

System identification of steel framed structures with semi-rigid connections

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(Received April 6, 2009, Accepted November 12, 2009)

Abstract. A novel system identification and structural health assessment procedure of steel framed structures with semi-rigid connections is presented in this paper. It is capable of detecting damages at the local element level under normal operating conditions; i.e., serviceability limit state. The procedure is a linear time-domain system identification technique in which the structure responses are required, whereas the dynamic excitation force is not required to identify the structural parameters. The procedure tracks changes in the stiffness properties of all the elements in a structure. It can identify damage-free and damaged structural elements very accurately when excited by different types of dynamic loadings. The method is elaborated with the help of several numerical examples. The results indicate that the proposed algorithm identified the structures correctly and detected the pre-imposed damages in the frames when excited by earthquake, impact, and harmonic loadings. The algorithm can potentially be used for structural health assessment and monitoring of existing structures with minimum disruption of operations. Since the procedure requires only a few time points of response information, it is expected to be economic and efficient.

Keywords: structural health assessment; semi-rigid connections; system identification; unknown dynamic force.

1. Introduction

The current state of practice in seismic design such as, UBC-97 (1997), Eurocode 8 (1998), IBC2006 (2000), and AASHTO (1998), infers that preventing collapse and loss of life is its main objective. However, economic losses due to seismic hazard are extensive and have never been addressed in current codes. Therefore, new design methodologies are emerging to account for design objectives other than loss of life, one such method is performance-based seismic design (PBSD). Primarily, PBSD is to design a structure to achieve preselected performance objectives in correspondence to certain seismic hazard levels. PBSD is not limited to design of new buildings, rather can be used to evaluate existing structures and/or retrofit, to reliable performance objectives. The decision of whether the structure is in need for rehabilitation or not after a major loading event, such as major earthquake, requires information about the health of the structure afterwards. Existing structures can also deteriorate as they age due to other dynamic loadings such as, wind gusts,

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explosions and machine excitations, which all can cause stiffness and/or strength degradation to a certain level. Therefore, health monitoring of existing structures to identify the state of the structure and to detect the damage when it occurs has recently become an important and challenging issue to the engineering profession. Extensive review of the literature on health monitoring can be found in (Doebeling *et al.* 1996, Housner *et al.* 1997).

The state of the structural system can be identified using a system identification (SI) technique. The available SI techniques can be broadly divided into two categories: frequency domain and time domain.

In the frequency domain approaches (Vestroni and Capecchi 2000, Lam *et al.* 2004, Kanwar *et al.* 2007), the structural parameters can be described in terms of dynamic properties, such as frequencies and mode shapes; the changes in these dynamic properties can be utilized to detect level of deterioration. Since frequency represents a global structural property, frequency domain procedures cannot predict the structural health at the element level, rather than are appropriate to predict whether the overall structure is damaged or not. It has been observed that local damage is not always sensitive to the identified modal properties. In addition, for large, complicated structural systems, the higher order modes are difficult to evaluate accurately.

In the time domain approaches (Ma *et al.* 2005, Koh *et al.* 2006, Rutherford *et al.* 2005, Ta *et al.* 2006), dynamic output response measurements can be used to identify the structural parameters, such as stiffness, and detect damages by tracking the changes in these responses. The least square-based approaches are very common in the time domain (Yang and Lin 2005, Yang *et al.* 2007), where the unknown parameters of structural systems are estimated by minimizing the sum of squared errors between the predicted and the measured outputs. Extended Kalman filter (EKF) is another successful technique for structural identification in time domain approaches, however it requires a known dynamic excitation force (Wang and Haldar 1997, Yang *et al.* 2006, 2007). EKF is an optimal recursive data processing algorithm which processes the available response measurements, regardless of their precision. The algorithm uses the prior knowledge about the system and limited response information to produce an estimation of the desired variables by statistically minimizing the error.

In 1994, Wang and Haldar (1994) established a conceptual framework for a SI approach without knowing the dynamic excitation force. They called it the iterative least square with unknown input (ILS-UI) method. They identified shear type buildings (girders/floors are assumed to be infinitely rigid compared with the columns), which is the simplest mathematical representation of complicated structural systems. The total mass of the structure is lumped at the floor levels, assigning one degree of freedom (DOF) for each floor corresponding to the horizontal displacement. Ling and Haldar (2004) improved the efficiency of the identification process, proposed by Wang and Haldar (1994) particularly for large shear type buildings through the use of Rayleigh type damping, i.e., the damping is proportional to mass and stiffness. They called it the modified ILS-UI method or MILS-UI.

Katkhuda *et al.* (2005) proposed the generalized ILS-UI or GILS-UI to identify damages at the local level using only dynamic output response information for more complicated two dimensional steel framed structures with rigid joints. Although the available literature on the health monitoring is very extensive; there is a very limited research on SI techniques and damage detecting in steel framed structures with semi-rigid joints. Wong *et al.* (1995) identified the stiffness of semi-rigid connections for different types of beam-column connections (T-stub, web-seat angle and web-T). Their identification technique was based upon experimental data extracted from response quantities

in the frequency domain. They concluded that the inertia of the member affects the stiffness value of the beam-column connections and that shear deformation cannot be neglected under dynamic loading.

In this paper, the (GILS-UI) method is extended to identify the stiffness of each member in two dimensional frames with semi-rigid connections and to detect the degradation of the stiffness induced in some members without knowing the dynamic excitation force. The concept is based on the axiom that the extent of degradation will be reflected in the changes in the behavior of the structure, i.e., changes in recordable dynamic output responses, and in turn is dependent on the changes in the structural parameters at the element level in terms of local stiffness and damping characteristics. The response of the structure to any dynamic loading is obtained by using finite element software. The structure is modeled as a two dimensional frame, where the connections are modeled as linear elastic rotational spring elements with a rotational stiffness analogous to the Eurocode 3 (2003) definition. The amplitude of the dynamic loads is assumed to be small enough to cause the structure to respond under its serviceability limit state. The hysteretic loops of the semi-rigid connection under the serviceability limit state are very small; therefore it is warranted to assume that the equivalent stiffness of the rotational spring is linear. The acceleration, velocity and displacement or rotation time histories for each node are expected to provide the necessary signature to identify the structure and detect damages in two dimensional framed structures with semi-rigid connections. It was demonstrated that the type of the force, whether blast, harmonic or earthquake is not important in the identification process. The proposed SI technique can be used to identify members' stiffness as well as to detect any degradation in the stiffness which might occur due to any dynamic loading.

2. Modeling of semi-rigid frame structures

The connection beam-to-column of framed structures is usually considered either pinned or rigid joint connection. The pinned joint allows free rotation, while the rigid joint restrains rotation. In actual construction most of connections possesses a moment capacity intermediate between those two cases, thus such connections are classified as semi-rigid joints. Historically, many standards have always accepted joints as either totally pinned or rigid to simplify the analysis and design of steel structures. Modern design codes, such as Eurocode 3 (2003), classify joints according to their actual behavior and incorporated the use of semi-rigid joints. Eurocode 3 (2003) relates the joint stiffness to the attached beam through γ value as in the following equation

$$k_j = \gamma \frac{EI_b}{L_b} \quad (1)$$

where k_j is the stiffness of the joint, i.e., spring; E , I_b , L_b are the material modulus of elasticity; the second moment of area of the cross-section, and the element length, respectively. The subscript 'b' refers to the beam.

Joints are classified as nominally pinned if $\gamma < 0.5$; nominally rigid in braced frames if $\gamma > 8$; and in unbraced frames if $\gamma > 25$. Flexible joints are the ones with γ value between 0.5 and 8. However, many researchers (Monforton and Wu 1963, Kassimali 1999, Cabrero and Bayo 2005) adapted a more attractive parameter named, the end-fixity factor P_i as given in Eq. (2). The advantage of this definition is that it directly relates the joint's rotational behavior with the rotational degrees of

freedom of the beam.

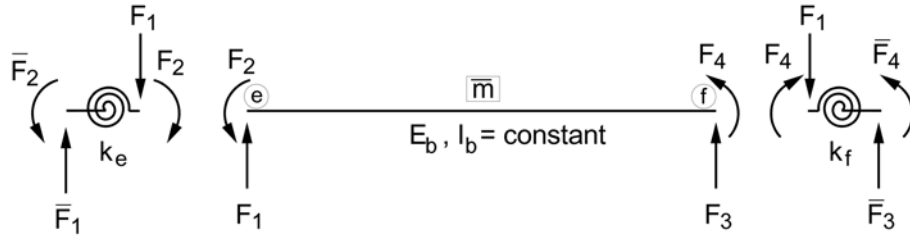
$$P_i = \frac{1}{1 + \frac{3EI_b}{L_b k_j}} \quad (2)$$

This parameter has a null value for theoretically pinned joint and a value of one for theoretically rigid joint. Relating the end-fixity factor to the γ factor proposed by the Eurocode 3 (2003) yields

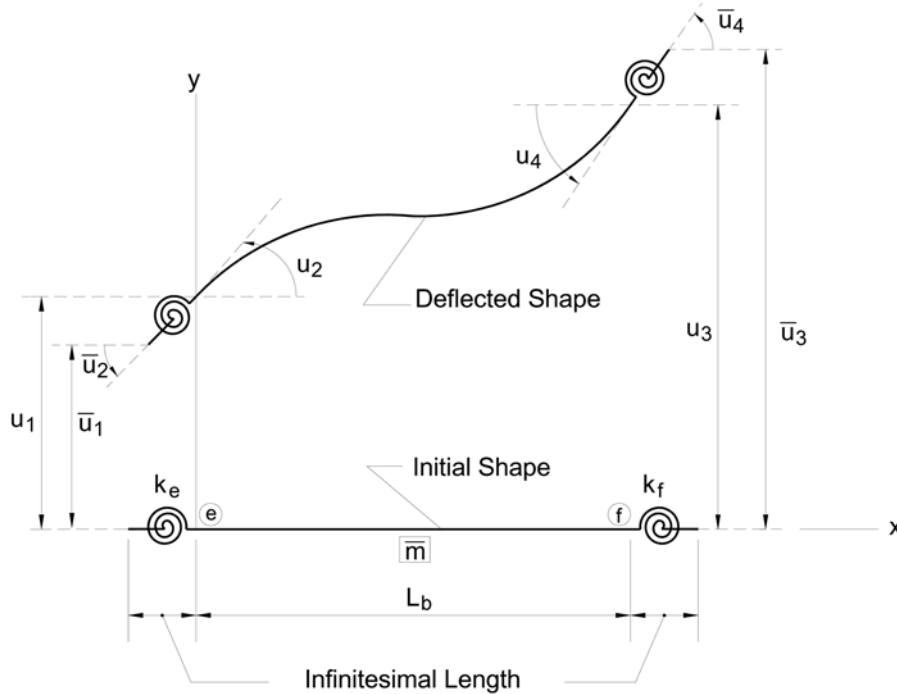
$$P_i = \frac{\gamma}{3 + \gamma} \quad (3)$$

Thus, unbraced semi-rigid frames as defined by the Eurocode 3(2003) have end-fixity factors between 0.14 and 0.73.

Consider a beam of length L_b , moment of inertia I_b , and mass per unit length \bar{m} with flexible



(a) Member forces for a beam element with semi-rigid connections



(b) Member displacements for a beam element with semi-rigid connections

Fig. 1 Modeling of a beam element with semi-rigid connections

joints at its both ends 'e' and 'f', as shown in Fig. 1. The joints are modeled as infinitesimal rotational springs with rotational stiffnesses of k_e and k_f , respectively. Due to the infinitesimal length of the springs, the displacement of the spring ends; u_i , are equal; that is

$$u_1 = \bar{u}_1 \quad (4a)$$

$$u_3 = \bar{u}_3 \quad (4b)$$

In terms of the joint flexibility, the relationship between the rotations (u_2 and \bar{u}_2) of the two ends of the spring at end 'e' can be depicted as follow

$$u_2 = \bar{u}_2 - \frac{\bar{F}_2}{k_e} \quad (5a)$$

Similarly for end 'f'

$$u_4 = \bar{u}_4 - \frac{\bar{F}_4}{k_f} \quad (5b)$$

For the dynamic analysis of a beam element, the field of displacements $w(x, t)$ can be defined in terms of four shape functions $\Psi_i(x)$ and of the nodal displacements $u_1(t), u_2(t), u_3(t), u_4(t)$ at time t , that is

$$w(x, t) = \Psi(x)u(t) = [\psi_1(x) \ \psi_2(x) \ \psi_3(x) \ \psi_4(x)] \begin{bmatrix} u_1(t) \\ u_2(t) \\ u_3(t) \\ u_4(t) \end{bmatrix} \quad (6)$$

The displacement functions, $\Psi_i(x)$, could be taken as an arbitrary shapes which satisfy nodal and internal continuity requirements, such shapes are hermitian polynomials which may be expressed as proposed by Clough and Penzien (1993). The formulation of the consistent mass matrix and stiffness matrix is traditional and can be found in many references, such as (Kassimali 1999, Filho *et al.* 2004, Ochoa 2001). For the purpose of this paper, the formulation of Filho *et al.* (2004) has been adopted.

The conventional consistent matrix (beam element with rigid ends) modified for the presence of semi-rigid connections with end-fixity factors at end 'e' and end 'f' to be P_e and P_f , respectively, is shown below

$$\mathbf{M}^i = \frac{\bar{m}_i L_i}{420 D^2} \begin{bmatrix} 140 D^2 & 0 & 4f_1(P_e, P_f) & & & \\ 0 & 2Lif_2(P_e, P_f) & 4L_i^2 f_5(P_e, P_f) & & & \\ 70 D^2 & 0 & 0 & 140 D^2 & & \\ 0 & 2f_3(P_e, P_f) & Lif_4(P_f, P_e) & 0 & 4f_1(P_f, P_e) & \\ 0 & -Lif_4(P_e, P_f) & -L_i^2 f_6(P_e, P_f) & 0 & -2Lif_2(P_f, P_e) & 4L_i^2 f_5(P_f, P_e) \end{bmatrix} \quad (7)$$

Where

$$D = 4 - P_e P_f$$

$$f_1(P_e, P_f) = 560 + 224P_e + 32P_e^2 - 196P_f - 328P_e P_f - 55P_e^2 P_f + 32P_f^2 + 50P_e P_f^2 + 32P_e^2 P_f^2$$

$$f_2(P_e, P_f) = 224P_e + 64P_e^2 - 160P_e P_f - 86P_e^2 P_f + 32P_e P_f^2 + 25P_e^2 P_f^2$$

$$f_3(P_e, P_f) = 560 - 28P_e - 64P_e^2 - 28P_f - 148P_e P_f + 5P_e^2 P_f - 64P_f^2 + 5P_e P_f^2 + 41P_e^2 P_f^2$$

$$f_4(P_e, P_f) = 392P_f - 100P_e P_f - 64P_e^2 P_f - 128P_f^2 - 38P_e P_f^2 + 55P_e^2 P_f^2$$

$$f_5(P_e, P_f) = 32P_e^2 - 31P_e^2 P_f + 8P_e^2 P_f^2$$

$$f_6(P_e, P_f) = 124P_e P_f - 64P_e^2 P_f - 64P_e P_f^2 + 31P_e^2 P_f^2$$

The stiffness matrix of an arbitrary beam with semi-rigid connection is also given in Eq. (8) below

$$\mathbf{K}^i = \frac{E_i I_i}{L_i} \begin{bmatrix} A_i/I_i & 0 & & & & \\ 0 & \frac{4(B_{11} + B_{12} + B_{22})}{L_i^2} & & & & \\ & & 2(2B_{11} + B_{12}) & & & \\ & & L_i & & & \\ & & & 4B_{11} & & \\ -A_i/I_i & 0 & & 0 & A_i/I_i & \\ 0 & \frac{-4(B_{11} + B_{12} + B_{22})}{L_i^2} & \frac{-2(2B_{11} + B_{12})}{L_i} & 0 & \frac{4(B_{11} + B_{12} + B_{22})}{L_i^2} & \\ & & & & & \\ 0 & \frac{2(B_{12} + 2B_{22})}{L_i} & 2B_{12} & 0 & \frac{-2(B_{12} + 2B_{22})}{L_i} & 4B_{22} \end{bmatrix} \quad (8)$$

where

$$B_{11} = \frac{3P_e}{4 - P_e P_f}$$

$$B_{12} = \frac{3P_e P_f}{4 - P_e P_f}$$

$$B_{22} = \frac{3P_f}{4 - P_e P_f}$$

Eq. (8) can be rewritten as

$$\mathbf{K}^i = k_i \mathbf{K}_{SR}^i \quad (9)$$

where $k_i = E_i I_i / L_i$, and \mathbf{K}_{SR}^i is the 6×6 matrix shown in Eq. (8) in the square bracket for the i^{th} member.

3. GILS-UI method formulation for elements with semi-rigid connections

The equation of motion for structural system of N degrees of freedom (DOF) can be written in matrix form as

$$\mathbf{M}\ddot{\mathbf{u}}(t) + \mathbf{C}\dot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = \mathbf{F}(t) \quad (10)$$

Where \mathbf{M} , \mathbf{C} , \mathbf{K} are the global mass, damping and stiffness matrices of the structure, respectively. $\ddot{\mathbf{u}}(t)$, $\dot{\mathbf{u}}(t)$, $\mathbf{u}(t)$ are acceleration, velocity and displacement vectors at time t , respectively, and $\mathbf{F}(t)$ is the dynamic excitation force vector.

In this method, it is assumed that the mass of each member and the dynamic responses in terms of acceleration, velocity and displacement for each DOF are known. While the stiffness of each member, damping and dynamic excitation force are unknown.

Rayleigh-type damping is used in this method and the damping matrix \mathbf{C} can be represented as

$$\mathbf{C} = \alpha\mathbf{M} + \beta\mathbf{K} \quad (11)$$

where factor α is the mass-proportional damping coefficient and β is the stiffness-proportional damping coefficient.

Eq. (10) can be rewritten as

$$(\alpha\mathbf{M} + \beta\mathbf{K})\dot{\mathbf{u}}(t) + \mathbf{K}\mathbf{u}(t) = \mathbf{F}(t) - \mathbf{M}\ddot{\mathbf{u}}(t) \quad (12)$$

For the purpose of system identification, Eq. (12) can be reorganized in a matrix form as

$$\mathbf{B}(t)_{(N \cdot m) \times h} \mathbf{D}_{h \times 1} = \mathbf{G}(t)_{(N \cdot m) \times 1} \quad (13)$$

where $\mathbf{B}(t)$ is a matrix composed of the system response vectors of velocity and displacement at each DOF at time t , N is the total number of DOFs at time t , m is the total number of sample points, h is the total number of unknown parameters, \mathbf{D} is a vector composed of the unknown system parameters to be identified, and $\mathbf{G}(t)$ is a vector composed of dynamic excitations and inertia forces at each DOF at time t .

The \mathbf{B} matrix for two dimensional steel plane frames with semi-rigid connections can be expressed as

$$\mathbf{B}_{(N \cdot m) \times h} = [\mathbf{K}_{SR}^1 \mathbf{u}(t) \quad \mathbf{K}_{SR}^2 \mathbf{u}(t) \quad \dots \quad \mathbf{K}_{SR}^{ne} \mathbf{u}(t) \quad \mathbf{K}_{SR}^1 \dot{\mathbf{u}}(t) \quad \mathbf{K}_{SR}^2 \dot{\mathbf{u}}(t) \quad \dots \quad \mathbf{K}_{SR}^{ne} \dot{\mathbf{u}}(t) \quad \mathbf{M}\ddot{\mathbf{u}}(t)] \quad (14)$$

where ne is total number of members in the structure and \mathbf{K}_{SR}^i as defined in Eq. (9).

The unknown system parameters \mathbf{D} vector in Eq. (13) can be expressed as

$$\mathbf{D}_{h \times 1} = [k_1, k_2, \dots, k_{ne}, \beta k_1, \beta k_2, \dots, \beta k_{ne}, \alpha]^T \quad (15)$$

where $k_i = E_i I_i / L_i$ is the unknown stiffness parameter for the i^{th} beam element that needs to be identified.

The $\mathbf{G}(t)$ vector in Eq. (13) can be defined as

$$\mathbf{G}(t)_{(N \cdot m) \times 1} = \mathbf{F}(t)_{(N \cdot m) \times 1} - \mathbf{M}\ddot{\mathbf{u}}(t)_{(N \cdot m) \times 1} \quad (16)$$

where $\mathbf{F}(t)$ is the unknown excitation force vector, \mathbf{M} is the known global consistent mass matrix that can be assembled from Eq. (7), and $\ddot{\mathbf{u}}(t)$ is the vector containing the dynamic responses in terms of acceleration as stated earlier.

The least-square technique is used to solve for the system parameter vector \mathbf{D} , i.e., Eq. (13); by

starting an iteration process. It is based on minimizing the total error, Er , in the identification of the structure as shown below

$$Er = \sum_{r=1}^{m \times N} \left(\mathbf{G}_r - \sum_{s=1}^h \mathbf{B}_{rs} \mathbf{D}_s \right)^2 \quad (17)$$

To minimize the total error, Eq. (17) can be differentiated with respect to each one of the \mathbf{D}_p parameters as

$$\frac{\partial Er}{\partial \mathbf{D}_p} = \sum_{r=1}^{m \times N} \left(\mathbf{G}_r - \sum_{s=1}^h \mathbf{B}_{rs} \mathbf{D}_s \right) \mathbf{G}_{rp} = 0 \quad (18)$$

It is relatively simple to solve Eq. (13) to obtain the unknown system parameters (vector \mathbf{D}) provided that the matrix $\mathbf{B}(t)$ and vector $\mathbf{G}(t)$ are known. However, as mentioned earlier, the dynamic excitation is not known; thus, the vector $\mathbf{G}(t)$ becomes a partially unknown vector. To overcome this issue; the iteration process is started by assuming the unknown dynamic excitation vector $\mathbf{F}(t)$ to be zero at all time points m . This assumption will assure a non-singular solution of Eq. (13) without compromising the convergence or the accuracy of the method. It is observed that the method is not sensitive to this initial assumption.

With this assumption, the $\mathbf{G}(t)$ vector in Eq. (13) can be obtained and a first estimate of the unknown system parameters \mathbf{D} . Using Eq. (12) and the estimated system parameters \mathbf{D} , the information on the dynamic excitation force $\mathbf{F}(t)$ can be generated at all time points m . Using the information on the generated dynamic excitation force and Eq. (13), the estimate of the system parameters \mathbf{D} can be updated. The algorithm will iterate until the system parameters are evaluated with a pre-determined accuracy. The convergence criterion is set with respect to the evaluated dynamic excitation force. At least one constraint to the unknown dynamic excitation force needs to be available; this constraint has to be related to the locations of the forces.

3.1 Summary of the GILS-UI iterative algorithm

The basic steps for the iterative algorithm are shown in Fig. 2 and can be summarized as follows:

Step 1: Assemble the global consistent mass matrix \mathbf{M} for the frame from the local mass matrices from Eq. (7) of all the elements with semi-rigid connections by considering their connectivity.

Step 2: Formulate the local semi-rigid stiffness matrix \mathbf{K}_{SR}^i for each element from Eq. (8).

Step 3: Formulate the $\mathbf{B}(t)$ matrix from Eq. (14) which is composed of global consistent mass matrix \mathbf{M} , the local semi-rigid stiffness matrices of each element \mathbf{K}_{SR}^i , and the velocity $\dot{\mathbf{u}}(t)$ and displacement $\mathbf{u}(t)$ responses of the system as at each DOF.

Step 4: Formulate the $\mathbf{G}(t)$ vector from Eq. (16) and assume the dynamic excitation force vector $\mathbf{F}(t)$ to be zero at all time points m .

Step 5: Obtain the first estimation of the system parameter vector \mathbf{D} by solving Eq. (13) using the least-square concept.

Step 6: Substitute system parameters \mathbf{D} estimated from Step 5 into Eq. (12) to obtain the unknown dynamic excitation force vector $\mathbf{F}(t)$ at all time points m .

Step 7: Apply force constraints to the dynamic excitation force $\mathbf{F}(t)$ estimated in step 6. For example, if the force is exactly zero at the i^{th} DOF, $F_i(t) = 0$ need to be introduced and if input force at the j^{th} DOF is equal to the input force, $F_i(t) = F_j(t)$.

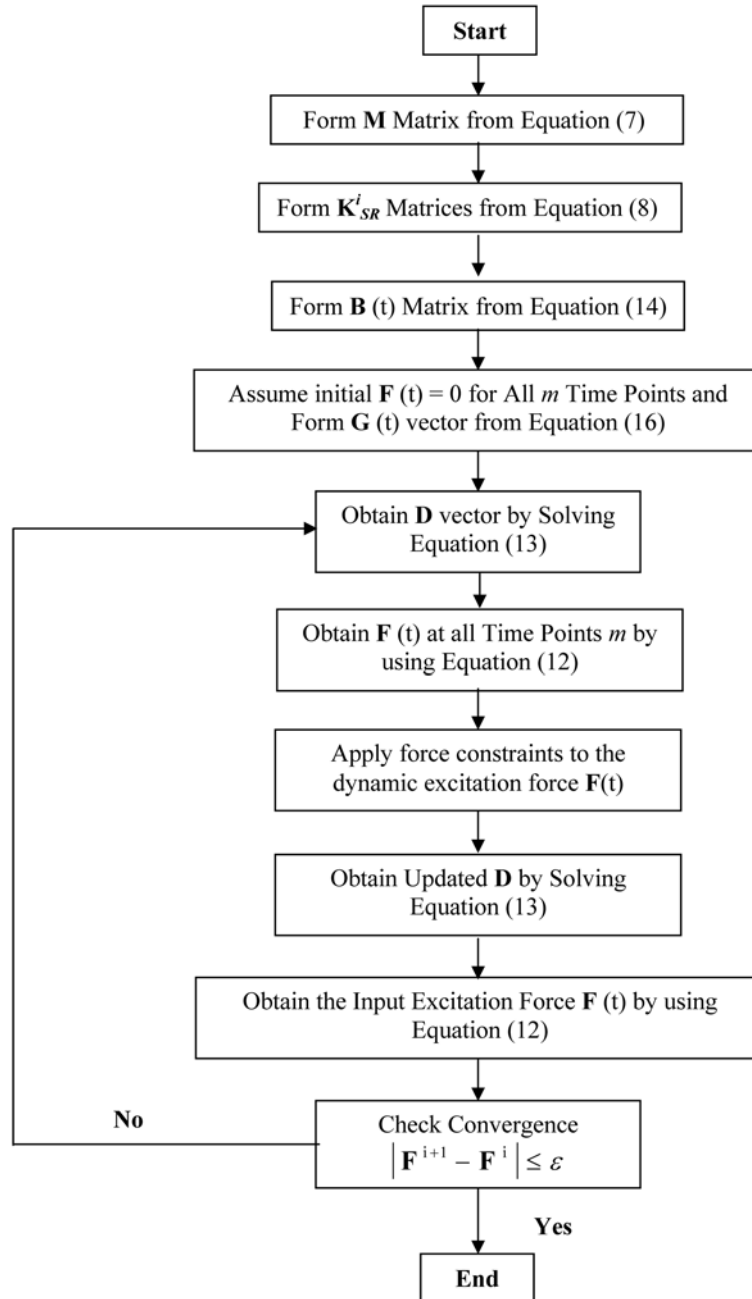


Fig. 2 Flow chart of the iterative algorithm GILS-UI for elements with semi-rigid connections

Step 8: Obtain the updated estimation on the system parameter vector **D** by solving Eq. (13).

Step 9: Update the dynamic excitation force vector **F**(*t*) using Eq. (12) and the updated system parameter vector **D**.

Step 10: Continue the iterative process until the convergence in the dynamic excitation force **F**(*t*) is obtained with a predetermined tolerance (ε). The tolerance (ε) is set to be 10^{-6} in this paper.

4. Numerical studies

Nine numerical examples are presented in this section to demonstrate the applicability, and assess the effectiveness of the proposed SI algorithm. In these examples the structural parameters were identified and damages were detected for two-dimensional steel framed buildings with semi-rigid connections. A one-bay semi-rigid frame (denoted as 1B) as shown in Fig. 3 and a two-bay semi-rigid frame (denoted as 2B) as shown in Fig. 4 were used in these examples. In the 1B frame the connections of the beam to the columns are assumed to be semi-rigid. The end-fixity factor P for the beam (i.e., member 3) at node 1 and node 2 are assumed to be 0.20 and 0.40, respectively as shown in Fig. 3. W14 \times 53 steel sections are used for all the members. The masses of the beam and columns are assumed to be known and are equal to 696 kg and 271 kg, respectively. The stiffness of the beam and columns ($k = EI/L$), are calculated to be 4,924 kN/m and 12,311 kN/m, respectively.

In the 2B frame all the connections are assumed to be semi-rigid where the end fixity factor P for each connection is shown in Fig. 4. W18 \times 71 steel sections are used for all the members. The masses of each beam and column are assumed to be known and are equal to 932 kg and 362.5 kg, respectively. The beam and column stiffnesses ($k = EI/L$) are calculated to be 10,650 kN/m and 26,625 kN/m, respectively.

Three cases were considered for each frame, namely, no damage (denoted ND), damage one (denoted D1), and damage two (denoted D2). Damage one (D1) was resembled in the 1B frame by a 10% reduction in the stiffness of the beam; whereas in the 2B frame it was resembled by a 10% reduction in the stiffness of the second beam, i.e., member 2, and 25% stiffness reduction in the third column, i.e., member 5. Damage two (D2) was resembled in the 1B frame by a 20% reduction in the stiffness of the beam; whereas in the 2B frame it was resembled by a 20% reduction in the stiffness of the second beam, i.e., member 2, and 50% stiffness reduction in the third column, i.e.,

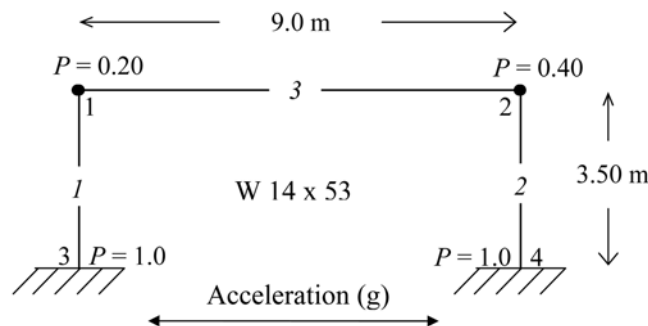


Fig. 3 Structural details of one-bay frame (1B)

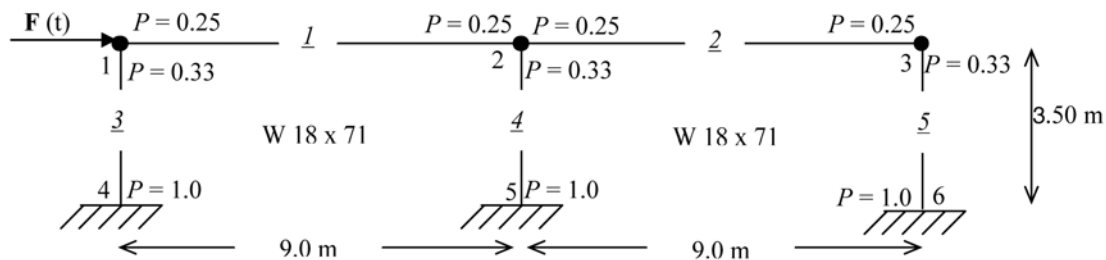


Fig. 4 Structural details of two-bay frame (2B)

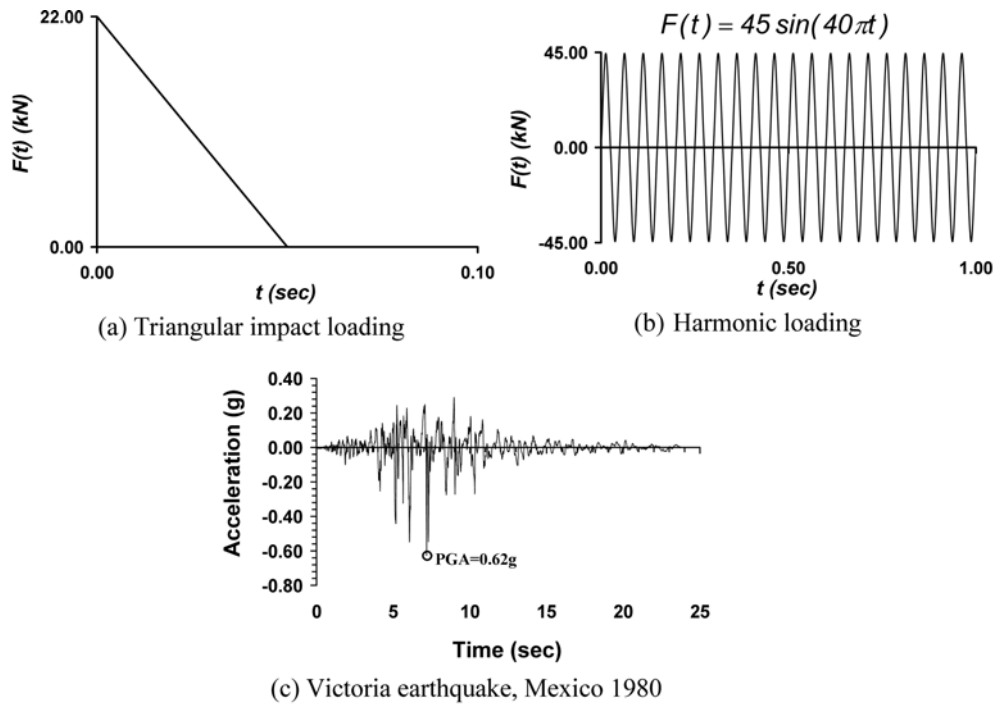


Fig. 5 Dynamic loads used in the numerical study, (a) Impact loading, (b) Harmonic loading, (c) Earthquake loading

Table 1 Details of the numerical cases

Case ID	One-bay	Two-bay	Earthquake	Impact	Harmonic	No damage	Damage 1	Damage 2
1B-E-ND	√		√			√		
1B-E-D1	√		√				√	
1B-E-D2	√		√					√
2B-I-ND		√		√		√		
2B-I-D1		√		√			√	
2B-I-D2		√		√				√
2B-H-ND		√			√	√		
2B-H-D1		√			√		√	
2B-H-D2		√			√			√

member 5. Three types of loadings were considered, namely, earthquake load (E), impact load (I), and harmonic load (H) as shown in Fig. 5. A total of 9 cases were studied; details of all these cases are shown in Table 1.

The responses of all the cases given in Table 1, i.e., the acceleration, velocity and displacement of all DOFs, subjected to the specified dynamic loadings, shown in Fig. 5, were obtained using RAUAMOKO (Carr 1996), a nonlinear finite element software package. These responses resemble the recorded data of a real structure under the effect of the dynamic loadings.

After the theoretical responses are evaluated, the information on the dynamic loadings is

Table 2 Stiffness ($k = EI/L$) identification for earthquake loading cases

Members	Theoretical (kN/m)	1B-E-ND		1B-E-D1		1B-E-D2	
		Identified (kN/m)	Error %	Identified (kN/m)	Effect %	Identified (kN/m)	Effect %
k_1	12311	12309.4	-0.013	12309.3	-0.014	12291.1	-0.16
k_2	12311	12310.8	-0.0016	12310.7	-0.0024	12292.8	-0.14
k_3	4924	4923.8	-0.0041	4432.4	-9.98	3936.2	-20.06

Table 3 Stiffness ($k = EI/L$) identification for impact load cases

Members	Theoretical (kN-m)	2B-I-ND		2B-I-D1		2B-I-D2	
		Identified (kN-m)	Error %	Identified (kN-m)	Effect %	Identified (kN-m)	Effect %
k_1	10650	10563.8	-0.809	10477.4	-1.62	10015.7	-5.95
k_2	10650	10568.8	-0.762	9583.6	-10.01	8852.7	-16.87
k_3	26625	26405.2	-0.826	26156.3	-1.76	24874.7	-6.57
K_4	26625	26413.5	-0.794	26431.4	-0.72	26210.4	-1.55
K_5	26625	26419.9	-0.770	19882.3	-25.32	13805.2	-48.15

Table 4 Stiffness ($k = EI/L$) identification for harmonic load cases

Members	Theoretical (kN-m)	2B-H-ND		2B-H-D1		2B-H-D2	
		Identified (kN-m)	Error %	Identified (kN-m)	Effect %	Identified (kN-m)	Effect %
k_1	10650	10569.3	-0.757	10322.2	-3.07	10012.6	-5.98
k_2	10650	10567.9	-0.770	9798.8	-7.99	9480.3	-10.98
k_3	26625	26419.2	-0.772	25575.9	-3.94	24765.5	-6.98
K_4	26625	26419.1	-0.773	26641.3	+0.061	27003.7	+1.42
K_5	26625	26416.7	-0.782	20769.3	-21.99	14823.5	-44.32

completely ignored, and the nodal responses are used in the algorithm to identify the stiffness of the elements. The responses used in the algorithm are from 0.02 sec to 0.83 sec at a time interval of 0.01 sec, yielding 82 time points. The theoretical and identified stiffness parameters (EI/L) of all members for cases 1B-E-ND, 1B-E-D1, and 1B-E-D2 are shown in Table 2. The results indicate that the algorithm identified the parameters almost exact for all three cases. In case 1B-E-D1 the algorithm identified a 9.98% stiffness reduction in the beam, whereas in case 1B-E-D2 a 20.06% stiffness reduction was identified. These values are almost equal to the pre-imposed reduction of 10% and 20% for cases 1B-E-D1 and 1B-E-D2, respectively.

Table 3 shows the theoretical and identified stiffness parameters (EI/L) of all members for cases 2B-I-ND, 2B-I-D1, and 2B-I-D2. The algorithm identified correctly the parameters for the undamaged case and identified correctly the members with the pre-imposed damage. Finally, Table 4 shows the theoretical and identified stiffness parameters (EI/L) of all members for cases 2B-H-ND, 2B-H-D1, and 2B-H-D2. The algorithm identified correctly the parameters for the

undamaged case and identified correctly the members with the pre-imposed damage. In case 2B-H-D2, the pre-imposed damage was identified in member 2 and members 5.

5. Discussion

For the cases studied in this paper, the GILS-UI method for steel frames with semi-rigid connections identified the stiffnesses of the damage-free members very well with an error less than 1%. The responses used in the algorithm were 82 time points, i.e., from 0.02 to 0.83 sec. Although the algorithm used a small number of time points, the results of identification are considered to be accurate. It has been observed by the authors that the number of sample points does not have major impact on the accuracy of the identified stiffnesses. This can be considered as an advantage since most of time domain methods available in the literature are very sensitive to the number of sample points used in identification.

All the cases showed clearly that the method was able to identify the damages pre-imposed in the frames. The most accurate identification process was in the 1B frame, which was subjected to Victoria earthquake acceleration time-history. The results showed that there is a stiffness reduction of 9.98% and 20.06%, respectively in the beam and no damage in the remaining members. For the 2B-I frame cases; the results showed that the method detected the pre-imposed damages in one of the beams and columns when the damages were relatively small and large. However, in the largely damaged case, i.e., 2B-I-D2, there was a false detection of damages in two members where the algorithm identified a stiffness reduction of almost 6% in member 1 and member 3. As mentioned earlier; the location and amount of stiffness reduction were selected randomly and the damages were simulated theoretically. In this case a stiffness reduction of 20% and 50% were pre-imposed in two members and the other members were kept damage-free. Usually, in a real case with a major blast load there will be a large damage in some members and minor in others and that is exactly what the algorithm identified.

The results showed that the method was able to identify the stiffnesses and detect damages for the frames that are subjected to different types of loading whether earthquake, blast or harmonic. Also as a by-product, the method identified the unknown dynamic forces subjected to the structures very accurately since it is the convergence criteria used in the algorithm as shown in Fig. 6. This can be considered another advantage of the method since most of the methods available in the literature need the time-history of the dynamic force, and for the methods that does not require a pre-knowledge of the dynamic force, the type of the force is very important in the identification process.

The responses used for identification in all the cases presented are considered noise-free. Many procedures were proposed by researches in the last decade to overcome the issue of noise in the responses. For instance, Cooper (2004) proposed a weighted least-square approach that can be applied to a wide range of different time domain system identification algorithms and models. At different stages in developing the GILS-UI method, it was verified theoretically by adding noise in the responses (Wang and Haldar 1994, Ling and Haldar 2004, Katkhuda *et al.* 2005) and lately Vo and Haldar (2008) verified the method experimentally on beams, whereas Martinez-Flores *et al.* (2006), Martinez-Flores and Haldar (2007), and Haldar *et al.* (2008) verified it experimentally on a two-dimensional steel frame with rigid connections. The authors decided not to consider noise since the issue of noise is not a problem in the method as verified by others.

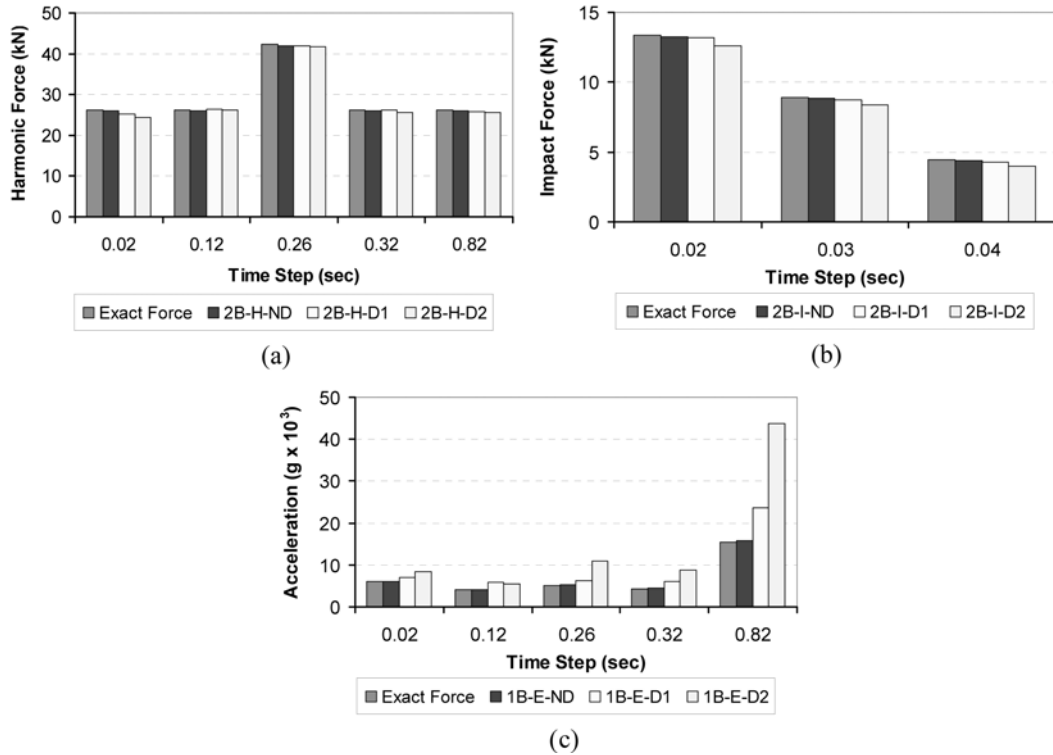


Fig. 6 (a) Identified harmonic load at selected time steps, (b) Identified impact load at selected time steps, (c) Identified earthquake load at selected time steps

6. Conclusions

A system identification procedure is presented to identify the structural parameters at the local element level of damaged and undamaged framed structures with semi-rigid connections. The procedure detects damages by tracking the changes in stiffness properties of each element based upon the response data for all the degrees-of-freedom of the frame structure. The most attractive feature of the procedure is that it does not require dynamic excitation force information as an input for the identification process. With the help of several numerical examples it is shown that the method can accurately identify structures and detect damages for frames excited by different types of dynamic loadings including impact, harmonic, and earthquake. Damages can be small or relatively large, yet correctly detected. In all cases presented, the algorithm identified the structures correctly and the error in the identification is reasonably small. The procedure has the potential to be used as a structural health assessment and monitoring technique. It is expected to be simple and economic, yet reliable and accurate.

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