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Damage detection of frames using the increment of lateral displacement change

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Abstract: The method proposed in this paper is based on the fact that the damage in different types of structural members has distinctive influence on the structural stiffness. The intrinsic mechanical property of the structure is tapped and fully utilized for damage detection. The simplified model of the flexibility of frames treats the individual storeys as springs in series and the frame as an equivalent column. It fully considers the main deformation of all beams and columns in the frame. The deformation property of the simplified model accorded well with that of the actual frame model. The obtained increment of lateral displacement change (IOLDC) at the storey level was found to be very sensitive to the local damage in the frame. A damage detection method is proposed using the IOLDCs as the damage identification parameters. Numerical examples demonstrate the potential applicability of this method.

Key words: Damage detection, Increment of lateral displacement change, Static test, Frame, Equivalent column
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INTRODUCTION

Damage detection of structures is very important for ensuring and evaluating the safety of structure systems during their lifetime. The approaches developed in this field may be generally classified into dynamic identification approach using dynamic test data and static identification approach using static test data (Wang *et al*., 2001). The dynamic identification approach has been highly developed, although several inherent drawbacks and problems handicap the full utilization of this approach: (1) The necessary excitation response is generally difficult to be obtained satisfactorily, especially for practical structures in the field; (2) The pollution of the measurement noise greatly affects the precision of test data, which is more serious in the high-order modal data, such as the strain mode, curvature mode; (3) Damage in structures will cause changes in their stiffness, mass and damping. In some dynamic identification methods, it is assumed that there is no damping or that the damage does not cause changes of the mass and/or damping. How to make the analytical method reasonably applicable to actual damage detection is a key problem; (4) The change in dynamic response of the structure is generally a macro reflection of the damage situation, and is insensitive to the local element damage.

Compared with dynamic identification methods, the static identification methods are usually simple and easily implemented. The static identification method has practical value, particularly in the laboratory where static test data can be obtained with relatively high accuracy. The problems in the static identification methods are that the usable information is relatively less, and that the identification result is more influenced by the selected loading cases.

In both the dynamic and static identification methods, the incompleteness of the test data and the finiteness of the obtainable damage information are two great obstacles. Various model condensation techniques were developed (Guyan, 1965; Koh *et al*.,

1995) to resolve the incompleteness of the test data. For many large or complex structure systems, various simplified models were developed, e.g., lumped-mass models or shear-building models. A problem arising from the oversimplification of models is that the change in stiffness induced by the local damage tends to "spread out" or "diffuse" into other adjacent elements. This stiffness diffusion problem makes the damage detection uncertain and inaccurate (Koh *et al*., 1995; Natke and Yao, 1988). By introducing an improved condensation method to identify the local damage in multi-storey frame buildings, Koh *et al*. presented a method that can identify the stiffness reduction at the storey level only due to the column damage, in which, the beam was assumed to be rigid. To correlate well the experiment model with the assumed situation, the floor stiffness of the model was intensified by brace bars.

In reality, beams in many moment-resisting frame-type structures are not rigid. The deformation and possible damage in beams are existent. The assumption of rigid beams will undoubtedly substantially overestimate the frame stiffness. To make the theoretical analysis have practical application meaning, the simulation model should be possibly correlated with real structures. The model cannot be oversimplified, and the main deformation of all members in structures should be considered.

Many advanced computing methods have been developed for structural damage detection as an inverse problem, e.g. genetic algorithm (Chou and Ghaboussi, 2001), artificial neural network (Pandey and Barai, 1995; Yun and Bahng, 2000; Waszczyszyn and Ziemiański, 2001; Ko *et al*., 2002) and various minimization optimization algorithms (Koh *et al*., 1995; Hjelmstad and Shin, 1997). There are several methods for solving the damage detection problems only through purely numerical optimization algorithms. These methods exhaustively searched the damaged members from all members in the structure solely according to the errors between the calculated and measured results. This scheme will work ineffectively as damage cases are complicated and the structure scale is large and complex. In fact, to tap and utilize the intrinsic mechanical characteristics of the structure is most essential, whichever technique or method is developed for damage detection.

Although the deterioration in strength or the accumulation of plastic strain is sensitive to damage, the changes in these parameters are mainly concentrated on the local damaged members. The measurement of these parameters is relatively difficult and inconvenient, especially for large structure systems. With respect to these parameters, the change in the displacement of the structure due to the damage is relatively easily to be measured, and it directly reflects the change in the structural flexibility. Therefore, to analyze the flexibility change for damage detection has more practical meaning.

According to the above-mentioned analysis, the authors fully considered the main deformation of beams together with that of columns when formulating the lateral flexibility of multi-storey frame structures. By reasonably simplifying the model and formulating the lateral flexibility of the frame, the obtained increment of lateral displacement change (IOLDC) at the storey level was found to be very sensitive to the local damage. At the same time, the rules on the damage in different types of structural members has its distinctive influence on the IOLDCs of the structure were obtained. At last, a damage detection method is proposed. The distinguishing characteristic of the method is that, it can effectively identify the damage in different members (e.g. the column and/or beam) at the storey level in the frame structure. Even though for complicated damage situations with many damaged members, using the IOLDCs and the distinctive rules of the damage in different member types on the IOLDCs can approximately identify the damage at the storey level.

FLEXIBILITY FORMULATION OF FRAME STRUCTURES

The method simplifying the multi-storey frame structure model and formulating the lateral flexibility used for damage detection is based on the work of Sameer and Jain (1992; 1994), Dutta *et al*.(2000a; 2000b). To facilitate introduction, a one-bay, multi-storey standard plane frame model was adopted, as shown in Fig.1. All beams and columns are similar and have identical cross-section dimensions, respectively. All storey heights are equal. The formulation analysis is based on the following assumptions (Dutta *et al*., 2000a; 2000b):

1. The structure behavior is elastic and linear;

2. Center-line dimensions are used, neglecting the stiffness contribution of finite-sized joint zones at the junctions of beams and columns;

3. The columns in the bottom storey are fixed against translation and rotation on the base; other nodes are free;

4. The shear deformation in the columns is negligible, and the lateral displacement due to the axial deformation in columns is not considered.

To formulate the lateral flexibility of the frame, the frame is first divided into individual storeys, and the flexibility of the individual storey is analyzed. The analytical unit is isolated by making cuts at each beam. For generalization of analysis, the isolated analytical unit model in Green (1978) was used and shown in Fig.2. It is assumed that the points of inflection for all beams and columns except columns in the bottom storey occur at the mid span; and that the location of inflection points of columns in the bottom storey is determined by Eq.(40) in Sameer and Jain (1994). The lateral deformation of a column consists of two parts: one is due to the rotation of the top and bottom ends of the column, and the other is the deflection of the column acting as a cantilever supported at the joints (Sameer and Jain, 1992), as shown in Fig.2.

Lateral flexibility of intermediate storeys

For the column in the intermediate storeys, the lateral deflection can be expressed as in Sameer and Jain (1992):

$$\Delta_c^m = \Delta_{b1} + \Delta_{b2} + \Delta_{t1} + \Delta_{t2} = 2\frac{V\left(\frac{h}{2}\right)^3}{3E_cI_c} + \frac{\theta_t h}{2} + \frac{\theta_b h}{2} \quad (1)$$

where Δ_c^m=lateral relative displacement between the ends of the column; E_c=Young's modulus of the column; I_c=the inertia moment of the column; V=shear force applied on the column; h=the storey height; θ_t, θ_b=rotation angles of top and bottom ends respectively.

$$\theta_t = \frac{V(h_a + h)}{2K_{\theta_t}},\ \theta_b = \frac{V(h_b + h)}{2K_{\theta_b}} \quad (2)$$

in which, h_a, h_b=heights of storeys above and below the current storey respectively; K_{θ_t}, K_{θ_b} = rotation stiffness of top and bottom ends respectively.

$$K_{\theta_t} = \sum_1^{N_b} \frac{6E_bI_{bt}}{L} = 6\sum K_{bt}$$
$$K_{\theta_b} = \sum_1^{N_b} \frac{6E_bI_{bb}}{L} = 6\sum K_{bb} \quad (3)$$

here, L=length of the beam; E_b=Young's modulus of the beam; N_b=number of beams meeting the column at the end.

$$\sum K_{bt} = \sum_{i=1}^{N_b} \frac{E_b^i I_{bt}^i}{L},\ \sum K_{bb} = \sum_{i=1}^{N_b} \frac{E_b^i I_{bb}^i}{L} \quad (4)$$

I_{bt}, I_{bb}=inertia moments of the beams meeting the column at the top and bottom ends, respectively. Substituting Eqs.(2)–(4) into Eq.(1) yields

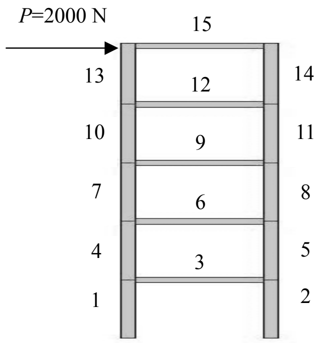

Fig.1 Plane frame model

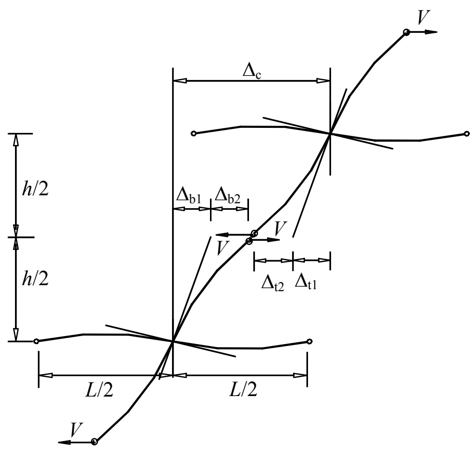

Fig.2 Isolated unit of intermediate storeys for analyzing deformation due to bending of beams and columns

$$\Delta_c^m = \frac{Vh^3}{12E_cI_c} + \frac{V(h_a+h)h}{24\sum K_{bt}} + \frac{V(h_b+h)h}{24\sum K_{bb}} \quad (5)$$

For the plane frame, the rotation stiffness of the top and bottom ends of the column is the contribution of the bending stiffness of beams lying in the bending plane of the frame and meeting the end. It is assumed that the axial deformation of the beams is negligible. The compatibility requires equal lateral displacement of all columns in the same storey.

The lateral stiffness of a single column in the intermediate storey of the frame is expressed as

$$K_{col}^m = \frac{V}{\Delta_c^m} = \frac{12E_cI_c}{h^3}\frac{1}{1+0.5\frac{E_cI_c}{h^2}\left(\frac{h_a+h}{\sum K_{bt}}+\frac{h_b+h}{\sum K_{bb}}\right)} \quad (6)$$

For standard frames with identical beams, identical columns and equal storey heights, Eq.(6) may be further simplified as

$$K_{col}^m = \frac{12E_cI_c}{h^3}\frac{\sum K_b}{\sum K_b + 2K_c} \quad (7)$$

where $\sum K_b = \sum K_{bb} = \sum K_{bt}$, $K_c = E_cI_c/h$.

At last, the stiffness of individual columns in an intermediate storey is summed to form the stiffness of the storey,

$$K_m = \sum^{N_c} K_{col}^m \quad (8)$$

in which, N_c is the number of columns in the storey.

The lateral flexibility of the intermediate storey is as follows,

$$f_m = 1/K_m \quad (9)$$

Lateral flexibility of the top storey

The formulation of the lateral stiffness of the top storey is similar to that of the intermediate storey. The only difference is that there is nothing above the top storey. Therefore, the term h_a in Eq.(5) equals zero for the top storey, and the stiffness of individual columns is deduced as

$$K_{col}^t = \frac{12E_cI_c}{h^3}\frac{2\sum K_b}{2\sum K_b + 3K_c} \quad (10)$$

The lateral stiffness of the top storey is formulated as

$$K_t = \sum^{N_c} K_{col}^t \quad (11)$$

The lateral flexibility of the top storey is as follows,

$$f_t = 1/K_t \quad (12)$$

Lateral flexibility of the bottom storey

For the bottom storey, because the columns are fixed against translation and rotation on the base, the restraints on two ends of the columns are not identical. The point of inflection will have a shift towards the end where the moment-resisting stiffness is smaller, as shown in Fig.3. The column has its point of inflection determined by Eq.(40) in Sameer and Jain (1994).

For the standard frame, y_1, y_2 may be approximately calculated as follows

$$y_1 = \frac{\frac{3E_bI_b}{L}+\frac{E_cI_c}{h}}{\frac{6E_bI_b}{L}}h,\ y_2 = \frac{\frac{3E_bI_b}{L}-\frac{E_cI_c}{h}}{\frac{6E_bI_b}{L}}h \quad (13)$$

The lateral relative displacement between the ends of the column in the bottom storey is expressed as

$$\Delta_c^b = \Delta_{b2} + \Delta_{t1} + \Delta_{t2} = \frac{V(y_1)^3}{3E_cI_c} + \theta_t y_2 + \frac{V(y_2)^3}{3E_cI_c} \quad (14)$$

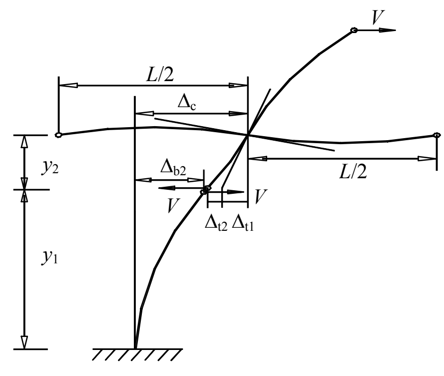

Fig.3 Isolated unit of bottom storey for analyzing deformation due to bending of beams and columns

And the lateral stiffness of the individual column is expressed as follows

$$K_{\mathrm{col}}^{\mathrm{b}}=\frac{E_{\mathrm{c}}I_{\mathrm{c}}}{h^3}\cdot\frac{216\left(\sum K_{\mathrm{b}}\right)^3}{18\left(\sum K_{\mathrm{b}}\right)^2\left(\sum K_{\mathrm{b}}+K_{\mathrm{c}}\right)+\left(K_{\mathrm{c}}\right)^2\left(K_{\mathrm{c}}-3\sum K_{\mathrm{b}}\right)} \tag{15}$$

The lateral stiffness of the bottom storey is formulated by summing the stiffness of individual columns

$$K_{\mathrm{b}}=\sum^{N_{\mathrm{c}}}K_{\mathrm{col}}^{\mathrm{b}} \tag{16}$$

The lateral flexibility of the bottom storey is

$$f_{\mathrm{b}}=1/K_{\mathrm{b}} \tag{17}$$

Lateral flexibility of the frame structure

After the analysis for all individual storeys, the lateral flexibility of the frame can be obtained by treating the storeys as springs in series,

$$F_{\mathrm{L}}^{n}=f_{\mathrm{b}}+\sum_{i=2}^{n-1}f_{\mathrm{m}}^{i}+f_{\mathrm{t}} \tag{18}$$

The lateral flexibility at the jth storey height of the frame is formulated as

$$F_{\mathrm{L}}^{j}=f_{\mathrm{b}}+\sum_{i=2}^{j}f_{\mathrm{m}}^{i},\qquad 2\le j\le n-1 \tag{19a}$$

$$F_{\mathrm{L}}^{j}=f_{\mathrm{b}},\quad j=1 \tag{19b}$$

In this way, the entire frame is simplified and modeled as a single equivalent column to formulate the lateral flexibility. The simplified model is shown in Fig.4a. First, the stiffness of individual columns in a storey is summed to obtain the stiffness of the individual storey; and then, the overall flexibility of the frame is obtained by treating all individual storeys as a tandem springs system. When the frame structure is subjected to a lateral point load at the top end, the total lateral deformation of the frame equals the summation of the interstorey deformation of all individual storeys. The following is used to verify the accuracy of this method to formulate the flexibility of the frame.

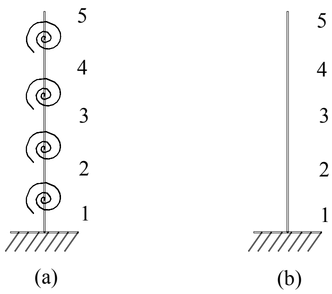

Fig.4 Simplified frame model and real column
(a) Simplified model; (b) The real column

Verification of the flexibility simplification method

The physical dimension of the frame model in Fig.1 is, span width W=500 mm, frame height H=2500 mm, storey height h=500 mm, cross-section of beams and columns is 12500 mm^2, Young's modulus of material E=206 GPa, Poisson's ratio υ=0.3. Under the lateral point force P=2000 N, the lateral displacement of the frame at the storey level calculated from the simplified method and that from the finite element method (FEM) are shown in Fig.5. It is clear that the result calculated from the simplified method is satisfactory and agrees well with that from FEM.

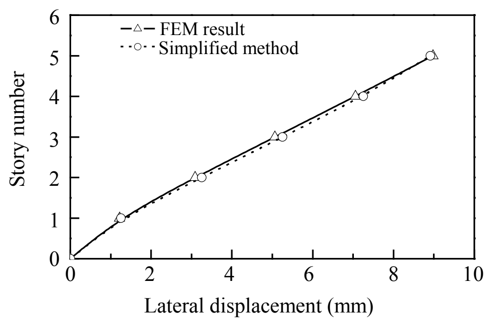

Fig.5 Lateral displacement of the frame

To make the simplified method being more accurate, Eqs.(18) and (19) are further modified as

$$F_{\mathrm{L}}^{n}=w_1f_{\mathrm{b}}+\sum_{i=2}^{n-1}w_if_{\mathrm{m}}^{i}+w_nf_{\mathrm{t}}$$

$$F_{\mathrm{L}}^{j}=w_1 f_{\mathrm{b}}+\sum_{i=2}^{j} w_i f_{\mathrm{m}}^{i},\ 2\le j\le n-1$$
$$F_{\mathrm{L}}^{j}=w_1 f_{\mathrm{b}},\ j=1 \quad (20)$$

where w_i is the weight corresponding to the ith storey.

Under the loading case of a lateral point force P applied on the top end of the frame column, the lateral displacement at each storey height of the frame is approximately expressed as

$$D_{\mathrm{L}}^{j}=PF_{\mathrm{L}}^{j} \quad (1\le j\le n) \quad (21)$$

The interstorey relative displacement of the frame is as follows,

$$R^{j}=D_{\mathrm{L}}^{j}-D_{\mathrm{L}}^{j-1}=Pf^{j} \ (D_{\mathrm{L}}^{0}=0;\ 1\le j\le n) \quad (22)$$

f^{j} is the lateral flexibility of the jth individual storey.

ANALYSIS OF THE DAMAGE DETECTION

According to the method of formulating the flexibility in the last section, the damage in different types of members (the beam and column) in the frame will have distinctive influence on the lateral displacement. At the same time, a new damage identification parameter, the obtained increment of lateral displacement change (IOLDC) at the storey level of the frame is found to be very sensitive to the local damage. The following gives the detailed analysis.

Definition of the IOLDC

It is assumed some member in the jth storey of the frame is damaged. The interstorey relative displacement of the frame after the member damage in the jth storey is expressed as

$$(R^{j})'=(D_{\mathrm{L}}^{j})'-(D_{\mathrm{L}}^{j-1})'=P(f^{j})' \ (D_{\mathrm{L}}^{0}=0,\ 1\le j\le n) \quad (23)$$

Here, the IOLDC of the jth storey in the frame is defined as

$$I_{\mathrm{d}}^{j}=(R^{j})'-R^{j}=[(D_{\mathrm{L}}^{j})'-D_{\mathrm{L}}^{j}]-[(D_{\mathrm{L}}^{j-1})'-D_{\mathrm{L}}^{j-1}]$$
$$=\Delta D_{\mathrm{L}}^{j}-\Delta D_{\mathrm{L}}^{j-1}=P[(f^{j})'-f^{j}]=P\Delta f^{j}$$
$$(1\le j\le n,\ \Delta D_{\mathrm{L}}^{0}=0) \quad (24)$$

in which, $\Delta D_{\mathrm{L}}^{j}, \Delta D_{\mathrm{L}}^{j-1}$ are the lateral displacement changes at the jth and (j−1)th storey heights of the frame before and after damage; Δf^{j} is the flexibility change of the jth storey due to the member damage. Eqs.(22)–(24) show that a definite relation exists between the IOLDC and the change in the lateral flexibility of the individual storey.

Taking the intermediate storey in the standard frame as an example, it can be seen from Eq.(7) that the lateral stiffness of the individual storey is influenced by the flexure rigidity of the columns in the storey and that of the beams meeting the columns at the top and bottom ends. The flexibility formula of the individual storey shows that the damage in beams and columns has different influence characteristics on the stiffness of the individual storey: The damage of the column only reduces the stiffness of its own storey (when it is individually analyzed). Different from the column, the beam lies between two adjacent storeys. Its damage will simultaneously induce reduction in the stiffness both of the storeys above and under the beam. In particular cases, the reduction of stiffness in storeys above and under the damaged beam are approximately equal in the standard frame.

Fig.6 shows the IOLDCs of the frame model in Fig.1 due to the damage in different member types. It is clear that the damage in different types of members follows distinctive rules of the IOLDCs at the storey level. When column 4 (which lies in the second storey) in the frame is damaged by simulating 30% reduction of the Young's modulus, the IOLDCs in other storeys except the second storey are very small. Beam 6 lies between the second and third storeys; its damage causes approximately equivalent IOLDCs in the second and third storeys, and the IOLDCs in other storeys are extremely small.

Influence of the damage in beams and columns on the lateral displacement

After simplification of the frame model and formulation of the overall lateral flexibility, Eqs.(18)–(20) show that, when there is damage in the ith storey column, the lateral flexibilities at the storey heights above the ith storey all increase. The increment approximately equals that of the ith storey

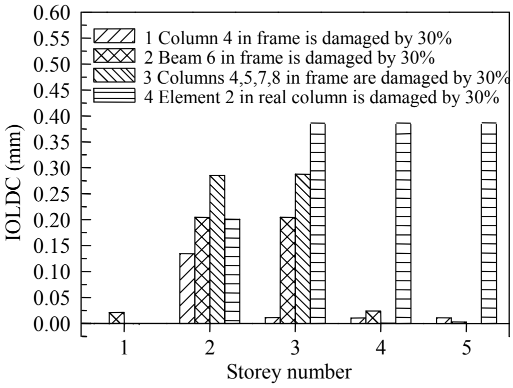

Fig.6 IOLDC at the storey level of the frame and real column

$$\Delta F_{\mathrm{L}}^{j} = (F_{\mathrm{L}}^{j})' - F_{\mathrm{L}}^{j} \approx \Delta f^{i} \quad (i \le j \le n) \tag{25}$$

$$\Delta D_{\mathrm{L}}^{j} = (D_{\mathrm{L}}^{j})' - D_{\mathrm{L}}^{j} \approx \Delta D_{\mathrm{L}}^{i} \quad (i \le j \le n) \tag{26}$$

where, F_{L}^{j}, $(F_{\mathrm{L}}^{j})'$, D_{L}^{j}, $(D_{\mathrm{L}}^{j})'$ are the flexibilities and the lateral displacements at the jth storey height of the frame before and after the damage; Δf^{i} is the flexibility change of the damaged storey when it is individually analyzed.

For the damage in beams, the flexibility changes at the heights of storeys above the damaged beam approximately equal the sum of the flexibility changes of the two storeys above and under the damaged beam when they are individually analyzed.

$$\Delta F_{\mathrm{L}}^{j} = (F_{\mathrm{L}}^{j})' - F_{\mathrm{L}}^{j} \approx \Delta f^{i} + \Delta f^{i-1} \quad (i \le j \le n) \tag{27}$$

$$\Delta D_{\mathrm{L}}^{j} = (D_{\mathrm{L}}^{j})' - D_{\mathrm{L}}^{j} \approx \Delta D_{\mathrm{L}}^{i} + \Delta D_{\mathrm{L}}^{i-1} \quad (i \le j \le n) \tag{28}$$

Δf^{i}, Δf^{i-1} are flexibility changes of the two storeys above and under the damaged beam during the individual analysis.

The changes in the lateral displacement due to the damage in different member types are shown in Fig.7. In addition, the change in the lateral displacement of the frame due to the member damage is different from that of the real column, which is also shown in Fig.7. The real column model in Fig.4b is used for comparison. Its physical dimensions are: the height H=1000 mm, cross-section is 12500 mm^2, Young's modulus E=206 GPa, Poisson's ratio υ=0.3. Under a lateral point force P=500 N applying on the top end, its displacement change due to the damage in element 2 is shown. Fig.7 shows that when there is damage in the real column, the displacement changes at the node heights above the damaged element all have a relatively large increment. However, the situation is different in the frame structure.

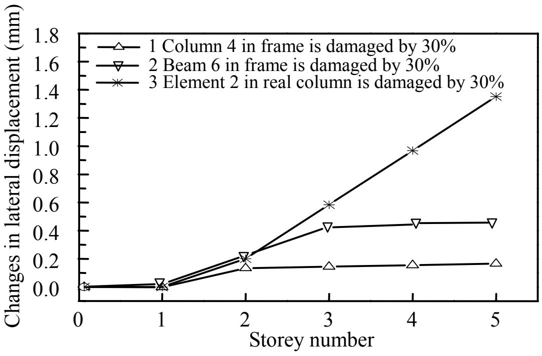

Fig.7 Displacement change due to element damage in frame and real column

Scheme using the IOLDCs for damage detection

According to above-mentioned analysis, the IOLDC of each storey in the frame is a parameter very sensitive to the local damage; and the damage in different member types follows distinctive rules of the IOLDC at the storey level. A damage detection method is developed based on these characteristics. Due to the physical symmetric property of the frame, the damage in different columns in the same storey has the same influence on the displacement. It is assumed in the following analysis that the damage in the column means that all columns in the same storey have same extent of damage. The columns in the same storey are seen as a unit to be detected.

From Fig.6, both Cases 2 and 3 have approximately the same influence characteristics on IOLDCs of frame. In this situation, only the IOLDCs mightily work ineffectively in distinguishing the different damage patterns. Therefore, the natural frequency changes of the frame are combined together with the IOLDCs for damage detection by designing a BP neural network. The IOLDCs at the storey level and the frequency change rates of the first several modes are normalized respectively, and then are combined together as the input vector of the neural network; the output of the network is the damage extent of members. A detailed illustration for this method is given through numerical examples in the next section.

NUMERICAL EXAMPLES

The effectiveness of the presented method is verified by two numerical examples. In this paper, all the possible damage patterns when there are two damaged members at most are considered.

Example 1

The plane frame model in Fig.1 is adopted. The frequency change rates of the first three lateral free-vibration modes are combined with the IOLDCs as the input vector of the neural network. The output of the network is assumed to have ten units, i.e. a beam and a column unit in each storey of frame. The number of all possible damage patterns considered is $N_p = 2\times C_5^1 + 2\times C_5^2 + C_5^1 \times C_5^1 = 55$. Of which, fifty patterns are selected to train the network, and the other five patterns are used as the testing patterns. Two damage extents are considered during generation of the damage samples, 20% and 40% reduction in the Young's modulus. One hundred training samples are generated.

The three-layer BP neural network structure is adopted; the activation functions on the hidden and output layers of the BP network are taken as the log-sigmoid and linear transfer functions, respectively. The improved back-propagation algorithm is employed to train the network. After successful training, the structure of the network is 8-18-10. The eight testing samples listed in Table 1 are divided into two groups, the first four samples are not included in the training pattern set; the last four are included in the training pattern set, but the damage extent is different from that in the training samples. The IOLDCs of the testing samples are shown in Fig.8, and the testing results from the network are shown in Fig.9. Damage extents of less than 5% are ignored in the identification results.

It can be seen that the network generally can give satisfactory identification both of the damaged members and damage extent. For the last four cases, the detection results are relatively more satisfactory. In Case 6, the column in the third storey is identified as having trivial damage together with the beam in the second storey. By further combining with the corresponding IOLDC rule in Fig.8, it can be concluded that only the beam in the second storey is damaged.

Combining with the rules of IOLDCs in Fig.8, the identification results of the first four testing samples directly from the network can be further correctly modified. In Case 1, it is very clear that only the column(s) in the second storey is (are) damaged. Therefore, the beam in the first storey can be eliminated from the damaged elements. In the same way, the beam of the first storey can also be eliminated from the damaged elements in Case 3; and beams of the first and fourth storeys can be eliminated from the damaged elements in Case 4.

Comparison of the identification results of all the testing samples showed that the resulting errors of Cases 2 and 3 are relatively larger. In Case 2, the column in the second storey of the frame is misjudged with trivial damage; and in Case 3, beams in the second and the fifth storeys are misjudged with trivial damage. This is mainly because the assumed damage extents in these two cases are relatively smaller.

Example 2

Fig.10 shows a space frame designed to further verify the proposed method. The physical dimension of the frame is: frame height H=2500 mm, storey height h=500 mm, span widths in X and Y directions are W_x=500 mm, W_y=400 mm, Young's modulus E=3000 MPa, cross-section of columns and beams is 50×50 mm^2. A lateral point force P_1=2000 N is applied on the top end of the frame in X direction.

The damage in columns and beams in X direction is considered. The same methods as that in Example 1 are used to design and train the neural network. At last, four damage samples are presented to test the successfully trained BPNN. The testing results are plotted in Fig.11. It is clear that the proposed method gives relatively satisfactory identification results for the damage situation in the complex frame.

CONCLUSION

By simplifying the model and formulating the lateral flexibility of the frame, the IOLDC at the storey level of the frame was found to be very sensitive to the local damage. At the same time, it was found that the damage in different types of members follow-

Table 1 Testing damage samples

Examples	Damage samples	Damage description	
		Damage location	Damage extent (%)
1	Case 1	Columns in the second storey	40
	Case 2	Beams in the first and fourth storeys	20
	Case 3	Columns in the second storey and beam in the third storey	20
	Case 4	Columns in the second and fourth storeys	40
	Case 5	Columns in the second and third storeys	30
	Case 6	Beam in the second storey	30
	Case 7	Columns and beam in the second storey	30
	Case 8	Columns in the second storey and beam in the fourth storey	30
2	Case 1	Columns in the third storey	30
	Case 2	Beams in *X* direction in the second storeys	30
	Case 3	Columns in the first and fourth storeys	30
	Case 4	Beams in *X* direction in the first and third storeys	30

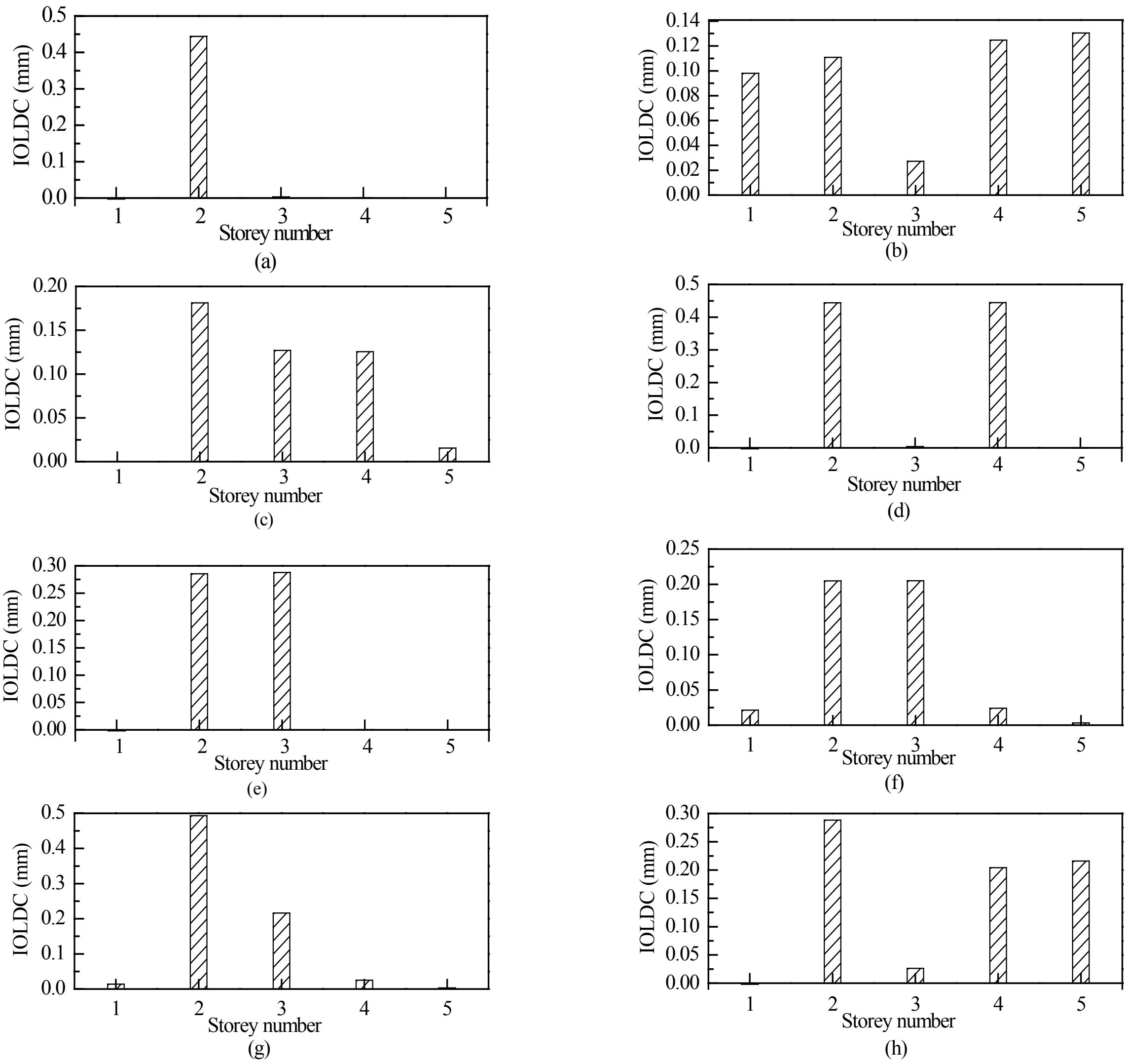

Fig.8 IOLDCs of testing samples in Example 1

(a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6; (g) Case 7; (h) Case 8

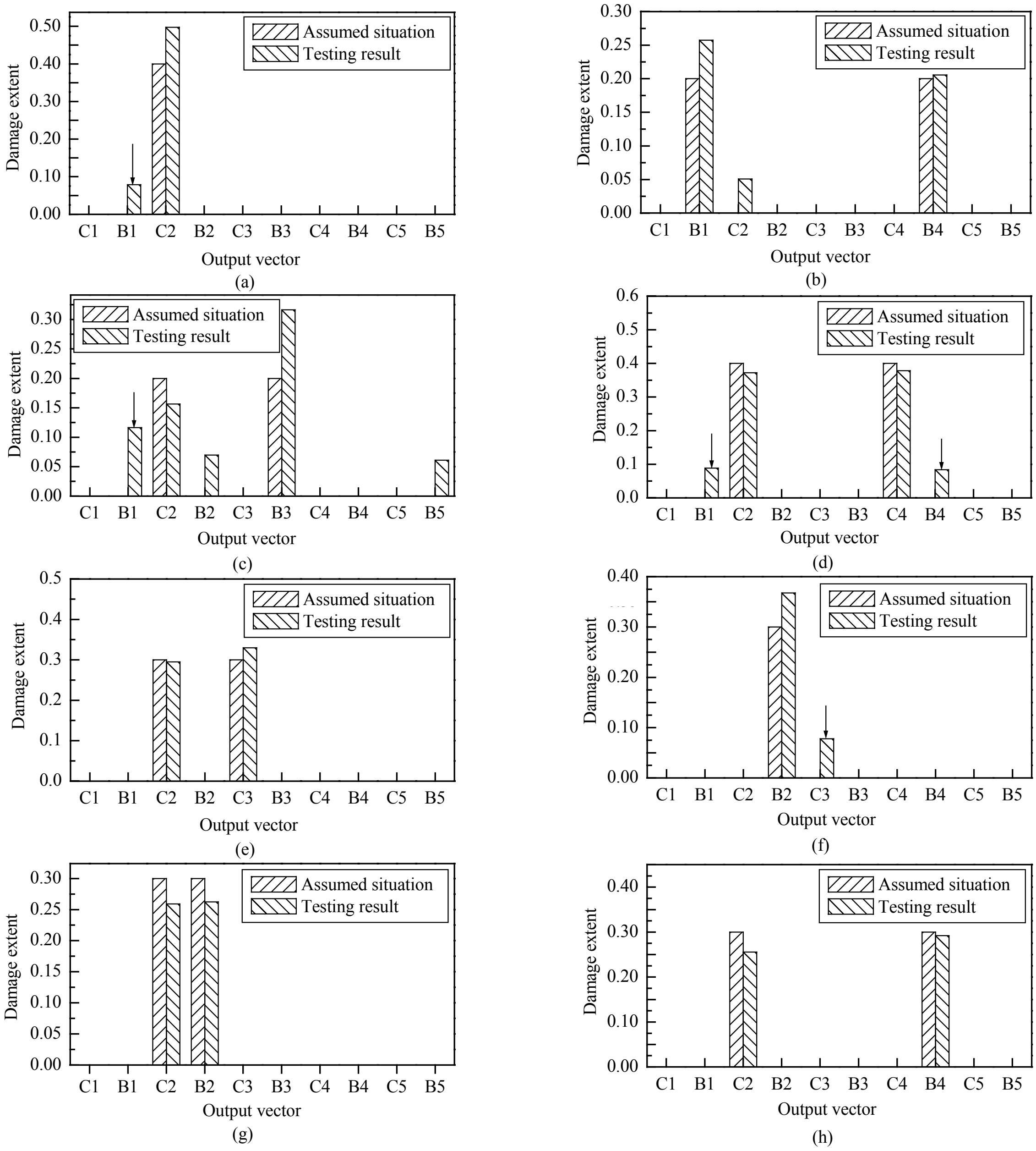

Fig.9 Identification results of testing samples in example 1 (C-Column; B-Beam)
(a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4; (e) Case 5; (f) Case 6; (g) Case 7; (h) Case 8

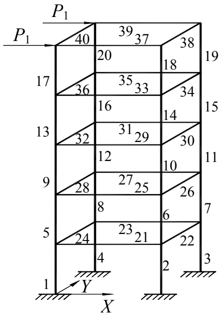

Fig.10 Space frame model

ed distinctive rules of IOLDCs. A damage detection method is proposed based on the aforementioned analysis. Numerical examples demonstrate the potential applicability of the method. The method can effectively identify the damage in different types of members at the storey level of the frame. Even for more complicated damage patterns, the IOLDCs at the storey level of the frame can approximately indicate which storeys are damaged. The detection of

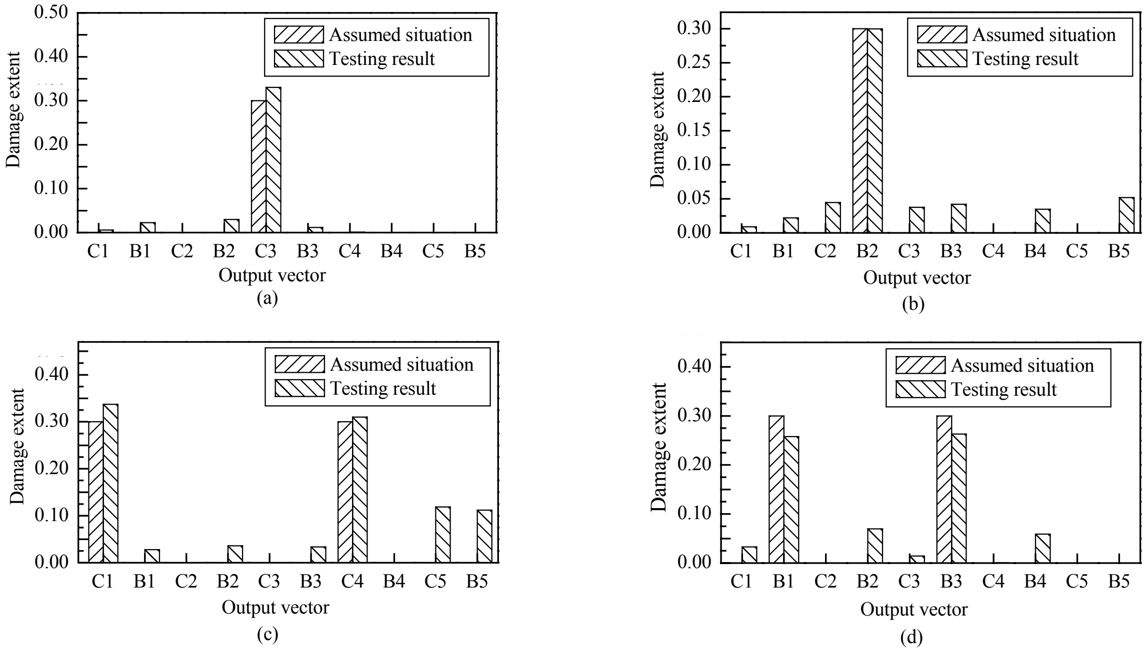

Fig.11 Identification results of testing samples in example2 (C-Column; B-Beam)
(a) Case 1; (b) Case 2; (c) Case 3; (d) Case 4

more complicated damage patterns in large-scale structures needs further study in future research to determine how this method "scales up".

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