL RESPONSE PREDICTION OF A STEEL MOMENT-FRAME WITH TAKING FATIGUE DAMAGE INTO ACCOUNT

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Abstract. *In this study, shaking-table test results of a steel moment-frame with end-plate bolted connections are presented. These experimental tests were carried out to understand the behavior of steel moment-frame structures subjected to dynamic and seismic loads. The purpose of these tests is also to study the changes in modal parameters due to the development of the elasto-plastic behavior and Low Cycle Fatigue LCF damage in bolted steel frame connections. The study presents a nonlinear numerical simulation of the steel moment-frame that was performed using a Fatigue Damage-Based Hysteretic FDBH model. The developed model is a degrading hysteretic model based on the LCF damage index. A comparison of the results obtained from numerical analysis and those of shaking table tests is presented. A reasonable agreement is observed, indicating a good simulation of the nonlinear behavior of the steel moment-frame by using the developed model.*

1 INTRODUCTION

In recent years, several attempts have been proposed in literature to take account of the stiffness degradation induced by the cumulative damage effects during Low Cycle Fatigue (LCF) deformations [1, 2, 3].

Ibarra *et al*. [1] presented the description, calibration and application of relatively simple hysteretic models. The modified models included most of the sources of deterioration. The hysteretic models included a post-capping softening branch, residual strength, and cyclic deterioration. The model could be employed to simulate the behavior of a bolted connection.

Lignos *et al*. [4] proposed a numerical model that was able to simulate complex deterioration phenomena and ultimately connection fractures due to LCF. The model was implemented in the open-source numerical simulation platform. The component model was calibrated against a large number of steel component tests that have been conducted over the past years around the world.

Saranik *et al*. [2, 5] developed a Fatigue Damage-Based Hysteretic (FDBH) model that allowed considering the stiffness degradation produced by the cumulative fatigue damage effects. The FDBH model was a hysteretic model based on LCF damage index. It can be applied to evaluate the performance of steel portal frames with bolted connections under dynamic loading.

Moreover, Saranik *et al*. [6] proposed an advanced approach to take into account the effects of nonlinear modes and frequencies by performing a nonlinear time history analysis. According to this approach, the nonlinear modes and frequencies were determined by an iterative procedure.

In this study, shaking-table test results of a steel moment-frame with end-plate bolted connections are presented. A steel portal frame was investigated under horizontal sinusoidal base excitations. These experimental tests were conducted to verify the validity of the FDBH model and to prove the efficacy of the nonlinear dynamic analysis techniques. The advanced approach proposed by Saranik *et al*. [2] was adopted to take into account the effects of nonlinear modes and frequencies by performing a nonlinear time history analysis. Consequently, this approach was used to evaluate the performance of steel frames under lateral loads applied by base excitation. A comparison of the results obtained from numerical analysis and those of shaking table tests is presented.

2 STIFFNESS DEGRADATION DUE TO FATIGUE DAMAGE

Several studies have indicated that bolted connections are vulnerable to the damage accumulation. Plastic hinges usually develop at beam-column connections [2, 3, 7]. The welds and the bolts can be largely affected by LCF which causes progressive and cumulative damage in the stiffness of bolted connections [3]. Korol *et al*. [7] found that the excessive yielding of the connection makes more prone to LCF and results in severe damage. Bolts can be loosen over time because of micro-macro slip in the bolt-nut and the assembled plates [8].

In recent years, different approaches have been developed on the use of a damage index to estimate the structural damage [1, 9]. They aim to clarify the different approach methodologies and to detail different proposed formulations. Several researchers have been performed in literature to take account of the stiffness degradation induced by the cumulative damage effects during LCF deformation [2, 3]. The experimental results, carried out by Tani *et al*. [3], showed that a damage variable D can be used to measure cumulative damage and provide a good estimate of the fatigue damage process, as well as to predict the fatigue life.

According to Tani *et al*. [3], a phenomenon of degradation in the stiffness of the bolted connection can be observed in the case of repeated loading. Tani *et al.* showed in a cyclic test the existence of the degradation phenomenon in the stiffness. They proposed to use the number of applied cycles n_i divided by the total number of cycles N_i, which corresponds to the fatigue damage according to Miner's rule [10], as damage parameters for the bolted connections. Then, Tani *et al.* constructed a model of connection stiffness degradation as a function of fatigue damage. Accordingly, the degradation of the connection stiffness k can be constructed on a normalized scale, as shown in Fig. 1, and an approximation model based on the mean curves of nine tests can then be written as:

$$\frac{k}{k_0} = f(D) \tag{1}$$

where k_0 is the initial connection stiffness for each fatigue test and $f(D)$ can be defined as:

$$f(D) = \begin{cases} 0.2exp(-3.5D) + 0.8 & for \;\; D < 0.9 \\ -2.4D + 3 & for \;\; 1 \geq D \geq 0.9 \end{cases} \tag{2}$$

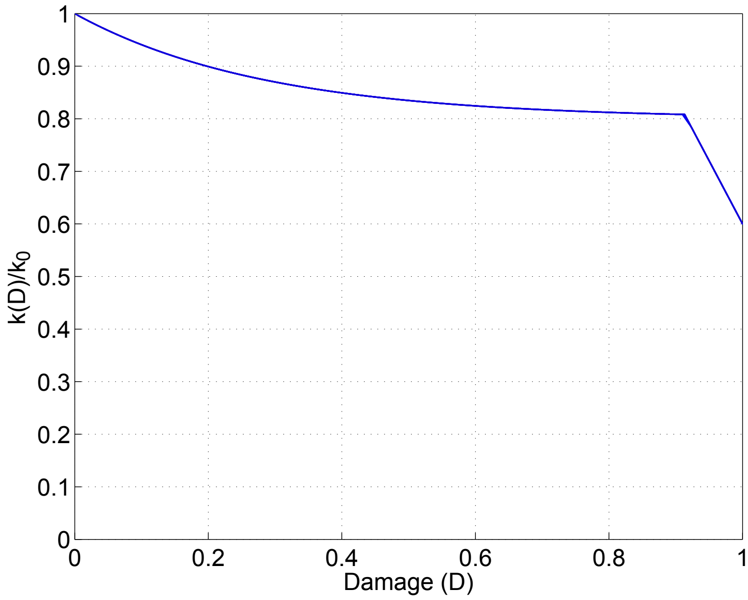


Figure 1: Degradation of connection stiffness as a function of the damage variable D.

The above results were used by Tani *et al*. [3] to performed fatigue analysis simulations with deterministic and random excitation.

Moreover, many experimental approaches confirm the presence of changes in modal parameters such as natural frequencies because of the damaged elements of the structure. The presence of damage or deterioration in a structure causes changes in the natural frequencies of the structure. Due to the fatigue damage in the bolted connection, the natural frequency can decrease [2, 3].

By using a standard Finite Element (FE) model, the evolution of the natural frequency can be evaluated based on the degradation of the connection stiffness as a function of fatigue damage on the basis of Miner's rule (*cf.* Eq. 2). Fig. 2 shows an example for the evolution of the natural frequency with respect to the number of cycles for a bolted connection. The results were obtained from numerical simulation carried according the numerical method proposed by Tani *et al*. [3] to integrate the connection stiffness degradation due to fatigue loading.

These results are adopted by Saranik *et al*. [2] and incorporated into a simulation of a complex portal frame structure to determine the structure's lifetime, by assuming that fatigue damage occurs only at the beam−column connections. These proposed developments will be presented in the next sections.

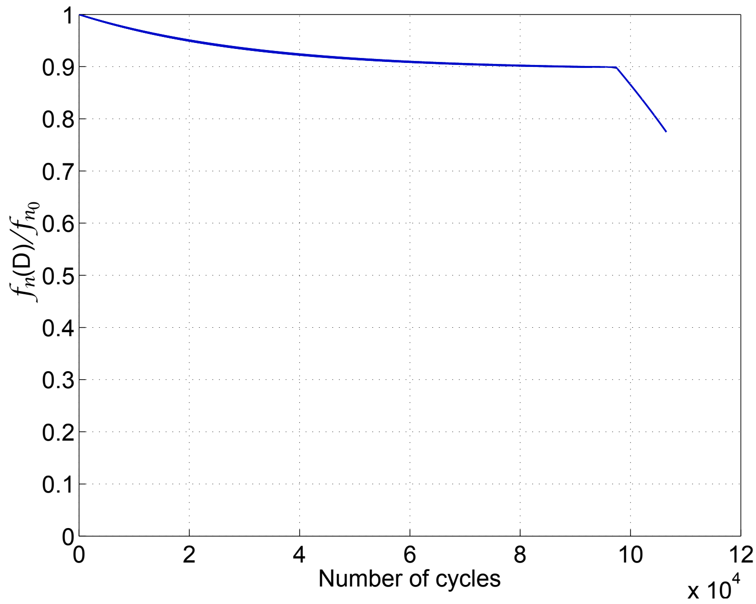


Figure 2: Evolution of the natural frequency as a function of number of cycles for a bolted connection.

3 DEVELOPED MODEL

Saranik *et al*. [2] modified Richard-Abbott model [11] to include the stiffness degradation of the connection produced by the cumulative phenomenon of LCF. The developed model is named Fatigue Damage-Based Hysteretic FDBH. This degrading hysteretic model is based on a LCF damage index D_n. In this model, the initial stiffness k_0 is modified with the factor $(1 - D_n)$ that takes account of the effect of cumulative fatigue (see Fig. 3).

In this way, the connection loses part of its stiffness in each cycle of excitation applied due to the cumulative phenomenon of LCF. The following equation presents FDBH model for the moment-rotation relationship:

$$M^* = M_a - \frac{(k_0.(1-D_n)-k_p).(\phi_a-\phi)}{\left(1+\left|\frac{(k_0.(1-D_n)-k_p).(\phi_a-\phi)}{2M_0}\right|^{\gamma}\right)^{\frac{1}{\gamma}}} - k_p.(\phi_a-\phi) \quad (3)$$

where M^* is the degraded connection moment, ϕ is the relative rotation between the connecting elements, M_0 is the reference moment, and γ is the curve shape parameter. D_n is the fatigue damage index and k_p is the final plastic stiffness. (M_a, ϕ_a) is the load reversal point and (M_b, ϕ_b) is the next load reversal point as shown in Fig. 3. Saranik *et al* also proposed a value of the plastic stiffness k_p = 0 which can simplify the model.

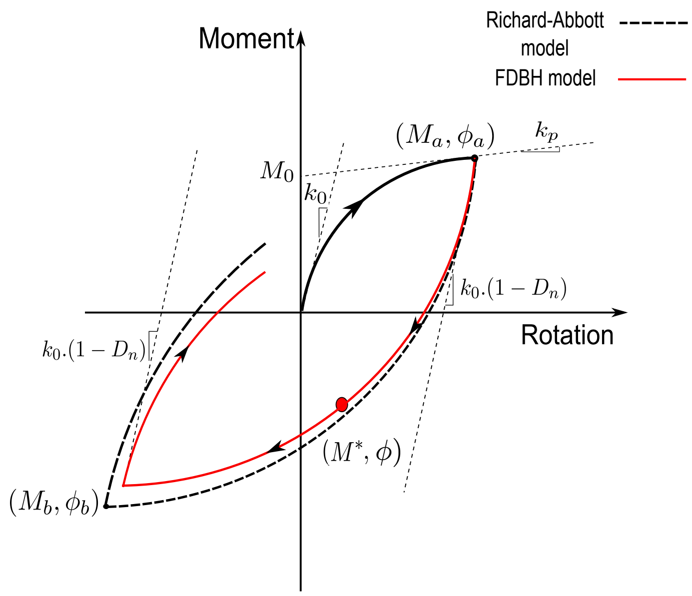


Figure 3: Fatigue Damage-Based Hysteretic model of bolted connection developed by Saranik *et al*. [2].

The tangent stiffness of the connection element can be written as:

$$k^* = \frac{(k_0.(1-D_n)-k_p)}{\left(1+\left|\frac{(k_0.(1-D_n)-k_p).(\phi_a-\phi)}{2M_0}\right|^{\gamma}\right)^{\frac{\gamma+1}{\gamma}}} + k_p \tag{4}$$

where k^* is the degraded secant stiffness.

In this way, the secant stiffness in Eq. (4) can be modified and it is possible to combine the two indices by the equation:

$$D_p = 1 - \frac{k^*}{k_0} \tag{5}$$

where D_p is the plastic damage index. It can be observed that D_p depends on the tangent stiffness k^* of the connection and the LCF damage index D_n.

The FDBH model of Saranik *et al*. [2] uses recent concepts in the structural damage evaluation and it can allow taking account of the stiffness degradation produced by the cumulative fatigue damage effects. Thereby, the FDBH model is an efficient model to simulate inelastic response of structure under dynamic loading by combined damage indices.

To evaluate fatigue damage in structural components under arbitrary loading histories, Miner's rule is commonly employed [3, 10]. Though many models of damage have been proposed, Miner's rule is still widely used in the engineering field [3]. According to this rule, applying a cycle n_i times with a stress or strain amplitude which corresponds to a lifetime of N_i cycles is equivalent to consuming a portion n_i/N_i of the whole lifetime. This rule implies that rupture occurs when 100% of the lifetime is consumed. It also describes the phenomena of the cumulative linearity of fatigue damage if another application stress or strain is employed.

For simplicity, some global parameter such as plastic rotation can be used rather than strains or stress for applying Miner's rule [12]. In the case of connection subjected to many cycles of rotation, Miner's rule is expressed by the following equation:

$$D_n = \sum_i \frac{n_i}{N_i} \tag{6}$$

where n_i is the number of applied cycles for a given rotation level i and N_i is the number of cycles to failure for a rotation level i.

Moreover, to calculate the number of cycles to failure N, an analogous model based on the plastic connection rotation must be utilised therein. The information provided by the Rotation-Number of cycles curve is mainly applied by engineers for the prediction of the lifetime and resistance of structures under repeated loading. A useful method of describing the LCF life for a bolted connection is expressed in literature [2, 13]. Thus using the well known Manson-Coffin relationship [14], the plastic rotation may be related to the number of cycles to failure N by the following equation:

$$N = c.(\Delta\phi_p)^{-b} \tag{7}$$

where ϕ_p is the plastic rotation of the connection. c and b are the parameters of fatigue which depend on both the typology and the mechanical properties of the considered steel element. From experimental fatigue results performed by Saranik *et al*. [2], the fatigue parameters of Manson-Coffin are (c = 22 $\times$ 10^{-5}) and (b = 3). These parameters are used to analyse steel portal frames in this study. Manson-Coffin relation describes linearly this function between the applied rotation and the number of cycles to rupture on a double-logarithmic scale.

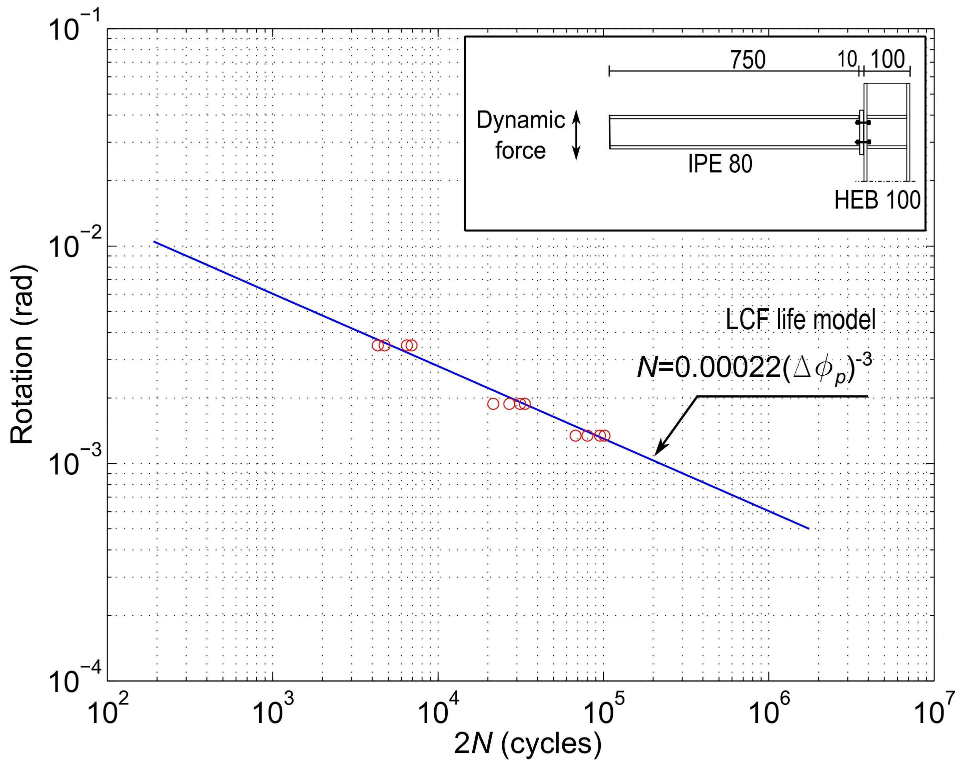
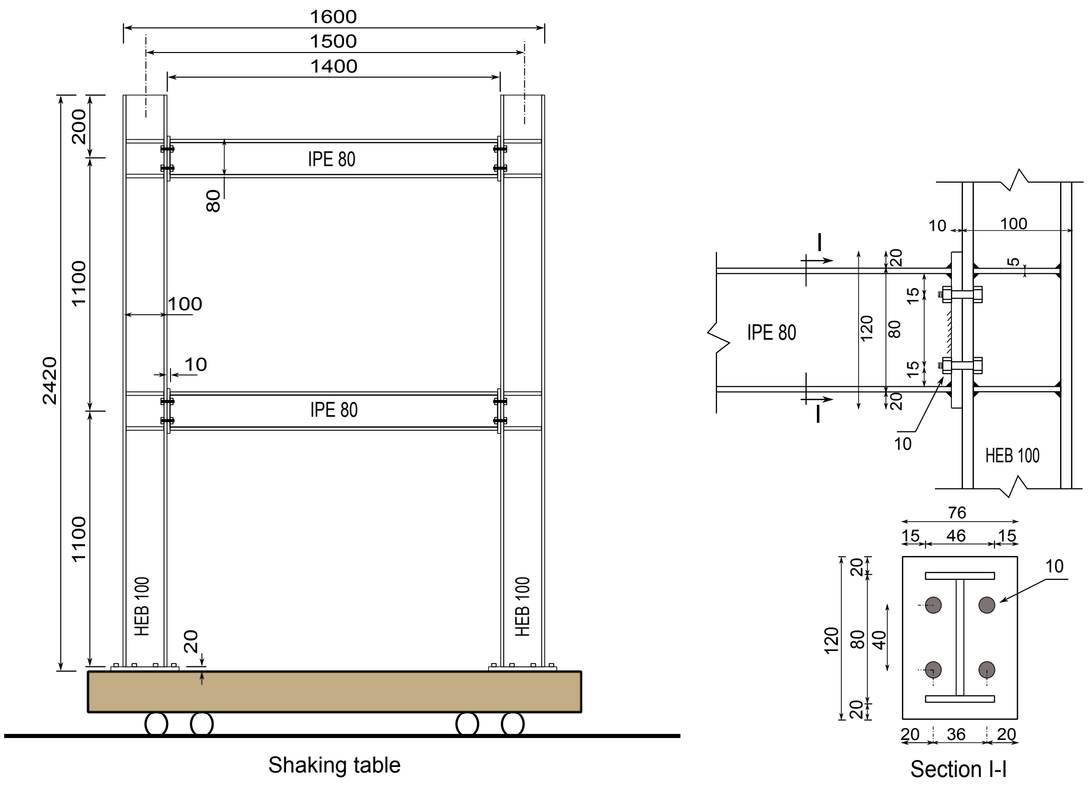


Figure 4: LCF life prediction model proposed by Saranik *et al*. [2].

4 TEST SPECIMENS AND EXPERIMENTAL PROCEDURES

A two story frame shown in Fig. 5(a) is used to carry out a series of shaking table tests. The test frame consists of elastic beam and column elements, joined together by bolted connections as shown in Fig. 5(b). The frame was constructed at the Structural Dynamics Laboratory of Ecole Centrale de Lyon.

The beam and column sections are chosen from Standard European Steel profiles IPE 80 and HEB 100, respectively. The nominal values of yield strength f_y and the ultimate values of tensile strength f_u of the I sections are respectively 260 MPa and 450 MPa. The steel members of the frame have a 210 GPa elasticity modulus.



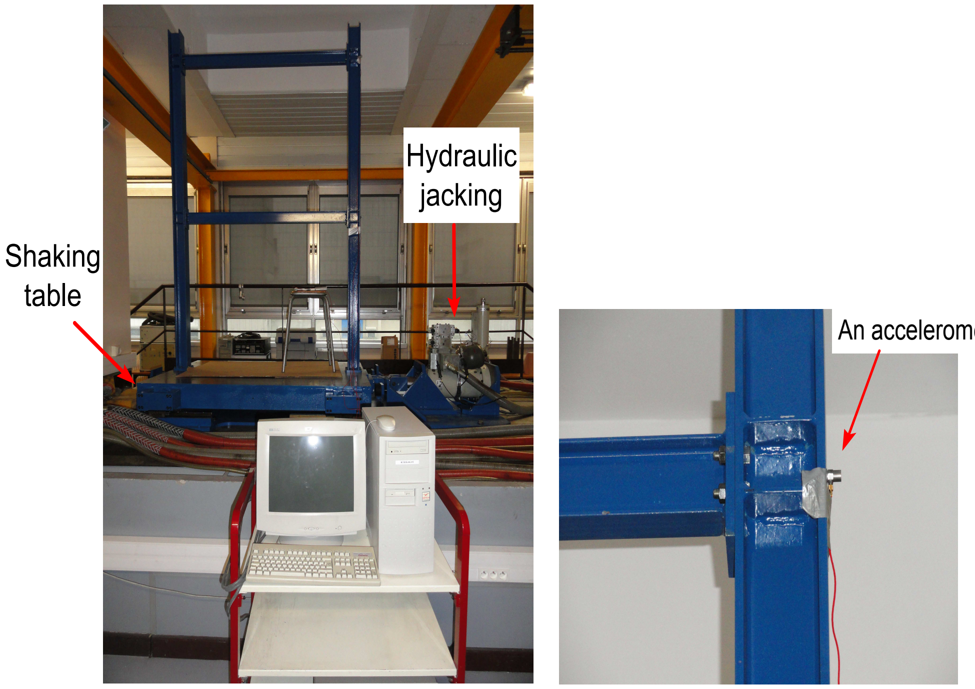
(a) (b)

Figure 5: Steel portal frame test set-up details (in mm): (a) details of the portal frame; (b) details of the bolted connection.

Four standard bolts (M10-1.25 × 35 mm Grade 6.8 DIN 975 carbon steel) are employed for each connection of the beam. The general layout of the end plate is shown in Fig. 5(b), with thickness (10 mm) and dimensions (120 mm × 76 mm) of the plate detailed. The experimental setup, shown in Fig. 6(a), was used during shaking table tests. The shaking table, shown in Fig. 6(a), was derived by a hydraulic jacking system that has the capacity to apply various excitation frequencies between 0.1 and 30 Hz and various displacements between 0.1 mm and 30 mm. Before the formal shaking table tests were performed, a pre-test was conducted under sinusoidal loads to evaluate the capacity, the characteristic of the shaking table and the measuring equipment. The command displacement given in the computer and the real displacement of the shaking table were compared in each test.

A horizontal sinusoidal shaking at a frequency of 15 Hz was applied in this experiment. The amplitudes of the horizontal sinusoidal excitation were 3, 5, 6 and 10 mm. During each test, the data were acquired and saved using a data acquisition system.

The test assembly was instrumented with two accelerometers to measure the specimen's response. The accelerometers, with sensitivity 100 mV/g, were mounted on the beam using super glue and duct tape to collect the horizontal displacement responses of the first and second story (see Fig. 6(b)). The accelerometers were connected to a device for measuring the displacements



(a) (b)

Figure 6: Test arrangement: (a) the portal frame; (b) the tested bolted connection and an accelerometer.

and to a data acquisition system to record the measured data.

Moreover, a hammer-shock was utilized to determine the natural frequency of the frame. Two mini accelerometers PCB with 1.03 mV/(m/s^2) sensitivity were installed to record the signal impact. The accelerometers were placed successively to both stories, oriented in the horizontal direction.

A torque wrench was used to tighten the 4 bolts for each bolted connection to the same level for all tests and to ensure consistency in the boundary conditions. The bolts were tightened to the required tightening torque 39 N.m.

Each test was started with the assembly of the frame model. Before testing, the two columns were positioned and fastened to the shaking table by making use of twenty-four M10 high-strength bolts conforming to DIN 975. Thereafter, the two beams were assembled to the columns with bolted connections by using four M10 bolts for each connection. Column base connections were made as rigid as possible. The frame model was fixed in displacement and rotation at the base level, free to move at the first and second story. For all tests, the bolts of these connections were tightened, applying a torque wrench, to the required torque value. In this way, the tested frame could be placed in position in the shaking table as shown in Fig. 5(a).

During a shaking table test, a loosening phenomenon reduces the clamping force of the bolts due to repeated excitation. Subsequently the cyclic loading can cause sliding in the assembled elements, which can change the stress distribution and gradually produce fatigue damage. The frame model was tested until the ultimate connection failure occurred. Subsequently, the time at failure was recorded and the level of displacement was used for this test as well. These experimental results were compared with numerical results. After each shaking table test, the bolts of the four connections must be changed and tightened to the required torque value.

Before running shaking table tests, a hammer-shock test on the model was conducted to determine the dynamic characteristics of the portal frame model such as natural frequencies. The hammer-shock test was also repeated before and after each shaking table test for detecting any changes in natural frequencies.

5 FINITE ELEMENT NUMERICAL SIMULATION

The damage in the structure could be identified as a change in FE stiffness matrices of the different beam elements of the structure. In this study, the FE model of the structure was a two-dimensional model developed by the Structural Dynamics Toolbox (SDT) under MATLAB 7.6 (R2008a) [2, 6]. The SDT, one of the widely used commercial FE analysis products that integrate OpenFEM as FE calculation source, was used to analyse the test frame.

The equation of motion for a damped structure with N degrees-of-freedom (dof) is given as follows:

$$\mathbf{M}\ddot{\mathbf{q}} + \bar{\mathbf{C}}\dot{\mathbf{q}} + \bar{\mathbf{K}}\mathbf{q} = \mathbf{F}(t) \tag{8}$$

where $\mathbf{M}$, $\bar{\mathbf{C}}$ and $\bar{\mathbf{K}}$ are respectively the mass, nonlinear damping and nonlinear stiffness matrix of the structure. $\mathbf{F}(t)$ is the applied forces vector and $\mathbf{q}$ is the relative response of system in normal coordinate. The stiffness matrix is nonlinear due to the nonlinearity of the beam-column connections and depends on the response of system and the nonlinear normal modes.

Rayleigh damping can be used to represent damping in the structure. It can be written as:

$$\bar{\mathbf{C}} = \alpha.\mathbf{M} + \beta.\bar{\mathbf{K}} \tag{9}$$

where α and β are the proportional damping factors and they are computed based on first two frequencies of the frame structure ω_1 and ω_2 as follows:

$$\alpha = \frac{2\xi\omega_1\omega_2}{\omega_1 + \omega_2}; \beta = \frac{2\xi}{\omega_1 + \omega_2} \tag{10}$$

where ξ is the damping ratio and a value of 3% is adopted for the frames analysed in the study.

The elements in the steel frame can be modeled as Euler-Bernoulli beams and each element beam or column is discretized into four finite elements. The matrix $\bar{\mathbf{K}}$ can be assembled by the stiffness matrices of each beam and column element in the system with the following equation:

$$\bar{\mathbf{K}} = \sum_{e=1}^{n_e} \bar{\mathbf{k}}_e \tag{11}$$

where n_e is the total number of elements in the system and members are generically identified by index e.

The nonlinear matrix for an element can be calculated using the correction matrix with the following equation [2, 15]:

$$\bar{\mathbf{k}}_e = \mathbf{k}_e\mathbf{Cr}_e \tag{12}$$

where $\mathbf{k}_e$ is the standard elastic stiffness matrix for the element. The correction matrix $\mathbf{Cr}_e$ is given as follows:

$$\mathbf{Cr}_e = \sum_q \sum_s c_{qs_e} : c_{qs_e} = f(D_{p_{e,l}}, D_{p_{e,r}}, L) \tag{13}$$

where $D_{p_{e,l}}$ and $D_{p_{e,r}}$ are the plastic damage indices of the connections (left l and right r, respectively) for the considered element. L is the length of this element. The plastic damages indices depend on LCF damage indices $D_{ne,l}$ and $D_{ne,r}$ for the considered element e.

According to Saranik *et al*. [2], eigenvalues and eigenvectors for a nonlinear system cannot be obtained by solving the standard eigenvalue problem. As the solution of a nonlinear system relies heavily on the amplitude of excitation, the frequencies and normal modes depend on the nonlinear modal amplitude. The introduction of the notion of nonlinear modes permits an extension of the method of linear modal synthesis to nonlinear cases in order to obtain the dynamical response of nonlinear multi-degree-of freedom systems.

The nonlinear normal modes and nonlinear frequencies can be calculated by an iterative procedure [2, 16]. In this paper, a procedure which is based on the method of equivalent linearisation was adopted. Considering the FE methods, the nonlinear modal problem can be written as following:

$$[\bar{\mathbf{K}}(\eta_{p_i}, \bar{\boldsymbol{\varphi}}_{p_i}(\eta_{p_i})) - \bar{\omega}^2_{p_i}(\eta_{p_i})\mathbf{M}]\bar{\boldsymbol{\varphi}}_{p_i}(\eta_{p_i}) = 0 \tag{14}$$

where η_{p_i}, $\bar{\omega}_{p_i}(\eta_{p_i})$ and $\bar{\boldsymbol{\varphi}}_{p_i}(\eta_{p_i})$ are respectively the structural response of the ith mode in modal coordinate, the nonlinear frequency and the nonlinear normal mode i.

For each time step of calculation, the stiffness and damping matrices of the system can be obtained that may transform the nonlinear system into an equivalent linear system. The frequencies and nonlinear normal modes can then be calculated using a standard solution of the eigenvalue problem given by Eq. (14). A set of N nonlinear modes and frequencies can be obtained according to their modal amplitudes. For this purpose, the response of nonlinear system in normal coordinate can be calculated efficiently by superposition of modal response as follows:

$$\mathbf{q}(t) = \bar{\boldsymbol{\varphi}}_p \boldsymbol{\eta}_p(t) \tag{15}$$

The response of system $\mathbf{q}(t)$ depends directly on the nonlinear modes matrix $\bar{\boldsymbol{\varphi}}_p$. This method is called nonlinear modal synthesis method.

6 NUMERICAL AND EXPERIMENTAL RESULTS

Experimental and numerical investigations were carried out on the two-story steel portal model for different base excitation displacements. This section presents the main observations obtained from the response of the test specimen during the sinusoidal displacement shaking. The shaking table tests were performed for four magnitudes of displacement excitations. The levels of displacements were 3, 5, 6 and 10 mm. The applied excitation frequency of all tests was fixed at 15 Hz. The results of the tests T3 and T4 will be discussed in detail in this section.

The simulated and measured story displacements along the height of the frame model are shown in Table 1.

Test	Δ Required (mm)	Δ Applied (mm)	Test duration (sec)	Time of measurement (t) (sec)	q_1 Exp (mm)	q_1 Num (mm)	q_2 Exp (mm)	q_2 Num (mm)	Failure modes
T3	5	2.32	300	10	2.8	2.3	5.9	5.5	Loosening of bolt-nut
				150	3.6	2.9	7.6	7	+
				260	11	9.6	25.5	24	Failure in bolts
T4	5	2.32	300	10	2.5	2.3	5.4	5.5	Loosening of bolt-nut
				150	3.4	2.9	7.4	7	+
				260	9	9.6	21.8	24	Failure in bolts

Table 1: Comparison of story displacements of the portal frame model.

In this table, q_1 and q_2 are the maximum relative story displacements of the frame. Furthermore, a comparison of maximum relative displacements between experimental and analytical results is shown in Table 1. The different failure modes resulting from these tests are also presented.

Under base excitation of $\pm$ 5 mm magnitude, the bolted connections were subjected to important damage after the tests T3 and T4. Based on the failure mechanism of bolted connection, the initial stiffness and the ultimate moment calculated are k_0 = 5.6 $\times$ 10^5 N.m/rad and M_u = 2.9 $\times$ 10^3 N.m, respectively. The stiffness values of connections were gradually degraded from an initial value k_0 to a finale value near to 0 as shown in Fig. 7. During tests T3 and T4, the location of the resonance point was observed at t = 260 sec. The displacements were amplified due to the damage of the connections at the 1st and the 2nd story. The damages occurred on the connections of both beams.

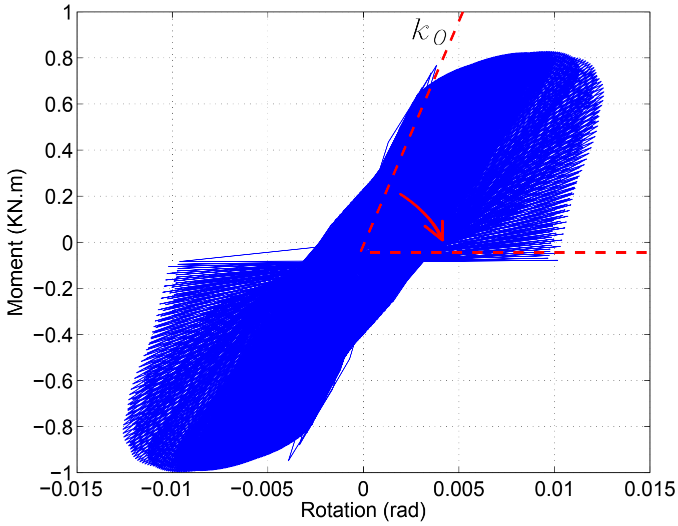


Figure 7: Numerical results corresponding to T3 and T4 : $M - \phi$ curve of the connection 1 at the first story.

Numerically from Fig. 8, the displacement responses of the frame increased gradually until the time t = 270 sec where the resonance may happen. Table 1 shows the comparison of the numerical and the experimental relative displacement responses under $\pm$ 5 mm required displacement excitation and at t = 10, 150 and 260 sec. Under this displacement, the numerical displacement responses q_1 and q_2 at the resonance point (t = 270 sec) were 9.6 and 24 mm, respectively. For test T3, the experimental displacement responses q_1 and q_2 were 11 and 25.5 mm. The results of the comparison between analytical and experimental responses indicated that the errors were 12.7% and 5.9%, respectively. Similarly, the displacement responses for test T4 were 9 and 21.8 mm at the resonance point, while the errors were 6.7% and 10%, respectively.

Fig. 9(a) and Fig. 9(b) display the LCF damage indices D_{n_1} and D_{n_2} and the plastic damage indices D_{p_1} and D_{p_2} for connections 1 and 2. The connection 1 was totally damaged at t = 275 sec after the resonance. At the same time, the index reached a value D_{n_2} = 66%. The plastic damage indices had the same values because the LCF damage was dominant. These results confirmed the experimental observation, because the damage in the 1st story was more than the 2nd story.

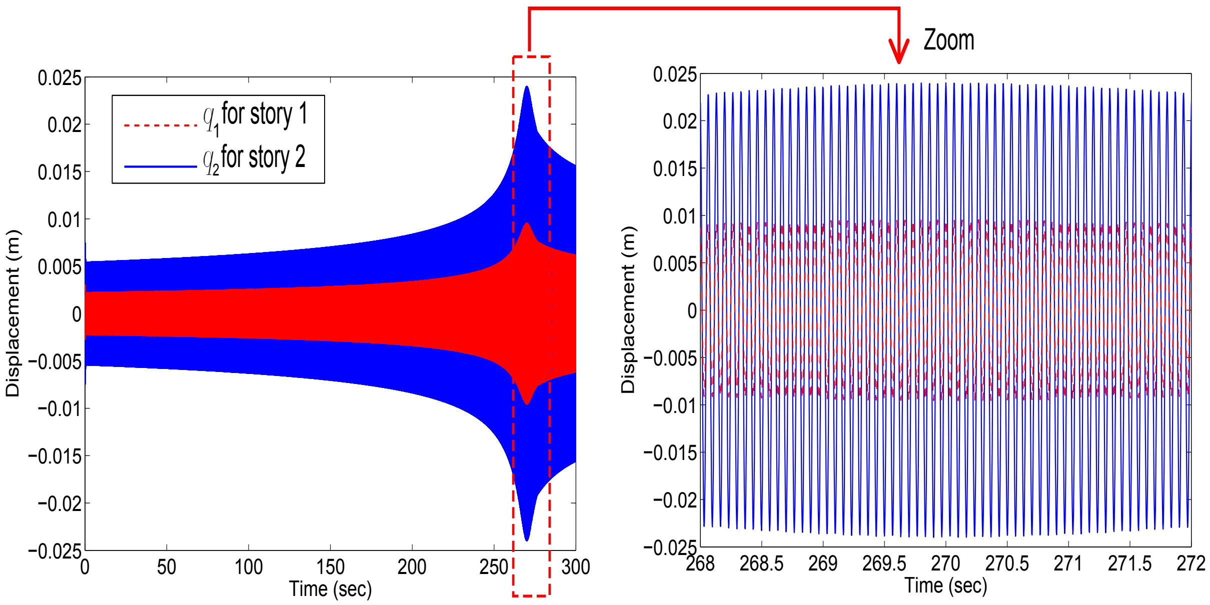


Figure 8: Numerical displacement responses q_1 and q_2 corresponding to tests T3 and T4.

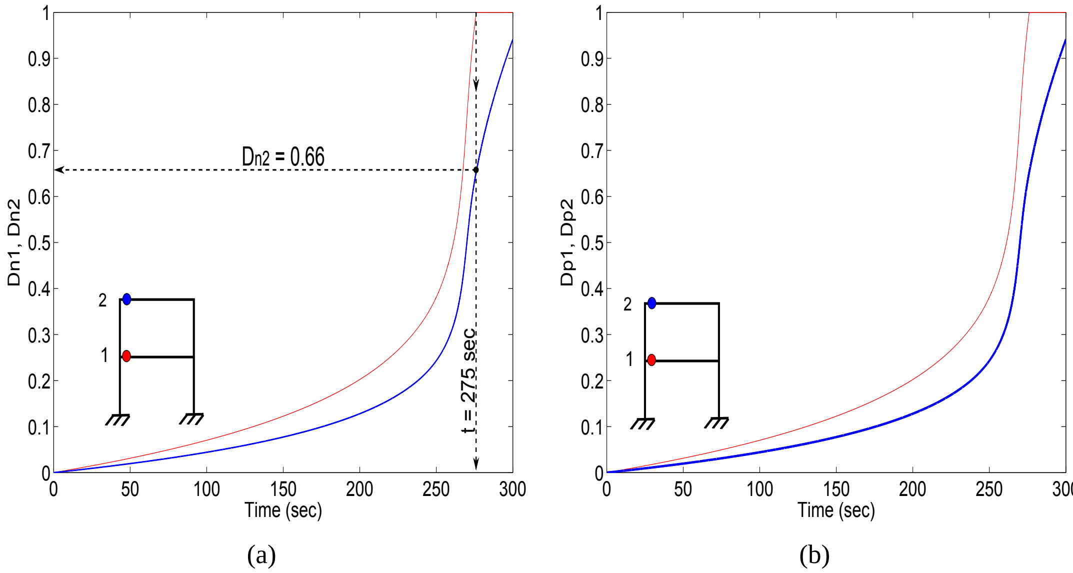


Figure 9: Numerical results corresponding to T3 and T4: (a) LCF damage indices of connections; (b) plastic damage indices of connections.

Moreover, the natural frequencies of the frame model were attained experimentally by hammer-shock tests and numerically by using the numerical simulation based on FE method instructed in MATLAB. Before the tests T3 and T4, the experimentally measured value of the first natural frequency f_{n1} was 18 Hz and the numerical value of f_{n1} was 18.5 Hz.

The loss of the first natural frequency was clearly observed in tests T3 and T4. According the numerical analyses, the natural frequency f_{n1} decreased gradually from initial value 18.5 Hz to a final value 12.55 Hz. Experimentally, the final value of f_{n1} was found to be 14 Hz in this case and by comparing with the numerical value, the error was 10.4%. Fig. 10, obtained from numerical analysis, presents f_{n1} versus the time and it indicates the resonance point at t = 270 sec where f_{n1} can have a value of 15 Hz. After this point, the loss of frequency was accelerated.

Another comparison can be done by determining the value of f_{n1} from the numerical curve (see Fig. 10) at t = 260 sec which presents the resonance point experimentally located. It is possible to say that the frame was very close to the resonance point with f_{n1} = 15.6 Hz.

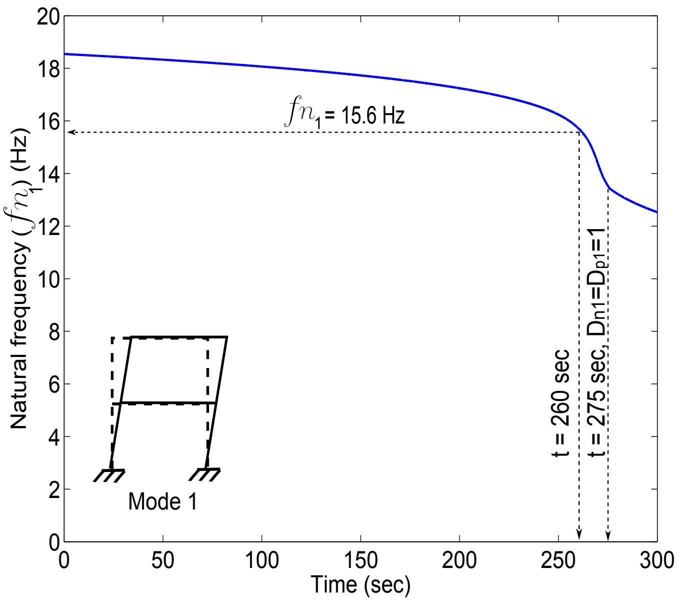


Figure 10: Numerical results corresponding to T3 and T4: frequency response $f_{n1}(t)$.

Figs. 11(a), (b), (c) and (d) display the failure modes of each bolted connection after test T3. A self-loosening and a backing-off of the nut were observed in connection bolts (see mark A). In addition, a failure by fatigue cracks due to repeated excitation was identified at a number of bolts. In these figures, arrows indicate the damaged bolts and it can be observed that the damage in bolts were much more severe in the first story.

Fig. 12 displays the measured frequency responses of the frame model after shaking table tests. The frequency responses indicated that the natural frequency values of a portal frame decrease as the damage increases. The amplitude (acceleration/force) increased in tests T3 to T5 because of the increase of damage and the same can be observed in test T6 (see Fig. 12). This proves that the modelling of damage due to LCF is important in order to predict the failure of a portal frame subjected to dynamic excitations.

From numerical and experimental results, the proposed FE method based on damage analyses can be employed to evaluate the damage in bolted connections and to estimate changes in dynamic properties. FBDH model integrated in FE method seems to provide reasonable estimates for the horizontal story displacements of the portal frame. This model can describe the accumulation of LCF damage and the damage induced by cyclic plasticity. Consequently, FDBH model can be used for nonlinear dynamic analysis of portal frame under long or short duration dynamic loading, for high or low amplitude of deformations. The model and the proposed approach can consider explicitly the main aspect of damage occurred in portal frames during dynamic excitation.

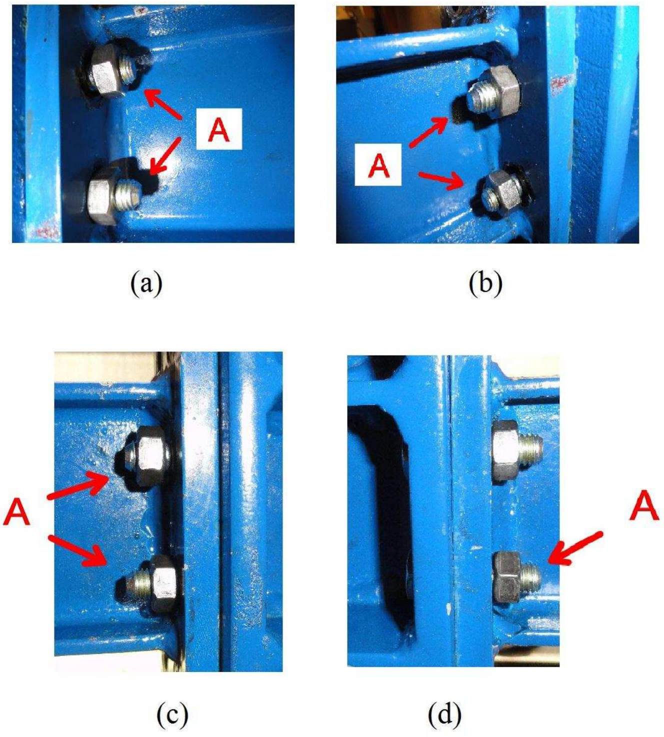


Figure 11: Damages in connections corresponding to T3 and T4: (a) connection 1 in the 1st story (side 1); (b) connection 1 (side 2); (c) connection 2 in the 2nd story (side 1); (d) connection 2 (side 2).

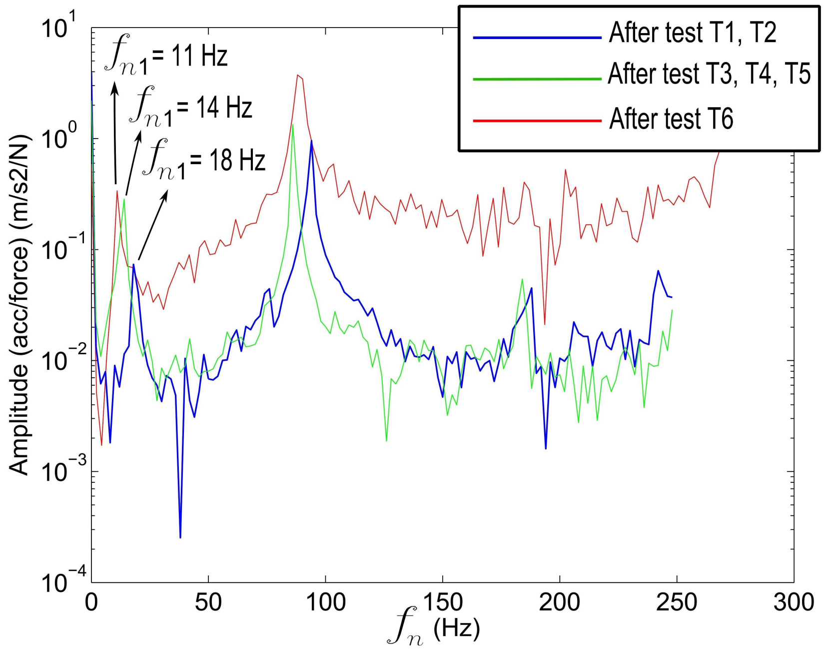


Figure 12: Comparison of experimental frequency responses.

7 CONCLUSIONS

In this study, a two story steel portal frame with bolted connections was investigated under horizontal sinusoidal base excitations. The results from shaking table tests were presented. The overall behavior of the structure was considered by comparing the numerical analysis and the experimental shaking table test results. The nonlinear numerical analyses of the frame were performed based on FDBH model. The study also proposed a contribution to the solution of nonlinear portal frame structures subjected to dynamic excitation by the method of nonlinear modal synthesis. This method involves a considerable simplification in the context of the analysis of nonlinear portal frame structures.

The results revealed a very good correlation between experimental and numerical displacement time history. The difference of the simulated response compared to the experimental results was attributed to the accuracy of measurements. Another reason for the observed differences is that there was uncertainty in bolted connection related to the variation in mechanical properties.

It is possible to recall that the LCF degradation of the initial stiffness of bolted connection was considered linear in the FDBH model using a factor (1-D_n). Therefore, an improvement could be made to improve the numerical results by adopting an experimental law.

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