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An improved finite element model updating method by the global optimization technique 'Coupled Local Minimizers'

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Abstract

Finite element (FE) model updating technique belongs to the class of inverse problems in classical mechanics. According to the continuum damage mechanics, damage is represented by a reduction factor of the element bending stiffness. In this study, a global optimization method called 'Coupled Local Minimizers' (CLM) is used for updating the finite element model of a complex structure. In CLM, the local optimization processes are coupled so that better solutions than multistart local optimization consisting of independent runs are obtained. This is achieved by minimizing the average cost function of the local minimizers subjected to pairwise synchronization constraints. An augmented Lagrangian which contains the synchronization constraints both as soft and hard constraints is used and a network is derived in which the local minimizers communicate and exchange information through the synchronization constraints. In this study, the finite element model updating method is applied on a complex structure with a complex damage pattern and 24 design variables using CLM. The damage scenario on the structure is based on the hinge pattern obtained from nonlinear dynamic time history analysis. The results show that damage is detected, localized and quantified very accurately by the FE model updating algorithm used. In the second phase of the paper, two levels of noise, namely; moderate and high noise are applied on the modal parameters. In the presence of noise, damage is located and detected very accurately. The extent of the damage is also quantified precisely and the MAC values as well as the relative eigenfrequency differences are improved substantially. In the third phase of the study, the CLM method is compared with other local optimization methods such as the Levenberg–Marquardt algorithm, Sequential Quadratic Programming and Gauss–Newton methods and the results show that the CLM algorithm gives better results in FE model updating problems compared to the above-mentioned local optimization methods.

Keywords: FE model updating; Continuum damage mechanics; Coupled Local Minimizers; Damage detection; Inverse problems in mechanics; Nonlinear dynamic analysis

1. Introduction

Structural damage in civil engineering structures results in changes in the modal parameters such as; the natural frequencies, mode shapes and modal damping values. Modal parameters can be easily obtained from vibration testing. Four levels of damage identification is possible as proposed by Rytter [1].

(1) Level 1 – *detection*: Is the structure damaged or not?
(2) Level 2 – *localization*: What is the location of the damage in the structure?
(3) Level 3 – *quantification*: What is the extent of damage?
(4) Level 4 – *prediction*: What is the remaining service life of the structure?

During the past few years, significant amount of damage identification techniques have been proposed and successfully validated on vibration data from a wide range of mechanical, aerospace and civil engineering structures [2]. A detailed literature survey on the topic can be found in

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[3–7]. Levin and Lieven [8] state that the FE model updating methods can be categorized into two. The first are the direct methods in which the mass and the stiffness matrices are updated directly. The second are indirect or parametric methods in which parametric changes are made to the model. It has been shown that direct methods are not physically meaningful [9]. Most studies in literature make use of an analytical model (e.g., finite element (FE) model) of the structure [10–14]. Other studies aim for the same objective without using such models [15,16].

There are different approaches in the literature for model updating via parametric changes to an existing model [17]. One common approach is to consider an objective function that quantifies the difference between the measured and the analytical data. A set of parameters are then updated to minimize this function. Thus, model updating becomes a constrained optimization problem. Levin and Lieven [8] state that an optimization problem is difficult if it satisfies criteria such as: high dimension, many local minima, high nonlinearity, non-smoothness, noisiness and discreteness. Model updating problems generally satisfy the first five of these six criteria. Therefore, optimization problems either do not converge or get stuck in local minima. Global optimization techniques are imperative to find the global minimum in optimization problems which may have many local minima. Levin and Lieven [8] have successfully applied two new global optimization methods; namely, genetic algorithms and simulated annealing to finite element model updating problems. Both of the methods are probabilistic search algorithms that are derived from analogies with the natural phenomena. Simulated annealing is developed based on inspiration from a thermodynamic cooling process and genetic algorithms from natural evolution.

In this study, a newly proposed global optimization method called ‘Coupled Local Minimizers’ (CLM) [18,19] is used for updating the FE model of a complex structure with 24 design variables. The initial starting values selected for the design variables affect the results of the optimization processes. Multi start local optimization methods have been used in the past to overcome this problem. The advantage of the CLM over the multistart local optimization methods is the fact that these points communicate and exchange information throughout the process. The local optimizers are coupled by pairwise synchronization constraints so that the design variables are forced to converge to the same point. The average of the objective function obtained from all the CLM points is then minimized. Instead of the probabilistic search common to the genetic algorithms and simulated annealing methods, CLM uses derivative information in each of the search points to direct the global search process [14,20] which results in faster convergence. In this study, FE model updating by the CLM method will be applied on a complex civil engineering structure that has a complex damage pattern.

This paper is organized as follows. In Section 2, the ‘Coupled Local Minimizers’ are explained. In Section 3, the theoretical background of the FE model updating is introduced. In Section 4, the description of the structure type considered in the FE model updating problem is given. In Section 5, the background for the damage scenario is discussed. Section 6 presents the analysis of the results. Section 7 gives the comparison of the CLM method with other optimization algorithms. In Section 8, the application of the method on actual structures is discussed. Section 9 summarizes the Conclusions.

2. Coupled Local Minimizers

The originality of the CLM comes from the fact that an ensemble of optimizers which is subject to pairwise state synchronization constraints are considered. When two local minimizers are stuck in different local minima, the state synchronization constraint urges them to take a decision and avoid the local minimum. The synchronization constraints have to be achieved in an asymptotical sense, i.e. the particles have to reach the same final state [18]. The difference of the CLM from the multistart local optimization methods is the interaction and communication between the several different starting points. In this study, the CLM technique is implemented with an augmented Lagrangian method. The augmented Lagrangian function $\mathscr{L}_A$ is defined by the average objective function of the population together with the synchronization constraints between the individual local minimizers. An unconstrained optimization algorithm is used to minimize $\mathscr{L}_A$.

2.1. Augmented Lagrangian method

The augmented Lagrange multiplier (ALM) method is explained in detail in the books of Rao [21] and Nocedal and Wright [22]. ALM method combines the Lagrange multiplier and the penalty function methods as shown below:

$$\text{Minimize} \quad f(\mathbf{x}) \tag{1}$$

$$\text{subject to} \quad h_j(\mathbf{x}) = 0, \quad j = 1, 2, \ldots, p, \quad p < n. \tag{2}$$

The Lagrangian corresponding to these equations will be given as

$$\mathscr{L}(\mathbf{x}, \lambda) = f(\mathbf{x}) + \sum_{j=1}^{p} \lambda_j h_j(\mathbf{x}), \tag{3}$$

where λ_j, $j = 1, 2, \ldots, p$, are the Lagrange multipliers. The exterior penalty function approach is used to define the new objective function $\mathscr{L}_A(\mathbf{x}, \lambda, \gamma)$, termed the *augmented Lagrangian function*, as

$$\mathscr{L}_A(\mathbf{x}, \lambda, \gamma) = f(\mathbf{x}) + \sum_{j=1}^{p} \lambda_j h_j(\mathbf{x}) + \gamma \sum_{j=1}^{p} h_j^2(\mathbf{x}), \tag{4}$$

where γ is the penalty parameter. It can be noted that the function $\mathscr{L}_A$ reduces to the Lagrangian if $\gamma = 0$ and to the function used for the classical penalty function methods if all $\lambda_j = 0$. It can also be shown that if the Lagrange multipliers are fixed at their optimum values λ_j^*, the minimization $\mathscr{L}_A(\mathbf{x}, \lambda, \gamma)$ gives the solution of the problem mentioned in Eqs. (1) and (2) in one step regardless of the value of γ. In such a case, there is no need to minimize the function $\mathscr{L}_A$ for an increasing values of sequence of γ. Since the values of λ_j are not known in advance, an iterative scheme is used to find the solution of the problem. In the first iteration, the values of λ_j are chosen as zero, the value of γ is set to an arbitrary constant and the function $\mathscr{L}_A$ is minimized with respect to x to find x^*. The values of λ_j and γ are then updated for the next iteration. The updating is carried out as follows. The necessary conditions for the stationary point of the Lagrangian $\mathscr{L}$ can be expressed as

$$\frac{\delta \mathscr{L}}{\delta x_i} = \frac{\delta f}{\delta x_i} + \sum_{j=1}^{p} \lambda_j^* \frac{\delta h_j}{\delta x_i} = 0, \quad i = 1, 2, \ldots, n, \tag{5}$$

where λ_j^* are the optimum values of the Lagrangian at the stationary point. Similar to the above case, the necessary conditions for the minimum of the augmented Lagrangian can be expressed as

$$\frac{\delta \mathscr{L}_A}{\delta x_i} = \frac{\delta f}{\delta x_i} + \sum_{j=1}^{p} (\lambda_j + 2\gamma h_j) \frac{\delta h_j}{\delta x_i} = 0, \quad i = 1, 2, \ldots, n. \tag{6}$$

A comparison of the right hand sides of the above two equations gives:

$$\lambda_j^* = \lambda_j + 2\gamma h_j, \quad j = 1, 2, \ldots, p. \tag{7}$$

These equations are used to update the values of λ_j as

$$\lambda_j^{k+1} = \lambda_j^k + 2\gamma_k h_j(\mathbf{x^{(k)}}), \quad j = 1, 2, \ldots, p, \tag{8}$$

where k is the iteration number. The value of γ_k is updated as

$$\gamma_{k+1} = c\gamma_k, \quad c > 1. \tag{9}$$

Teughels [10] has shown that updating the penalty parameter γ does not improve the CLM. Furthermore, updating γ implies the choice of an additional unknown parameter, the updating factor c. For this reason, the augmented Lagrangian method is implemented with a constant γ in the FE model updating study presented in this paper.

2.2. Coupled local minimizers method

In CLM, an ensemble of q local minimizers are considered whose average objective function are calculated as

$$\langle f \rangle = \frac{1}{q} \sum_{i=1}^{q} f(x^{(i)}). \tag{10}$$

Pairwise synchronization constraints are applied to the design vectors $x^{(i)}$ (=vectors of variables), resulting in a constrained minimization problem:

$$\min_{x^{(i)} \epsilon \Re^n} \quad \text{such that} \quad x^{(i)} - x^{(i+1)} = 0 \tag{11}$$

for $i = 1, 2, \ldots, q$ and with boundary condition $x^{(q+1)} = x^{(1)}$. The augmented Lagrangian is defined for this problem as

$$\mathscr{L}_A(\mathbf{x}, \mathbf{\Lambda}) = \frac{\eta}{q} \sum_{i=1}^{q} f(x^{(i)}) + \sum_{i=1}^{q} \langle \boldsymbol{\lambda}^{(i)}, [x^{(i)} - x^{(i+1)}] \rangle + \frac{\gamma}{2} \sum_{i=1}^{q} \| x^{(i)} - x^{(i+1)} \|^2 \tag{12}$$

with $\mathbf{x} = [x^{(1)}, \ldots, x^{(q)}]$ and $\mathbf{\Lambda} = [\boldsymbol{\lambda}^{(1)}, \ldots, \boldsymbol{\lambda}^{(q)}]$, $(x^{(i)}, \boldsymbol{\lambda}^{(i)} \epsilon \Re^n)$. $\langle .,. \rangle$ denotes the inner product for the hard constraints, and $\|.\|$ the Euclidean norm of a vector for the soft constraints, and η is a weighting factor of the average objective function. The synchronization constraints have to be achieved in an asymptotical sense, i.e. the particles have to reach the same final state.

2.3. Numerical aspects

Suykens et al. [18,19] applied the CLM method with a steepest descent algorithm. In this paper, just as implemented in Teughels et al. [14], the CLM method is applied with a Trust Region Newton Method [22] for minimizing $\mathscr{L}_A(\mathbf{x}, \mathbf{\Lambda})$ with respect to $\mathbf{x}$. Therefore, a quadratic approximation $m(\mathbf{p})$ of $\mathscr{L}$ from a truncated Taylor series is minimized within a trust region Δ_s:

$$\min_{p} \quad m(\mathbf{p}) = \mathscr{L}_A + [\nabla \mathscr{L}_A]^{\mathrm{T}} \mathbf{p} + \frac{1}{2} \mathbf{p}^{\mathrm{T}} [\nabla^2 \mathscr{L}_A] \mathbf{p}, \quad \text{such that } \|\mathbf{p}\| \leqslant \Delta, \tag{13}$$

where $\mathbf{p}$ denotes a step vector from $\mathbf{x}_s$, and $\mathscr{L}_A, \nabla \mathscr{L}_A$ and $\nabla^2 \mathscr{L}_A$ are the function, the gradient and the Hessian of $\mathscr{L}_A$ at $\mathbf{x}_s$, respectively. It is accepted that each local minimizer is independent of the values of other minimizers. The gradient as well as the Hessian of the augmented Lagrangian are expressed as

$$\nabla_{x^{(i)}} \mathscr{L}_A = \frac{\eta}{q} \nabla_{x^{(i)}} f(x^{(i)}) - \lambda^{(i-1)} + \lambda^{(i)} - \gamma[x^{(i-1)} - x^{(i)}] + \gamma[x^{(i)} - x^{(i+1)}], \tag{14}$$

$$\nabla^2_{x^{(i)}} \mathscr{L}_A = \frac{\eta}{q} \nabla^2_{x^{(i)}} f(x^{(i)}) + 2\gamma \mathbf{I}, \tag{15}$$

$$\nabla^2_{x^{(i)} x^{(i-1)}} \mathscr{L}_A = -\gamma \mathbf{I}, \tag{16}$$

$$\nabla^2_{x^{(i)} x^{(i+1)}} \mathscr{L}_A = -\gamma \mathbf{I}, \tag{17}$$

where $\mathbf{I}$ denotes the identity matrix $n \times n$. Since a Newton based method is used, the speed of the CLM algorithm is accelerated. The convergence of the algorithm is improved by the use of a Trust Region strategy. In addition, boundary constraints on the design variables are applied so that the trust region is restricted more effectively.

3. FE model updating

Sensitivity-based FE model updating method is the most frequently used updating method [11–13]. This method directly compares eigenfrequencies and mode shapes. The cost function is stated as a nonlinear least squares problem and the model updating is carried out by minimizing the residual:

$$f(\boldsymbol{a}) = \frac{1}{2}\|\mathbf{r}(\boldsymbol{a})\|^2, \tag{18}$$

where

$$\mathbf{r}(\boldsymbol{a}) = \begin{bmatrix} \mathbf{r}_{\mathrm{f}}(\boldsymbol{a}) \\ \mathbf{r}_{\mathrm{s}}(\boldsymbol{a}) \end{bmatrix} \quad \mathbf{r}_{\mathrm{f}} : \Re^n \to \Re^{m_{\mathrm{f}}}, \quad \mathbf{r}_{\mathrm{s}} : \Re^n \to \Re^{m_{\mathrm{s}}}. \tag{19}$$

$\boldsymbol{a}$ is the vector of design variables, m_{f} denotes the number of identified eigenfrequencies that are used in the updating process and m_{s} is equal to the product of the number of identified mode shapes m_{m} and the number of DOFs used for mode ϕ_i. The eigenfrequency residuals, r_{f}, can be expressed as

$$r_{\mathrm{f}}(\boldsymbol{a}) = \frac{\omega_j^2(\boldsymbol{a}) - \tilde{\omega}_j^2}{\tilde{\omega}_j^2}, \tag{20}$$

where ω_j and $\tilde{\omega}_j$ are the numerical and experimental eigenfrequencies in [rad/s], respectively. The weighted mode shape residuals $r_{\mathrm{s}}(\boldsymbol{a})$ can be expressed as

$$r_{\mathrm{s}}(\boldsymbol{a}) = \frac{\phi_j^l(\boldsymbol{a})}{\phi_j^r(\boldsymbol{a})} - \frac{\tilde{\phi}_j^l}{\tilde{\phi}_j^r}, \tag{21}$$

where l and r denote an arbitrary and a reference DOF of numerical mode shape ϕ_j (or experimental mode shape $\tilde{\phi}_j$), respectively. The value of each uncertain variable X^{e} is determined from the dimensionless updating parameter a^{e} as follows:

$$X^{\mathrm{e}} = X_{\mathrm{o}}^{\mathrm{e}}(1 - a^{\mathrm{e}}), \tag{22}$$

where $X_{\mathrm{o}}^{\mathrm{e}}$ is the initial value of the uncertain variable. The above-mentioned uncertain physical properties of the numerical model are the stiffness values in the presented FE model updating problem. Damage results in a stiffness reduction in civil engineering structures. The updated stiffness of an element of the model can be expressed as

$$\mathbf{K}^{\mathrm{e}} = \mathbf{K}_{\mathrm{o}}^{\mathrm{e}}(1 - a^{\mathrm{e}}). \tag{23}$$

The global stiffness matrix will then be:

$$\mathbf{K} = \mathbf{K}^{\mathrm{u}} + \sum_{e=1}^{n} \mathbf{K}_{\mathrm{o}}^{\mathrm{e}}(1 - a^{\mathrm{e}}), \tag{24}$$

where $\mathbf{K}_{\mathrm{o}}^{\mathrm{e}}$ and $\mathbf{K}^{\mathrm{e}}$ are the initial and updated element stiffness matrix, respectively; $\mathbf{K}$ is the global stiffness matrix and $\mathbf{K}^{\mathrm{u}}$ is the stiffness matrix of the elements whose properties remain unchanged, n is the number of elements that are updated. The gradient and the Hessian of $f(\boldsymbol{a})$ are:

$$\nabla f(\boldsymbol{a}) = \sum_{j=1}^{k} r_j(\boldsymbol{a}) \nabla r_j(\boldsymbol{a}) = \mathbf{J}_{\boldsymbol{a}}(\boldsymbol{a})^{\mathrm{T}} \mathbf{r}(\boldsymbol{a}), \tag{25}$$

$$\nabla^2 f(\boldsymbol{a}) = \mathbf{J}_{\boldsymbol{a}}(\boldsymbol{a})^{\mathrm{T}} \mathbf{J}_{\boldsymbol{a}}(\boldsymbol{a}) + \sum_{j=1}^{k} r_j(\boldsymbol{a}) \nabla^2 r_j(\boldsymbol{a}) \approx \mathbf{J}_{\boldsymbol{a}}(\boldsymbol{a})^{\mathrm{T}} \mathbf{J}_{\boldsymbol{a}}(\boldsymbol{a}), \tag{26}$$

where $\mathbf{J}_{\boldsymbol{a}}$ is the Jacobian matrix (or sensitivity matrix), which contains the partial derivatives of the residuals r_j with respect to $\boldsymbol{a}$:

$$\Delta r_j = \sum_{e=1}^{n} \frac{\delta r_j}{\delta a^{\mathrm{e}}} \Delta a^{\mathrm{e}} \tag{27}$$

or in full form:

$$[\mathbf{J}]_{m \times p} = \begin{bmatrix} \frac{\delta r_1}{\delta a_1} & \frac{\delta r_1}{\delta a_2} & \frac{\delta r_1}{\delta a_3} & \cdots & \frac{\delta r_1}{\delta a_p} \\ \frac{\delta r_2}{\delta a_1} & \frac{\delta r_2}{\delta a_2} & \frac{\delta r_2}{\delta a_3} & \cdots & \frac{\delta r_2}{\delta a_p} \\ \frac{\delta r_3}{\delta a_1} & \frac{\delta r_3}{\delta a_2} & \frac{\delta r_3}{\delta a_3} & \cdots & \frac{\delta r_3}{\delta a_p} \\ \cdots & \cdots & \cdots & \cdots & \cdots \\ \frac{\delta r_{\mathrm{m}}}{\delta a_1} & \frac{\delta r_{\mathrm{m}}}{\delta a_2} & \frac{\delta r_{\mathrm{m}}}{\delta a_3} & \cdots & \frac{\delta r_{\mathrm{m}}}{\delta a_p} \end{bmatrix}, \tag{28}$$

where $m = m_{\mathrm{f}} + m_{\mathrm{s}}$ and p is the number of the updating parameters. The first partial derivative of each frequency residual r_{f} and mode shape residual r_{s} with respect to $\boldsymbol{a}$ are:

$$\frac{\delta r_{\mathrm{f}}}{\delta a^{\mathrm{e}}} = \frac{1}{\tilde{\omega}_j^2} \frac{\delta \omega_j^2}{\delta a^{\mathrm{e}}}, \tag{29}$$

$$\frac{\delta r_{\mathrm{s}}}{\delta a^{\mathrm{e}}} = \frac{1}{\phi_j^r} \frac{\delta \phi_j^l}{\delta a^{\mathrm{e}}} - \frac{\phi_j^l}{(\phi_j^r)^2} \frac{\delta \phi_j^r}{\delta a^{\mathrm{e}}}. \tag{30}$$

4. FE model updating of a frame type structure

The natural frequencies and mode shapes are directly related to the stiffness of a structure. Therefore, a drop in natural frequencies or mode shapes will indicate a loss of stiffness which is a consequence of damage in certain elements of the structure. As the number of elements to be updated increases, the ill posedness of the model updating problem increase. Damage functions have been recommended for decreasing the number of parameters to be updated in the FE model updating of a reinforced concrete beam structure by Teughels et al. [10]. The principal idea in this methodology is that in order to prevent an ill-conditioned Jacobian matrix due to a high number of updating variables, the distribution of the unknown physical property is approximated by combining a limited set of damage functions. The updating parameters are then selected as the factors by which each of the damage functions has to be multiplied before combining them. By this methodology, not only the number of the updating parameters are reduced but also the ill posedness of the optimization problem is prevented. However, in structures with complicated geometries, the damage may have a distributed irregular pattern and the use of damage functions may not be appropriate. As mentioned above, in this study, the FE model updating problem is applied on a reinforced concrete frame structure in which, the use of the damage functions is not appropriate. In the following subsections, the reinforced concrete building type selected is explained first. The damage mechanisms of the reinforced concrete frames are then discussed followed by the description of the damage scenario applied on the building.

4.1. Criteria for the choice of the building type considered and the description of the building

The building type that is going to be studied in the present study is a 'good' quality typical existing building in the region to the north of the Marmara Sea in Turkey with a beam side-sway failure mechanism. The building is chosen such that its geometrical, material and limit state properties are within the confidence intervals of the 'good' building classification for the Marmara region.

The building considered is located in Bolu and is a moment resisting reinforced concrete frame system that represents a typical residential building in the Marmara region isolated from other buildings at both sides. The drawings of the building are taken from the curated depository of Turkish building data on the Kocaeli and Duzce earthquakes of 1999 maintained by Purdue University, University of Michigan and University of Texas at Austin [23]. The building is a four storey structure with three bays. The floor system is flat slab with beams. The building has a typical storey height of 2.85 m and considered as regular in elevation. The dimensions of the columns are typically $0.5\ \mathrm{m} \times 0.25\ \mathrm{m}$ and the dimensions of the beams are $0.25\ \mathrm{m} \times 0.5\ \mathrm{m}$ for beams. The concrete design strength is 16 MPa. The building does not have a basement and is fixed at the foundations. The building was under construction when the 1999 $M_w = 7.4$ Kocaeli and $M_w = 7.1$ Duzce earthquakes hit the region. After the 1999 earthquakes, it is reported that the reinforced concrete frame was moderately damaged.

4.2. FE model of the reinforced concrete frame

The structure is modeled by the FE package program ANSYS [24] and is idealized as a two-dimensional frame taking into account the weights of the slabs in the transverse direction in the FE model. Eight beam elements per beams and 6 beam elements for columns are generated for the FE model. The measurement locations are shown in Fig. 1.

In the first stage, in order to obtain the natural frequencies and mode shapes of the system, modal analysis is carried out by using the Block Lanczos extraction method with a sparse matrix solver. The mode shapes are normalized to unity. This stage of the analysis gives the undamaged structure's modal properties. The first two mode shapes of the frame are shown in Figs. 2 and 3.

5. Damage scenario and nonlinear dynamic time history analysis

Typically, in modeling the response of reinforced concrete structures to earthquake loading, it is assumed that

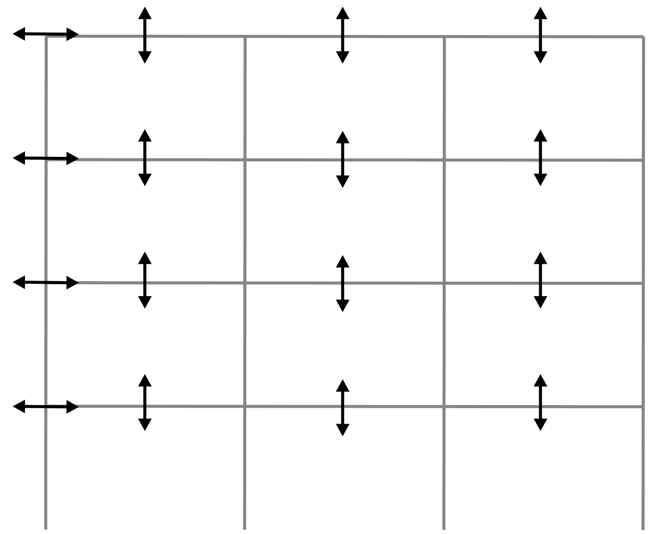

Fig. 1. The location of the response points of the reinforced concrete frame.

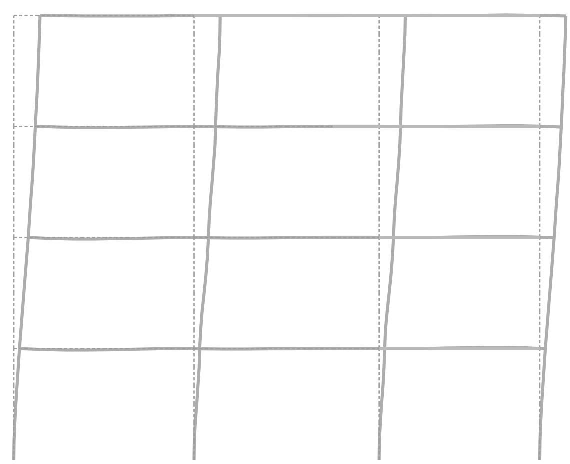

Fig. 2. The first vibration mode of the reinforced concrete frame.

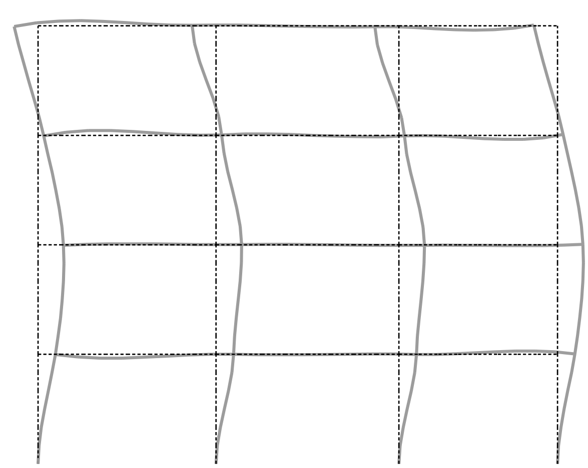

Fig. 3. The second vibration mode of the reinforced concrete frame.

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the joint regions are rigid and the damage is limited to flexural yielding of beams, columns, slabs and walls. If the beam–column joint regions are adequately designed, and if the reinforced concrete frame is designed appropriately according to the weak beam–strong column philosophy, then the plastic hinges will form in beams close to the face of the columns. On the other hand, the joints are anticipated to fail by bond slip or joint shear failure modes if the beam–column joint region does not have transverse reinforcement, the development lengths of beam bars or column depth to beam bar diameter ratio are less than the recommended values in codes and smooth bars are used for the beam longitudinal reinforcement. In this study, a damage scenario is considered such that the joint region is assumed to be adequately detailed with closely spaced stirrups and the frame is assumed to be designed according to the weak beam–strong column philosophy. Consequently, the joints are assumed to remain rigid during seismic excitations and the plastic hinges are expected to occur in the beams close to the beam–column joint regions spreading towards the point of contra flexure in the beams. Research [25] has shown that the damaged region in this case can not be represented by two springs because cracking spreads over a finite region at the ends of the reinforced concrete girders. For this type of scenario, the stiffnesses of the beam elements in the FE model close to the beam–column joint regions are decreased. It is reasonable to assume that the stiffness at this cracked region is constant. This is primarily due to the fact that the reinforcement layout will not change along the cracked zone length provided this zone does not extend beyond the quarter span point. Fig. 4 shows the elements of the reinforced concrete frame to which damage is simulated by reducing their stiffnesses.

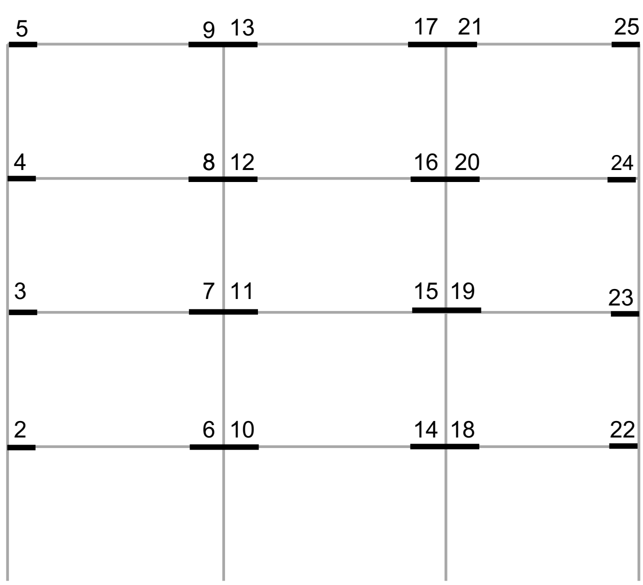

Fig. 4. The material numbers for the damaged elements in the FE model of the reinforced concrete frame. Materials numbers 2 to 25 represent the damaged elements whose stiffnesses will be updated. Material number 1 corresponds to the undamaged elements.

To assume a realistic damage scenario, nonlinear dynamic analysis is carried out. The modeling approach in the nonlinear dynamic analysis is explained in the Appendix section in detail. In order to ensure that the structure does not significantly yield, very strong ground motions should not be used for the dynamic analysis since the method proposed in this study is limited to structures which are damaged during an earthquake but that behave linear during an ambient vibration survey after the earthquake. In this study, Bolu record of the 1999 Duzce earthquake in Turkey is scaled down by a factor of 0.7 as the original record has significantly high PGA and PGV due to the forward directivity effects. For the nonlinear dynamic analysis, beam elements are modeled as nonlinear frame elements with lumped plasticity where the plastic hinges are defined at both ends of the beams. The force–deformation relationship for the plastic hinges is defined as shown in Fig. 5 according to FEMA-356 [26] and ATC-40 [27]. The performance levels of 'Immediate Occupancy (IO)', 'Life Safety (LS)' and 'Collapse Prevention (CP)', are also shown in the figure. The 'Immediate Occupancy' performance level corresponds to the post-earthquake damage state in which only very limited structural damage has occurred. The 'Life Safety' performance level corresponds to the damage state in which significant damage to the structure may have occurred but in which some margin against either total or partial structural collapse remains. The level of damage is lower than that for the 'Structural Stability' level that is represented by point C in the figure. The 'Structural Stability' performance level is the limiting post-earthquake structural damage state in which the building's structural system is on the verge of experiencing partial or total collapse. After point C, the system experiences a strength degradation and reaches point D followed by final collapse and loss of gravity load capacity at point E.

The hinge pattern of the 4 storey frame that is generated at the end of the nonlinear dynamic analysis is plotted in

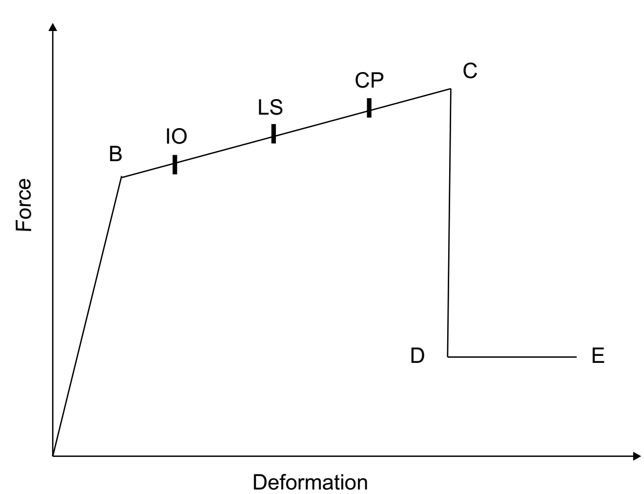

Fig. 5. Force–deformation relationship of a typical plastic hinge.

Fig. 6. The plastic hinges that have reached a deformation level between yield point B and IO level are assumed to have yielded lightly. The plastic hinges between IO and CP levels are accepted to have yielded moderately and the hinges beyond the CP level are assumed to have collapsed. Points B and C indicate the yield and ultimate curvatures, respectively.

The use of displacement-based design is becoming accepted as the logical direction for seismic design practice. The particular form known as Direct Displacement Based Design (DDBD) has been developed over the past 10 years by Priestley [28] and Priestley and Kowalsky [29] with recent specific studies by Pettinga and Priestley [30] demonstrating that it provides consistent results for reinforced concrete structural design. In this study, the stiffness reduction factors for each hinge region are calculated by evaluating the performance level obtained for each hinge in the reinforced concrete frame at the end of the dynamic analysis using the effective stiffness concept proposed by Priestley [30] where the stiffnesses of the hinge regions are represented by the equivalent secant stiffnesses at maximum rotation response reached. The stiffness reduction factors obtained in this way are given in Table 1.

In the rest of the paper, the model of the damaged frame will be referred to as the model of the simulated experiment.

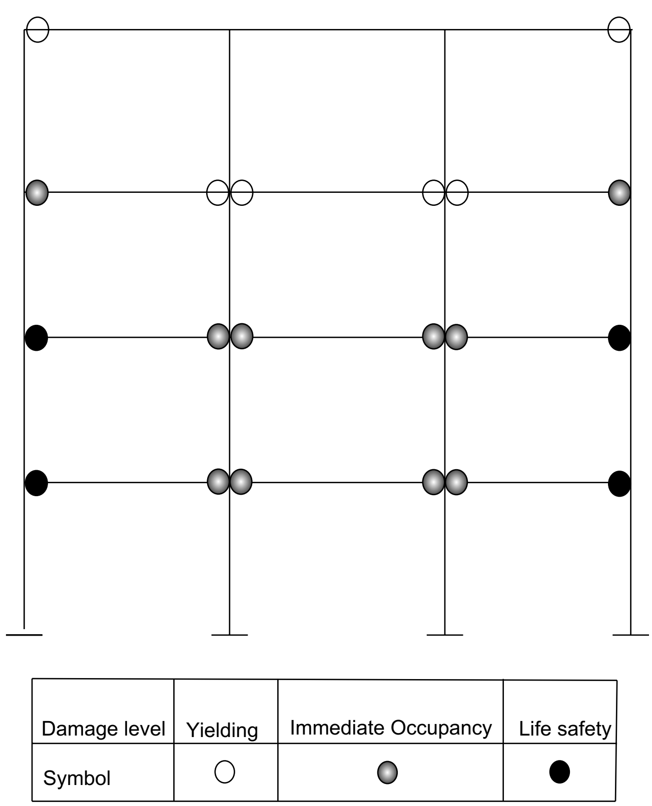

Fig. 6. The resultant plastic hinge patterns for the reinforced concrete frame structure at the end of the nonlinear dynamic analysis.

Table 1
Stiffness reduction factors for the 24 damaged elements

Material no.	Damage scenario	FEM update	Detected damage	Actual damage
	a	a	(Yes/No)	(Yes/No)
1	0	NA	NA	NA
2	0.90	0.90	Yes	Yes
3	0.80	0.80	Yes	Yes
4	0.65	0.65	Yes	Yes
5	0.30	0.30	Yes	Yes
6	0.60	0.60	Yes	Yes
7	0.45	0.45	Yes	Yes
8	0.20	0.20	Yes	Yes
9	0	0	No	No
10	0.50	0.50	Yes	Yes
11	0.55	0.54	Yes	Yes
12	0	≈0	No	No
13	0.10	0.11	Yes	Yes
14	0.55	0.55	Yes	Yes
15	0.40	0.39	Yes	Yes
16	0	≈0	No	No
17	0	≈0	No	No
18	0.65	0.65	Yes	Yes
19	0.60	0.60	Yes	Yes
20	0.10	0.10	Yes	Yes
21	0.15	0.15	Yes	Yes
22	0.85	0.85	Yes	Yes
23	0.75	0.75	Yes	Yes
24	0.60	0.60	Yes	Yes
25	0.30	0.30	Yes	Yes

6. Analysis of results

The index MAC (Modal Assurance Criterion) indicates the correlation between two sets of mode shapes [31]. MAC produces a matrix of inner products between the mode shape vectors as

$$\mathrm{MAC}(\phi_i, \tilde{\phi}_j) = \frac{|\phi_i \tilde{\phi}_j|^2}{(\phi_i^{\mathrm{T}} \phi_i)(\tilde{\phi}_j^{\mathrm{T}} \tilde{\phi}_j)}. \tag{31}$$

MAC matrix values change between 0 and 1. A MAC value close to 1 indicates a good correlation, and a MAC value close to 0 indicates a poor correlation. All the analytical modes are correlated with all the measured modes and the results are placed in a matrix. The MAC matrix is calculated for two cases. In the first case, the correlation between the initial FE model and the damaged model is investigated. In the second case, the reference FE model is updated and the correlation between the updated FE model and the damaged model is again calculated. Table 1 shows the actual and detected damage states by the model as well as the actual and predicted stiffness reduction factors. The table shows that all the damage states and the stiffness reduction factors are predicted accurately by the model. The FE model updating scheme used can accomplish the first three levels of damage identification, namely; detection, localization and quantification, successfully. Fig. 7 shows that the relative eigenfrequency differences between the numerical model and the simulated experimen-

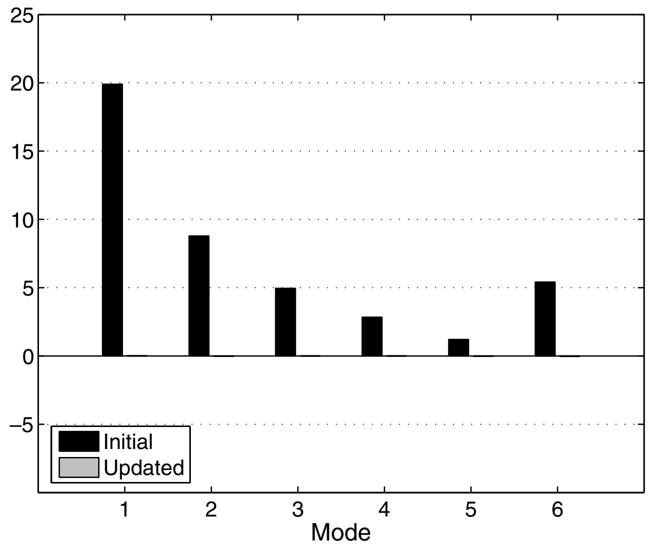

Fig. 7. Relative eigenfrequency differences $\frac{\omega-\tilde{\omega}}{\omega}$ [%] between numerical and simulated experimental modes for the reinforced concrete building using the CLM algorithm.

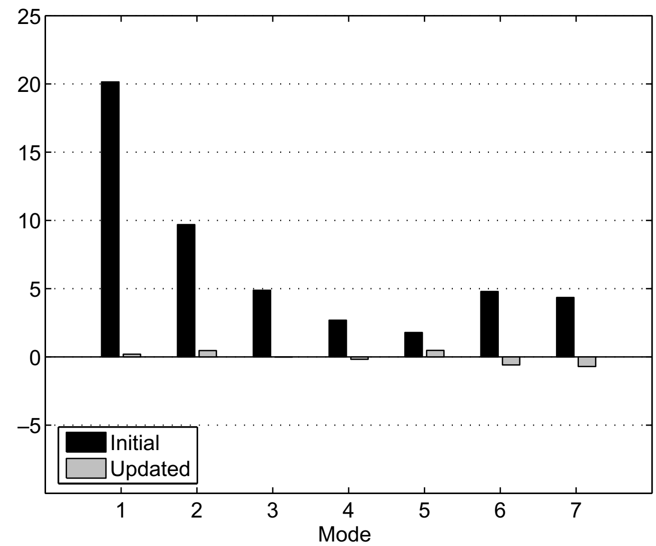

Fig. 9. Relative eigenfrequency differences $\frac{\omega-\tilde{\omega}}{\omega}$ [%] between numerical and simulated experimental modes in the presence of moderate noise using the CLM algorithm.

tal model are substantially decreased after model updating. Fig. 8 shows the comparison of the MAC values before and after model updating. It is apparent that the MAC values are also improved after FE model updating.

Next, the FE model updating scheme is tested in the presence of noise. In particular the *jth* component of the *ith* mode contaminated with noise for the *kth* measurement, ϕ_{ij}^{k}, is computed from the corresponding component of the same noise-free mode, ϕ_{ij} as

$$\phi_{ij}^{k} = \phi_{ij} * (1 + \varsigma\zeta_i^{k}), \tag{32}$$

where ς is the standard deviation; and ζ_i^k is a random number in the range $[-1, 1]$. For the frequencies contaminated with noise, the same definition has been used. Two noise levels are considered to simulate measurement errors. The first is moderate noise level in which ς is 0.5% and 2% for the eigenfrequencies and mode shapes, respectively. The second noise level simulates substantially noisy measurements. In the high noise level, ς is 3% and 10% for the eigenfrequencies and mode shapes, respectively.

Figs. 9 and 10 show the relative eigenfrequency differences and the MAC values between the numerical and the simulated experimental modes in the presence of moderate noise, respectively. The results show that the relative differences in the eigenfrequencies and the MAC values are considerably improved after the FE model updating. Table 2 shows the detected and actual damage as well as the pre-

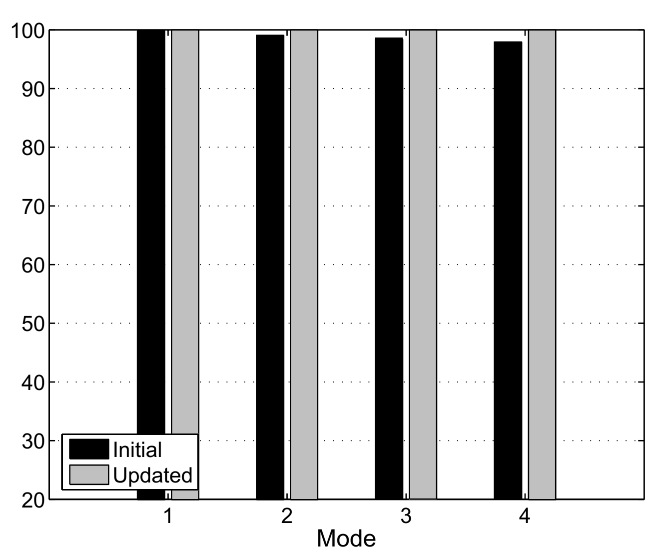

Fig. 8. The comparison of MAC values before and after model updating for the first four modes using the CLM algorithm.

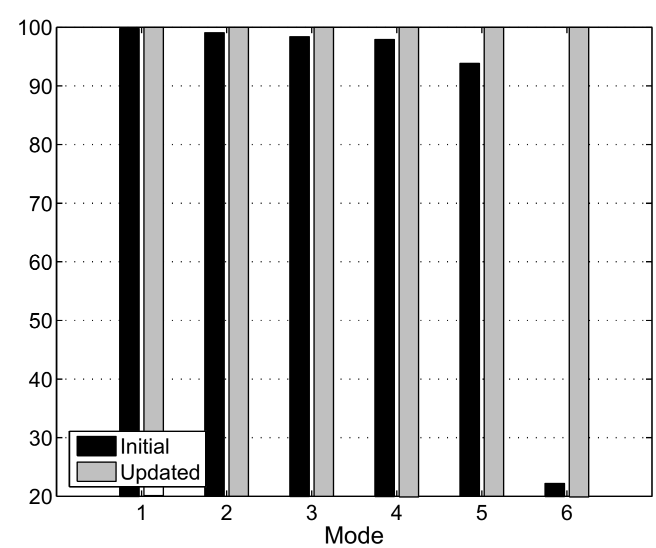

Fig. 10. MAC values between numerical and simulated experimental modes for the reinforced concrete building in the presence of moderate noise using the CLM algorithm.

Table 2
Stiffness reduction factors predicted for the 24 damaged elements in the presence of random noise with normal distribution and 0.5% standard deviation applied to the eigenvalues; 2% standard deviation relative to the maximum amplitude applied to the mode shapes

Material no.	Damage + noise	FEM update	Detected damage	Actual damage
	a	*a*	Yes/No	Yes/No
1	0	NA	NA	NA
2	0.90	0.90	Yes	Yes
3	0.80	0.79	Yes	Yes
4	0.65	0.66	Yes	Yes
5	0.30	0.30	Yes	Yes
6	0.60	0.59	Yes	Yes
7	0.45	0.41	Yes	Yes
8	0.20	0.31	Yes	Yes
9	0	≈0	No	No
10	0.50	0.45	Yes	Yes
11	0.55	0.57	Yes	Yes
12	0	≈0	No	No
13	0.10	0.09	Yes	Yes
14	0.55	0.55	Yes	Yes
15	0.40	0.45	Yes	Yes
16	0	≈0	No	No
17	0	≈0	No	No
18	0.65	0.67	Yes	Yes
19	0.60	0.57	Yes	Yes
20	0.10	0.17	Yes	Yes
21	0.15	0.17	Yes	Yes
22	0.85	0.85	Yes	Yes
23	0.75	0.74	Yes	Yes
24	0.60	0.63	Yes	Yes
25	0.30	0.29	Yes	Yes

Table 3
Stiffness reduction factors predicted for the 24 damaged elements in the presence of random noise with normal distribution and 3% standard deviation applied to the eigenvalues; 10% standard deviation relative to the maximum amplitude applied to the mode shapes

Material no.	Damage + noise	FEM update	Detected damage	Actual damage
	a	*a*	(yes/no)	(yes/no)
1	0	NA	NA	NA
2	0.90	0.90	Yes	Yes
3	0.80	0.80	Yes	Yes
4	0.65	0.64	Yes	Yes
5	0.30	0.39	Yes	Yes
6	0.60	0.62	Yes	Yes
7	0.45	0.52	Yes	Yes
8	0.20	0.25	Yes	Yes
9	0	≈0	No	No
10	0.50	0.50	Yes	Yes
11	0.55	0.52	Yes	Yes
12	0	≈0	No	No
13	0.10	0.02	Yes	Yes
14	0.55	0.51	Yes	Yes
15	0.40	0.40	Yes	Yes
16	0	≈0	No	No
17	0	≈0	No	No
18	0.65	0.68	Yes	Yes
19	0.60	0.63	Yes	Yes
20	0.10	0.18	Yes	Yes
21	0.15	0.22	Yes	Yes
22	0.85	0.86	Yes	Yes
23	0.75	0.80	Yes	Yes
24	0.60	0.60	Yes	Yes
25	0.30	0.50	Yes	Yes

dicted stiffness reduction factors in the presence of noise. It is apparent that the first three levels of damage identification are again possible with the FE model updating scheme in the presence of moderate noise.

The second case considered is the substantial noise in measurements. Table 3 shows the stiffness reduction factors predicted. The results show that in the presence of substantial amount of noise, the model can very slightly under or overestimate the stiffness reduction factors, but 22 out of 24 damage parameters are predicted accurately again even for the complex damage pattern adopted. Figs. 11 and 12 also show that the MAC values as well as the relative eigenfrequency differences are improved substantially after FE model updating.

7. Comparison of the coupled local minimizers with other optimization techniques

In this part of the study, the 'Coupled Local Minimizers' technique used for FE model updating is compared with other optimization techniques in terms of accuracy, condition number of the Jacobian and the CPU time. Relative eigenfrequency and relative mode shape differences have established and widespread use in the literature to estimate the accuracy of the updating process, e.g. Refs. [20,32]. Just as implemented by Teughels [20] and Modak et al. [32], relative frequency and mode shape difference indices are used in this study in which percentage average error in natural frequencies (IAENF) and percentage average error in mode shapes (IAEMS) are calculated as:

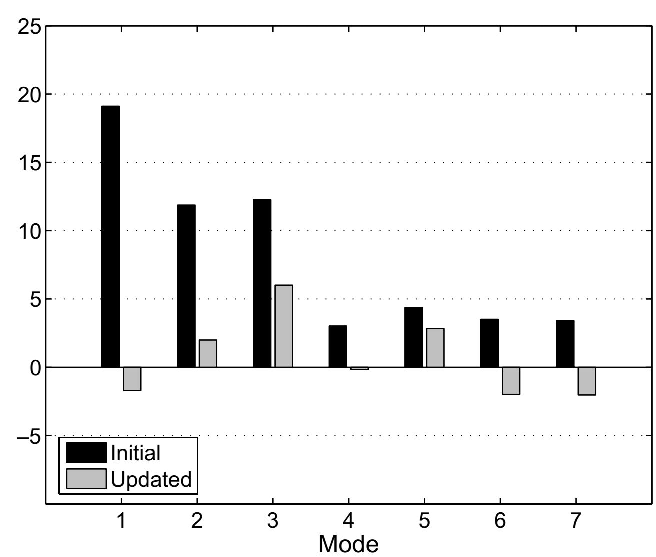

Fig. 11. Relative eigenfrequency differences $\frac{\omega-\tilde{\omega}}{\omega}$[%] between numerical and simulated experimental modes for the reinforced concrete building in the presence of substantial noise using the CLM algorithm.

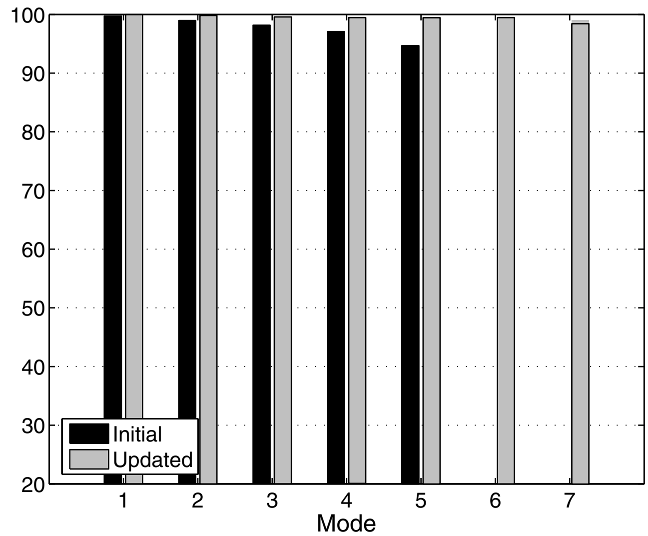

Fig. 12. MAC values between numerical and simulated experimental modes for the reinforced concrete building in the presence of substantial noise using the CLM algorithm.

$$\mathrm{IAENF} = \frac{100}{m_{\mathrm{f}}} \sum_{i=1}^{m_{\mathrm{f}}} \mathrm{abs}\left(\frac{\omega_{\mathrm{u}} - \tilde{\omega}}{\tilde{\omega}}\right) \tag{33}$$

$$\mathrm{IAEMS} = \frac{100}{m_{\mathrm{m}} N} \sum_{i=1}^{m_{\mathrm{m}}} \sum_{j=1}^{N} \mathrm{abs}\left(\frac{\phi^j_{i\,\mathrm{u}} - \tilde{\phi}^j_i}{\tilde{\phi}^j_i}\right), \tag{34}$$

where ω_{u} and ϕ_{u} are the updated natural frequencies and mode shapes, respectively and N is the number of measured degrees of freedom.

For comparison purposes, the FE model updating technique is applied with different optimization methods such as the Gauss–Newton method, Levenberg–Marquardt algorithm and Sequential Quadratic Programming. In the following subsections, these methods are briefly explained and the results of the comparative study are discussed. The error indices obtained from different optimization routines are summarized in Table 4 and the condition numbers, number of iterations, CPU time for a computer with an Intel-Core 2 Duo processor are compared in Table 5.

Table 4
Error indices before and after model updating for different number of modes

Optimization method	IAENF (BU) (%)	IAENF (AU) (%)	IAEMS (BU) (%)	IAEMS (AU) (%)
CLM	7.18	0.01	150.12	1.47
CLM (MN*)	6.9	0.37	185.18	3.56
CLM (SN*)	7.49	2.72	208.76	17.54
SQP*	7.18	NI*	150.12	NI*
LM*	7.18	3.08	150.14	55.47
GN*	7.18	NI*	150.14	NI*

Note:∗ NI means that no improvement in error index is achieved, MN is moderate noise, SN is substantial noise, LM is the Levenberg–Marquardt algorithm, GN is the Gauss–Newton algorithm, SQP is the Sequential quadratic programming method.

Table 5
Number of iterations, CPU time and the condition number of the Jacobian for the model updating algorithm for different number of modes

Optimization method	Iteration no.	CN* of Jacobian	CPU [s]
CLM	8	137.2	911
CLM (MN*)	8	181.24	1016
CLM (SN*)	23	182.675	3330
SQP*	4	–	13,177
LM*	2	204.39	52.82
GN*	2	61.24	33.91

Notes: ∗ CN is the condition number, MN is moderate noise, SN is substantial noise, LM is the Levenberg–Marquardt algorithm, GN is the Gauss–Newton algorithm, SQP is the Sequential quadratic programming method.

7.1. Gauss–Newton method

Gauss–Newton method is a modification of Newton's method with line search. In Newton method, the search direction p_k is generated by solving the standard equation:

$$\nabla^2 f(x_k) p = -\nabla f(x_k). \tag{35}$$

In Gauss–Newton method, the second term in Eq. (26) is excluded and the solution p_k^{GN} is obtained by solving:

$$J_k^{\mathrm{T}} J_k p_k^{GN} = -J_k^{\mathrm{T}} r_k. \tag{36}$$

In the Gauss–Newton method, a line search is performed along the direction p_k^{GN}.

$$\mathbf{x}_{k+1} = \mathbf{x}_k + \alpha_k \mathbf{p}_k^{GN}. \tag{37}$$

The Gauss–Newton search direction is calculated with a QR decomposition of $\mathbf{J}_k$. The step length α can be determined using the Wolfe sufficient decrease and curvature conditions given below:

$$f(\mathbf{x}_k + \alpha_k \mathbf{p}_k) \leqslant f(\mathbf{x}_k) + c_1 \alpha_k \nabla f_k^{\mathrm{T}} \mathbf{p}_k \tag{38}$$

$$\nabla f(\mathbf{x}_k + \alpha_k \mathbf{p}_k)^{\mathrm{T}} \mathbf{p}_k \geqslant c_2 \nabla f_k^{\mathrm{T}} \mathbf{p}_k \tag{39}$$

with $0 < c_1 < c_2 < 1$.

The Gauss–Newton algorithm is applied on the FE model updating problem for comparison purposes. It is apparent from the results that no improvement is achieved in the MAC values and the eigenfrequency errors after updating. The comparison of the updated stiffness reduction factors and the reduction factors from the damage scenario shows that only 9 out of 24 stiffness reduction factors are predicted within reasonable accuracy using the Gauss–Newton algorithm.

7.2. Sequential quadratic programming

The Sequential Quadratic Programming (SQP) methods are known to be powerful when solving problems with significant nonlinearities. A quadratic programming subproblem is solved at each iteration which can be expressed as

$$\min_{\mathbf{p}} q(\mathbf{p}) = \nabla f^{\mathrm{T}} \mathbf{p} + \frac{1}{2} \mathbf{p}^{\mathrm{T}} [\nabla^2_{xx} \mathscr{L}] \mathbf{p}, \tag{40}$$

where $\mathscr{L}$ is the Lagrange function defined by Eq. (3). The quadratic programming subproblems are solved with an active set method which starts with an initial guess of the optimal active set A^*, which is also called the working set W and which repeatedly adapt the current working set by dropping and adding constraints to the set. In the Sequential Quadratic Programming, a new step length $\mathbf{p}_k$ and new Lagrange multiplier estimates $\boldsymbol{\lambda}_{k+1}$ are obtained from Eq. (40). The iterations are stopped when the algorithm converges and the optimum minimizer x^* and Lagrange multipliers λ^* of the subproblem are determined. A line search strategy is pursued in the sense that the solution of the quadratic programming subproblem produces a vector $\mathbf{p}_k$, which is used to generate a new iterate:

$$\mathbf{x}_{k+1} = \mathbf{x}_k + \alpha_k \mathbf{p}_k^{\mathrm{SQP}}, \tag{41}$$

where α is the step length parameter which is determined in order to result in a sufficient decrease in a merit function $\Psi(x)$ expressed as

$$\Psi(\mathbf{x}) = f(\mathbf{x}) + \sum_{j\epsilon\varepsilon} \lambda_j |h_j(\mathbf{x})| + \sum_{j\epsilon I} \gamma_j \max(h_j(\mathbf{x}), 0), \tag{42}$$

where $\gamma_j > 0$ are penalty parameters. The line search algorithm requires that the Hessian of the Lagrangian function B, is calculated using the Broyden–Fletcher–Goldfarb–Shanno (BFGS) formula:

$$B_{k+1} = B_k + \frac{q_k q_k^{\mathrm{T}}}{q_k^{\mathrm{T}} s_k} - \frac{B_k^{\mathrm{T}} B_k}{s_k^{\mathrm{T}} B_k s_k}, \tag{43}$$

where

$$s_k = x_{k+1} - x_k, \tag{44}$$

$$q_k = \nabla f(x_{k+1}) + \sum_{i=1}^{n} \lambda_i \nabla h_i(x_{k+1}) - \left(\nabla f(x_k) + \sum_{i=1}^{n} \lambda_i \nabla h_i(x_k) \right). \tag{45}$$

The FE model updating problem is applied with the Sequential Quadratic Programming (SQP) method. It is apparent from the results that no improvement in percentage average errors in frequencies and mode shapes is achieved by the implementation of the SQP algorithm. The CPU time was the highest for this algorithm and the stiffness reduction factors could not be predicted accurately.

7.3. Levenberg–Marquardt algorithm

The Levenberg–Marquardt algorithm is applied with a line search strategy where the search direction is calculated by solving:

$$(\mathbf{J}_k^{\mathrm{T}} \mathbf{J}_k + \mu_k \mathbf{I})\mathbf{p}^{\mathrm{LM}} = -\mathbf{J}_k^{\mathrm{T}} \mathbf{r}_k, \tag{46}$$

where $\mu_k \geqslant 0$ is the parameter which limits the size of $\mathbf{p}$. It should be noted that when this parameter has a value equal to zero, then the solution to the Levenberg–Marquardt algorithm is identical with the solution to the Gauss–Newton algorithm. On the other hand, if the parameter $\mu_k \to \infty$, then $\|\mathbf{p}_k\| \to 0$ and $\mathbf{p}_k$ approaches the steepest descent direction $\mathbf{J}_k^{\mathrm{T}} \mathbf{r}_k$. Therefore, the Levenberg–Marquardt direction is between the steepest descent and Gauss–Newton directions depending on the value of μ.

For comparison purposes, the Levenberg–Marquardt algorithm is applied on the FE model updating problem considered. The algorithm terminated abruptly after just 2 iterations as no improvement in the search direction is achieved. Fig. 13 shows that the eigenfrequency errors decreased after updating for all modes. As shown in Fig. 14, the MAC values also improved for all modes after model updating. Nevertheless, if the MAC values and the eigenfrequency errors obtained from the CLM method in Figs. 7 and 8 are compared with the MAC and eigenfre-

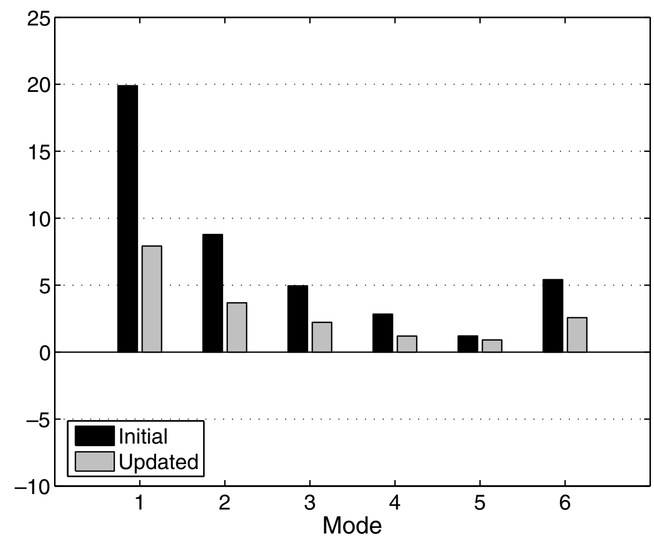

Fig. 13. Relative eigenfrequency differences $\frac{\omega-\tilde{\omega}}{\omega}$ [%] between the numerical and simulated experimental modes for the reinforced concrete building using the Levenberg–Marquardt algorithm.

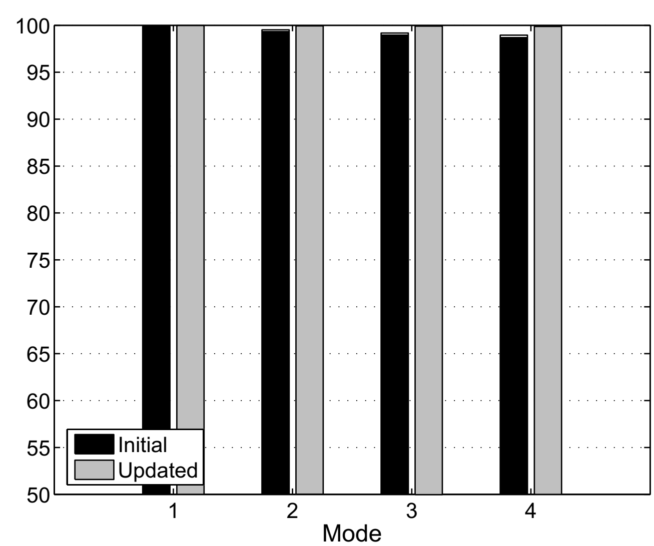

Fig. 14. The comparison of MAC values before and after model updating for the first four modes using the Levenberg–Marquardt algorithm.

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quency error values in Figs. 13 and 14 obtained from the Levenberg–Marquardt algorithm, it becomes apparent that the CLM method gives better results than the Levenberg–Marquardt algorithm in FE model updating problems. Moreover, although the CPU time is shorter, only 3 stiffness reduction factors out of 24 are predicted within reasonable accuracy using the Levenberg–Marquardt algorithm.

7.4. Discussion of the results of the comparative study

The analysis of the results show that the CLM method always gives better results than the other optimization methods. Table 4 shows that the percentage average error in mode shapes and natural frequencies are substantially lower using the CLM algorithm in comparison to the Levenberg–Marquardt, Gauss–Newton and Sequential Quadratic Programming algorithms. In this study, the CLM method is applied using an Augmented Lagrangian that is implemented with a trust region Newton method to improve convergence. The trust region strategy adopted prevents the iterates from taking very large steps and consequently avoids divergence of the process. As a result, the optimization process gives better results than the Levenberg–Marquardt, Gauss–Newton and the Sequential Quadratic Programming algorithms as shown in Tables 4 and 5. Secondly, the trust region strategy adopted within the CLM algorithm acts as a regularization technique. There are also two reasons for this. First, the radius of the trust region is updated in each iteration according to the accuracy of the approximating model function. This strategy, reduces oscillations in the design variables and results in a more robust optimization method. Second, in addition to the application of a trust region, explicit bound constraints are introduced into the optimization procedure which prevents overshooting and improves stability.

The second most important advantage of the global optimization method CLM over the local optimization methods compared is that the CLM does not get stuck in the local minima. This advantage becomes more pronounced especially when working with noisy data. In this case, the CLM algorithm always gives superior results than other local optimization techniques.

It is true that the convergence speed of the CLM method will be lower than the other algorithms due to the fact that in global optimization methods such as the CLM method, a population is used for the initial starting values of a design variable as opposed to the local optimization methods where a single initial value is required. However, since robustness is the major issue to be considered in ill-conditioned FE model updating problems, the convergence speed may be considered as rather secondary.

8. Remarks for application of the technique on actual structures

This is an ongoing research and this phase of the research is a feasibility study for the application of the 'Coupled Local Minimisers' method in FE model updating. In the next phase of the study, the method will be applied on actual structures. Although the above numerical examples have demonstrated that FE model updating using 'Coupled Local Minimizers' has the capability to efficiently identify, locate and quantify damage in frame type structures, further issues have to be taken into consideration in the application of the technique on actual structures. In a complex structure, if the initial analytical model neglects some important effects that exist in the real structure, it would be very difficult to find the real model by updating parameters of the inadequate analytical model alone. The following further issues have to be considered in the application of the algorithm on real structures.

First, in FE model updating of an actual structure, it is immensely important that a good model of the structure should be used as reference to reduce the modeling errors to an acceptable level. Modeling errors occur due to unavoidable uncertainties that are related to modeling of material properties, effects of non-structural elements, support conditions, less accurate element types used in the FE model such as the use of the Euler–Bernoulli beam elements instead of the Timoshenko beam elements that include a parameter to model shear strain. In this study, shear deformations are also taken into account in all members in the FE models in order to more accurately model the behaviour of actual structures.

Another important parameter to be considered is the mesh size of the initial FE model. Using a fine mesh leads to more accurate results. Especially in cases where a high number of modes are used for the updating problem, a coarse mesh will lose accuracy as greater spatial resolution is required to precisely detect the higher and more complex mode shapes and corresponding frequencies.

Equivalently important is to be able to predict the possible damage locations for the structure type that will be updated. If the number of the model parameters that have to be updated are too large, numerical difficulties will arise in the updating problem. To avoid these difficulties and to realistically reduce the number of the updating parameters, it is vital to predict a good damage scenario in the FE model updating of actual structures. For instance, in well confined building type structures designed according to the weak beam–strong column philosophy, plastic hinges should be expected in beams adjacent to the columns. Therefore, the damage scenario applied in this paper for well confined buildings can be considered as realistic due to our experiences from past earthquakes. But for structures with unconfined beam–column joint regions, damage is also expected to occur in the beam–column joint regions and different model parameters should be selected for updating in the initial FE model for that case.

Environmental effects should also be considered in damage identification of real structures. It is clear that there is an influence of temperature on the eigenfrequencies of a structure [33]. Nevertheless, this has to be taken into account in the system identification phase of a damage identification campaign when the eigenfrequencies and eigenmodes are determined from the measured response data in the structure. Peeters [34] has proposed a method for detecting damage in the presence of varying environmental parameters such as temperature.

9. Conclusions

A FE model updating scheme using a new global optimization method called 'Coupled Local Minimizers' is applied on a numerical model of an actual residential building from Turkey that had been subjected to the 1999 Kocaeli and Duzce earthquakes. In this study, the CLM algorithm is implemented with a Trust Region Newton method. In CLM, the local minimizers are forced to communicate and exchange information using synchronization constraints. A damage scenario is applied on the reinforced concrete frame such that the stiffnesses of the elements close to the beam–column joints are deliberately decreased to simulate damage. A complex worst case damage scenario in which adjacent elements have substantially different stiffness levels is considered. The damage scenario is based on a nonlinear dynamic time history analysis where the reductions in stiffness values are determined by evaluating the performance level of each hinge according to the 'Performance Based Design' philosophy adopted in ATC 40. The initial FE model is then updated to tune the initial modal parameters with the modal parameters from the numerical model in which damage is simulated. The results of the updating showed that the first three levels of damage identification as proposed by Rytter; namely, damage detection, localization and quantification are successfully accomplished by the FE model updating algorithm used. The relative eigenfrequency differences and the MAC values are substantially improved after updating. The FE model updating algorithm is also tested in the presence of two noise levels which simulate moderate and substantial noise in measurements, respectively. In the presence of moderate noise levels, damage is successfully detected and located in all elements. With the exception of one element, all the 23 stiffness reduction factors are predicted quite accurately. The relative eigenfrequency differences and the MAC values are considerably improved after model updating. In the presence of high amount of noise, damage is detected and located correctly in all the damaged elements. The extent of damage is over or underestimated in two elements alone. The majority of the stiffness reduction factors (22 out of 24) are predicted within reasonable accuracy. The CLM method is compared with the Sequential Quadratic Programming, Gauss–Newton and Levenberg–Marquardt algorithms. It is apparent that the FE model updating with the CLM method detects, locates and quantifies damage more accurately than the other optimization methods. Better MAC values and the relative eigenfrequency differences are obtained using the CLM method. The results demonstrate that FE model updating using Coupled Local Minimizers is promising for the detection of damaged elements in actual multistorey buildings.

Acknowledgements

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Appendix A. Modeling approach in nonlinear dynamic analysis

In this study, a moment-curvature analysis is carried out for each element where point B is obtained using approximate component initial effective stiffness values according to ATC-40 and a recent study on defining plastic hinge properties in nonlinear analysis [35]. The initial stiffness value adopted is 0.4 EI for beams as proposed by the new Turkish Earthquake Design Code [36]. The ultimate curvature is defined as the curvature corresponding to the extreme compression fiber reaching the ultimate concrete compressive strain according to the following relation proposed by Priestley et al. [37]:

$$\varepsilon_{cu} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f_{cc}}, \tag{A.1}$$

where ε_{cu} is the ultimate concrete compressive strain, ε_{su} is the steel strain at the maximum tensile stress, ρ_s is the volumetric ratio of confining steel, f_{yh} is the yield strength of transverse reinforcement, and f_{cc} is the peak confined concrete compressive strength. The IO, LS, CP, and C points in Fig. 5 are assumed be reached at 12.5%, 40%, 55%, and 63% of the ultimate rotation capacity, respectively in accordance with the ultimate rotation and moment values obtained from a moment-curvature analysis as well as ATC-40. On the beams, the axial forces are assumed to be zero. Plastic hinge length is used to calculate the rotation values from curvatures. The maximum of the two plastic hinge lengths as given below are considered in the analysis. The first of these equations is proposed in the new Turkish Earthquake Resistant Design Code [36]. The second has been proposed by Priestley [37].

$$L_p = 0.5H, \tag{A.2}$$

$$L_p = 0.08L + 0.022 f_{ye} d_{bl} \geqslant 0.044 f_{ye} d_{bl}, \tag{A.3}$$

where L_p is the plastic hinge length, H is the section depth, L is the critical distance from the critical section of the plastic hinge to the point of contraflexure, and f_{ye} and d_{bl} are the expected yield strength and the diameter of the longitudinal reinforcement, respectively.

Since an existing reinforced concrete building is investigated in this study, shear hinges are also introduced at both ends of beams. The shear strength of each member is computed according to TS500 [38] as shown below:

$$V_r = V_c + V_s, \tag{A.4}$$

$$V_c = 0.182bd\sqrt{f_c}\left(1 + 0.07\frac{N}{A_c}\right), \tag{A.5}$$

$$V_s = \frac{A_{sh} f_{yh} d}{s}, \tag{A.6}$$

where V_c and V_s are the concrete and transverse reinforcement, respectively, b is the section width, d is the effective depth, f_c is the unconfined concrete compressive strength, N is the axial load on the section, A_{sh} is the area of the transverse reinforcement, f_{yh} is the yield strength of the transverse reinforcement and s is the spacing of the transverse reinforcement.

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