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Loss-of-stability induced progressive collapse modes in 3D steel moment frames

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This paper deals with the progressive collapse analysis of a tall steel frame following the removal of a corner column according to the alternate load path approach. Several analysis techniques are considered (eigenvalue, material nonlinearities, material and geometric nonlinearities), as well as 2D and 3D modelling of the structural system. It is determined that the collapse mechanism is a loss-of-stability-induced one that can be identified by combining a 3D structural model with an analysis involving both material and geometric nonlinearities. The progressive collapse analysis reveals that after the initial removal of a corner column, its two adjacent columns fail from elastic flexural-torsional buckling at a load lower than the design load. The failure of these two columns is immediately followed by the failure of the next two adjacent columns from elastic flexural-torsional buckling. After the failure of these five columns, the entire structure collapses without the occurrence of any significant plastification. The main contribution is the identification of buckling-induced collapse mechanisms in steel frames involving sequential buckling of multiple columns. This is a type of failure mechanism that has not received appropriate attention because it practically never occurs in properly designed structures without the accidental loss of a column.

Keywords: progressive collapse; structural failures; loss of stability; buckling; column removal

1. Introduction

Progressive collapse of buildings is a phenomenon usually characterised by a triggering event of local structural failure or damage which results into partial or total collapse of the structure. The interest in the field of progressive collapse has greatly increased after the collapses of the World Trade Center in New York in 2001 and the Alfred P. Murrah building in Oklahoma in 1995. In response to this phenomenon, two recent publications have dominated the field of regulative progressive collapse: the Progressive Collapse Analysis and Design Guidelines for new federal buildings and major modernisation projects (GSA, 2003) and the Unified Facilities Criteria (DoD, 2009).

Numerous researchers have studied the problem of progressive collapse of steel frames through a multitude of different approaches. Some broad classifications of this body of research work are the following: 2D or 3D modelling, linear or nonlinear structural behaviour, static or dynamic loading, and energy-based or conventional force and deformation approach of progressive collapse assessment. When 3D modelling is used, another distinction is whether or not the slab is modelled.

In particular, Marjanishvili and Agnew (2006) considered linear, nonlinear, static and dynamic analyses and

compared the responses in a 3D context. They concluded that both material and geometric nonlinearities should be accounted for, and that catenary action improves the overall structural behaviour. Foley, Martin, and Schneeman (2007) performed linear dynamic analysis in 3D frames (after concluding that linear and nonlinear responses do not differ significantly), where slabs have been replaced by horizontal stiffeners. Kwasniewski (2010) and Szyniszewski and Krauthammer (2012) performed nonlinear dynamic analysis (both material and geometric nonlinearities included) in 3D frames incorporating the slab. The latter used an energy-based approach and identified buckling-initiated failure mechanisms. Kim, Kim, and Park (2009) and Khandelwal and El-Tawil (2011) conducted nonlinear vertical (push-down) static analyses in 2D frames. The former considered material nonlinearities only, while the latter considered also geometric nonlinearities and performed a hybrid 2D simulation, modelling a planar frame with the ability to develop out-of-plane buckling. Alashker, LI, and El-Tawil (2011) performed nonlinear dynamic analysis accounting for both material and geometric nonlinearities with special emphasis on modelling. They compared 2D and 3D responses and studied the contribution of the floor system to the response.

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Gerasimidis and Baniotopoulos (2011) performed linear dynamic analysis studying the effect of the time step, while Fu (2012) performed nonlinear dynamic analysis under consecutive column-removal scenarios and suggested measures to mitigate yielding-type failure mechanisms. Schafer and Bajpai (2005) developed fragility curves based on elastic stability degradation of damaged 2D frames.

Ettouney and DiMaggio (1998), Ettouney et al. (2004) and, in particular, Ettouney, Smilowitz, Tang, and Hapij (2006) emphasised at a conceptual level the critical importance of investigating the global stability of the structure when performing progressive collapse analysis, as well as the necessity to examine the global response of a damaged structural system.

This paper studies the collapse modes of a 20-storey steel building by performing a series of vertical (push-down) static nonlinear analyses, comprising 2D and 3D modelling of the structure. Emphasis is put on whether or not both material and geometric nonlinearities should be accounted for (i.e. whether or not a large displacement finite element formulation is necessary) and on their effect on the observed collapse mode. Further distinction is made between elastic and inelastic buckling modes when a loss-of-stability failure is observed.

2. Modes of progressive collapse

This section is dedicated to a brief, yet comprehensive, description of the possible progressive collapse modes of a steel moment frame following the removal of a column. Although a steel moment frame is considered a generally simple structural system, its behaviour after a column removal is very sensitive to several parameters.

One of the major progressive collapse modes which has been identified for steel moment frames involves a series of yielding-type failures triggered by the plastification of certain parts of the structural system (mainly beams) above the column removal. Such a nonlinear response may induce catenary action in the beams above the column removal and subject the frame connections and the slab to unexpected tensile stresses. A common characteristic of this collapse mode – given that shear failure is excluded – is that it is generally ductile and allows for some load redistribution through the structural system.

Although a wide range of steel frames will indeed fail through such vertical collapse modes, there is another wide range of commonly occurring structures, as the steel frame considered herein, that will collapse through a loss-of-stability-induced mode. Depending on the exact structural configuration, such a mode might be triggered before the ductility reserves of the system will have been mobilised.

For common structural applications, loss-of-stability phenomena can usually be captured by performing an eigenvalue analysis which provides the lowest buckling load and corresponding mode of the structure. However, for

progressive collapse analysis, this method alone is often insufficient, as the phenomenon of progressive collapse is strongly associated with nonlinearities which cannot be accounted for through a (linear) eigenvalue-buckling analysis. The recent work by Spyridaki, Gerasimidis, Deodatis, and Ettouney (2013) has identified and described in detail a progressive collapse mechanism which is triggered by the inelastic buckling of a column in a 2D steel moment frame. This paper determined that a finite element analysis involving both material and geometric nonlinearities is the only appropriate approach to capture computationally such inelastic loss-of-stability phenomena. A loss-of-stability-induced collapse mode does not allow in general for any meaningful load redistributions and is characterised by the sudden failure of column elements.

3. Analysis of a 20-storey steel frame building structure

3.1. Description/modelling of the structure

The 3D multi-storey steel frame studied herein is a model building from the SAC steel project and corresponds to a 20-storey frame in Boston with design based on practices prevalent before the Northridge earthquake. The force–displacement relationship of structural elements is described by the material law. The material used for all beams and columns is A572 Gr.50 steel with isotropic strain hardening, yield strength 50 ksi to 345 MPa, ultimate strength 65 ksi to 450 MPa at strain 18%. Geometry, cross-sectional areas and other specifications can be found in Appendix B of FEMA-355C (2000).

The elevation in the N–S direction and the floor plan are depicted in Figure 1. In particular, the structural system of this 20-storey office building with a two-level basement consists of perimeter moment frames and internal gravity frames. There are five-bay frames in the N–S direction and six-bay frames in the E–W direction. The connections of the six-bay frames to the five-bay frames are flexible in order to avoid biaxial bending of the corner columns. The 3D model includes only the beams spanning from column to column, with the secondary beams being neglected. The storey height is 18 ft (5.5 m) at the ground level and 13 ft (4.0 m) elsewhere, resulting in a total height of 265 ft (81.5 m) above ground. The bay width is 20 ft (6.0 m), resulting in a total width of 100 ft (30.0 m) in the N–S direction and 120 ft (36.0 m) in the E–W direction. The orientation of the column cross sections is displayed in Figure 1. The steel-concrete composite slab of 5.5 in. (0.14 m) total thickness (consisting of 3-in. metal decking with 2.5 in. of normal weight concrete fill) is replaced for modelling simplicity by an equivalent uniform and homogeneous slab of the same thickness exhibiting elastic behaviour with Young's modulus equal to 30 GPa.

To study the effect of 2D versus 3D modelling of the structure on progressive collapse behaviour, the 2D planar

Figure 1. N–S elevation and floor plan of the 20-storey steel frame considered.

perimeter moment frames are also separately analysed. All analyses are performed using the nonlinear FEM code ABAQUS (Simulia, 2012). To produce compatible results between the 2D and 3D cases, the same global mesh size and element type are used in all cases considered (i.e. linear beam element B21 ABAQUS for the 2D case and linear beam element B31 ABAQUS for the 3D case). In the 3D case, the slabs are modelled using the four-node shell element S4R ABAQUS. Thus, the 2D five-bay planar frame comprises 850 nodes, the 2D six-bay planar frame 1007 nodes, while the 3D space frame 19,044 nodes. Base nodes as well as the perimeter nodes of the two-level basement are considered pinned as mentioned in FEMA-355C (2000).

The progressive collapse analysis conducted herein follows the alternate load path philosophy (DoD, 2009; GSA, 2003) that involves the removal of an element of the structure. This element is selected to be the column at the S–E corner of the ground level indicated in Figure 1. The loading pattern follows the guidelines of the nonlinear static procedure of the Unified Facilities Criteria (DoD, 2009), accounting for the dynamic increase factor Ω_N and applying it on the loads of slabs (or beams) above the column removal. The calculation of the dynamic increase factor for steel-framed structures follows the process described in paragraph 3-2.12.5 of DoD (2009) which first identifies the smallest ratio (among the primary elements within the area of the column removal) of the plastic rotation angle θ_{pra} in the acceptance criteria of DoD (2009) over the yield rotation θ_y .

$$\Omega_N = \max \left\{ 1.08 + \frac{0.76}{(\theta_{pra}/\theta_y) + 0.83} \right\}$$

For the specific frame, Ω_N is calculated and found to be equal to 1.25 in all cases examined. The load-control method, incrementally increasing the distributed vertical

load q and solving the problem repeatedly until failure occurs, is adopted for all progressive collapse analyses considered in this study. It must be mentioned that for the presentation of the results from the 2D analyses, the line load (kN/m) applied on the beams is converted into equivalent pressure load q (kPa) in order to produce comparable units to the 3D analyses.

The analysis is performed by using the following three methods:

- (1) Eigenvalue buckling analysis (denoted by $[E]$),
- (2) Static load-control analysis involving material nonlinearities only (denoted by $[M]$) and
- (3) Static load-control analysis involving both material and geometric nonlinearities (denoted by $[M + G]$).

3.2. 2D modelling

3.2.1. 2D eigenvalue buckling analysis $[E]$

The eigenvalue buckling analysis is a linear procedure that estimates the critical (bifurcation) load of a structure. Figure 2 displays the first eigenmode of the two planar frames in the N–S and E–W directions. A lateral failure mode, triggered at the level of the column removal, is observed in both cases. The elastic buckling load is of the order of 400 kPa as shown in Figure 2, way larger than the load of 7 kPa which was used for the design of the structure. A load of 7 kPa is the sum of the dead and live loads and is used here as a simple design reference load. The DoD guidelines recommend a design load of $1.2 \cdot \text{Dead} + 0.5 \cdot \text{Live}$ which would yield to a value of 6.72 kPa for the specific frame which is essentially the same as 7 kPa. It was the intention of the authors not to

Figure 2. 2D Eigenvalue buckling analysis: first buckling eigenmode of E–W (left) and N–S (right) planar frames.

enter the topic of accidental loading conditions which is a field of research on its own and that is why a reference load of 7 kPa (dead plus live) was considered. This choice is not affecting the findings of this work. Thus, the results of the planar eigenvalue buckling analyses do not yield any concern for the structure's integrity.

3.2.2. 2D static load-control analysis involving material nonlinearities only [M]

The Von Mises stress diagram on the deformed shape of both planar frames at failure is depicted in Figure 3 for the analysis involving material nonlinearities [M] only. This collapse mode is characterised by extensive yielding of

Figure 3. 2D static load-control analysis involving material nonlinearities only [M]: Von Mises stress diagram on the deformed shape of the structure at failure for the E–W (left) and N–S (right) planar frames.

many elements of the structure including several beam and column cross sections, especially at the lower levels, where numerous sections have exhausted their plastic strain limit (as defined by the elastic–plastic material law used) and by large (unrealistic) vertical displacements over the column removal as displayed in Figure 4. Figure 5 displays the horizontal displacement of the mid-height node of the column adjacent to the removed one. Figure 6 displays the axial force of the column adjacent to the removed one. The quantities in Figures 4–6 are plotted as functions of the vertical load q applied on the slabs.

Figure 4. 2D static load-control analysis: vertical displacement of the node exactly above the column removal as a function of the vertical load q .

Figure 5. 2D static load-control analysis: horizontal (lateral) displacement of the mid-height node of the column adjacent-to-the-removed one as a function of the vertical load q .

Figure 6. 2D static load-control analysis: axial force on the column adjacent-to-the-removed one as a function of the vertical load q .

It is difficult to estimate the failure load from Figures 4–6 because of the strain-hardening behaviour of the material. The collapse load in this case is defined by observing the evolution of the displacements in several parts of the structure. Figure 3 displays a conservative value of 30 kPa for the failure load. It is emphasised that in this case ([M]), the failure mode observed is of the yielding-type and there is no loss-of-stability detected.

3.2.3. 2D static load-control analysis involving both material and geometric nonlinearities [M + G]

In this case, the Von Mises stress diagram on the deformed shape of both frames is displayed in Figure 7, where a lateral collapse mechanism, triggered by the buckling of the column adjacent to the removed one, is manifested clearly. In fact, the Von Mises stress diagram (plotted in the same chromatic scale as for the [M] case) shows that considerable yielding has occurred at the buckled column. Figure 6 displaying the axial force in the

column adjacent to the removed one exhibits a distinct horizontal branch, a characteristic of buckling. The value at which this horizontal branch starts forming coincides with the value of the collapse/inelastic buckling load. The analysis is terminated with the appearance of negative eigenvalues of the stiffness matrix, another classic characteristic of buckling. Figure 5 displays the horizontal displacement of the mid-height node of the column adjacent to the removed one. The dramatic increase in the value of this horizontal displacement (snapping behaviour) constitutes another classic characteristic signature of the loss of stability.

The combination of the aforementioned three factors characterise the failure mode observed in this case as a loss-of-stability-induced one. The corresponding failure loads are displayed in Figure 7.

3.2.4. Conclusions for 2D modelling

Comparing Figures 2, 3 and 7, it becomes immediately obvious that in order to capture the real/actual collapse

Figure 7. 2D static load-control analysis involving material and geometric nonlinearities $[M + G]$: Von Mises stress diagram on the deformed shape of the structure at failure for the E-W (left) and N-S (right) planar frames.

mode (loss-of-stability-induced) and the corresponding collapse loads (displayed in Figure 7), it is necessary to perform a nonlinear analysis involving both material and geometric nonlinearities. A nonlinear analysis involving only material nonlinearities yields a wrong collapse mode (yielding-type) and a corresponding unconservative collapse load. Finally, an eigenvalue buckling analysis provides a totally unrealistic (and grossly unconservative) collapse load, as the response of the structure is highly nonlinear when failure is imminent.

It should be pointed out that the real/actual collapse load of 21 kPa (displayed in Figure 7) is significantly higher than the design load of 7 kPa.

3.3. 3D modelling

3.3.1. 3D eigenvalue buckling analysis $[E]$

Figure 8 displays the first 3D buckling eigenmode which corresponds to 6.3 kPa and involves a flexural-torsional out-of-plane (about the weak axis) buckling of the column adjacent to the removed one in the N-S perimeter frame. The load associated with this out-of-plane buckling is lower than the design load (7 kPa) and consequently, unlike the 2D case, the 3D eigenvalue buckling analysis cannot be disregarded. Actually, as it will be shown later, the 3D eigenvalue analysis successfully predicts the collapse mechanism and collapse load of the structure.

Figure 8. First 3D buckling eigenmode (elastic buckling load: 6.3 kPa) in perspective and side view.

Figure 9. 3D static load-control analysis: axial force on the column adjacent-to-the-removed one as a function of the vertical load q . [D] denotes the design reference load and [E] the eigenvalue buckling analysis collapse load.

3.3.2. 3D static load-control method of analysis involving material nonlinearities only [M]

Figure 9 displays the axial force in the column adjacent to the removed one in both the N–S and E–W directions as a function of the vertical load q applied on the slabs. The [M] curves in Figure 9 indicate that the axial force keeps increasing as a function of q beyond 20 kPa. The structure exhibits extensive plastification and the analysis is eventually terminated when the available tensile strain of certain cross sections is exhausted (refer to Figure 11). Figure 10 depicts the vertical displacement of the node exactly above the column removal plotted again as a function of the vertical load q applied on the slabs. The large (and unrealistic) vertical displacements for the [M] case reflect the extensive plastification of the structure already mentioned above. The Von Mises stress diagram at failure is shown in Figure 11 where the observed collapse mechanism is characterised by extensive yielding of columns at lower floors. It is again difficult to estimate the failure load from Figures 9 and 10 because of the strain-hardening behaviour of the material. Figure 11 displays a conservative value of 21 kPa for the failure load.

Figure 10. 3D static load-control analysis: vertical displacement of the node exactly above the column removal as a function of the vertical load q applied on the slabs.

3.3.3. 3D static load-control analysis involving both material and geometric nonlinearities [M + G]

Comparing the [M] and [M + G] curves in Figure 9, there is a clearly different behaviour between the two. While the [M] curves display an essentially linear behaviour up to 15 kPa and keep increasing for values of q beyond 20 kPa, the [M + G] curves behave linearly up to relatively low values of q (6.3 and 6.5 kPa for the N–S and E–W directions, respectively) and then exhibit a clear and short horizontal branch before the analyses are terminated because of the appearance of negative eigenvalues in the stiffness matrix (both typical characteristics of buckling).

In Figure 9, the horizontal branch of the axial force for the adjacent column in the N–S direction ([M + G] model) starts forming at the same vertical load q (6.3 kPa) determined by the 3D eigenvalue buckling analysis [E]. This perfect agreement between the results of the two analyses indicates clearly an elastic flexural-torsional buckling mode. The [M + G] model has undergone very limited yielding at failure (compared with the [M] model) as indicated in Figure 11. It is noteworthy that elastic instability is triggered in the 3D model even before it can reach the design reference load of 7 kPa.

The value of the axial force in the column when buckling occurs (equal to 5000 kN for both columns as they have the same cross section) can provide valuable information about the equivalent boundary/support conditions of these members. This is a way to quantify the slab effect on the column buckling capacity. The vertical displacement at failure of the node exactly above the column removal for the [M + G] case (Figure 10) is much smaller compared with the corresponding displacement for the [M] case, reflecting the detected elastic buckling failure mode.

3.3.4. Conclusions for 3D modelling

In the case of 3D modelling of the structure, it is obvious that in order to capture the real/actual collapse mode

Figure 11. 3D static load-control analysis: Von Mises stress diagram at failure on the deformed shape of the 3D structure for the $[M]$ and the $[M + G]$ analyses (colour scale same for both cases).

(induced by elastic flexural-torsional buckling) and the corresponding collapse load, it is necessary to perform either an eigenvalue analysis or a nonlinear analysis involving both material and geometric nonlinearities. A nonlinear analysis involving only material nonlinearities yields a wrong collapse mode (yielding-type) and a corresponding unconservative collapse load. It should be pointed out that the real/actual collapse load of 6.3 kPa is lower than the design load of 7 kPa.

4. Discussion on the analysis results

4.1. The necessity for 3D analysis

The necessity for the $[M + G]$ method of analysis (compared with the $[M]$ method of analysis) is obvious. In this section, the necessity for 3D modelling of the structure (compared with 2D modelling) will be demonstrated. Figure 12 displays the axial force in the columns adjacent to the removed one (both in the N–S and E–W directions) plotted against the vertical load q applied on the slabs. The curves of the 2D and 3D models bear noteworthy similarities and differences. The appearance of a horizontal branch in all curves in conjunction with the fact that all analyses are terminated with the appearance of negative eigenvalues in the stiffness matrix are indicators of loss of stability (buckling failure of columns). The columns adjacent to the removed one in the N–S and

E–W frames exhibit almost identical behaviour; the column on the N–S frame collapses a little bit earlier though (at 6.3 kPa vs 6.5 kPa in 3D and at 21 kPa vs 23 kPa in 2D).

In the 2D case, the adjacent column buckles at a load of 21 kPa (significantly higher than the 7 kPa design reference load) *about its strong axis*. At this failure load of 21 kPa, extensive yielding/plastification has occurred in the column underconsideration, which leads to the conclusion that this is an inelastic buckling failure. This horizontal inelastic collapse mechanism is manifested by the deformed shape and the Von Mises stress diagram in Figure 7. Due to its linearity, the eigenvalue buckling analysis is of course irrelevant in the 2D case.

In the 3D case, the adjacent column buckles at 6.3 kPa about its weak axis. This happens at a load lower than the design reference load (7 kPa). In this case, the buckling of the adjacent column occurs before any of its cross sections have reached their yielding point. Consequently, as discussed earlier, this is an elastic buckling failure. In the 3D case, the buckling failure mechanism is not depicted as explicitly as before in the deformed shape (see the $[M + G]$ analysis in Figure 11), because the analysis is terminated due to negative eigenvalues before any substantial deformation occurs. Due to its linearity, the eigenvalue buckling analysis predicts nicely the collapse load of 6.3 kPa in the 3D case.

Figure 12. Axial force on the column adjacent-to-the-removed one in the N-S (above) and in the E-W (below) direction plotted as a function of the vertical load q applied on the slabs. Comparison between 2D and 3D cases for $[M + G]$ analysis. $[D]$ denotes the design reference load.

The main conclusion here is that although both the 2D and 3D $[M + G]$ analyses predict a loss-of-stability-induced collapse mechanism, only the 3D analysis can determine the actual/real collapse mode and corresponding collapse load of 6.3 kPa (the 2D analysis yields a significantly higher and unconservative value of 21 kPa).

4.2. The progression of damage in the 3D $[M + G]$ analysis

As discussed previously in detail, the 3D model of the structure experiences elastic buckling of the columns adjacent-to-the-removed-one about their weak axes at a load which is slightly below the design reference load. This raises great concern about the overall integrity of the structure.

To verify the progression of damage after the identified elastic buckling of the two columns adjacent to the removed one, an additional analysis is performed simulating the progression of the damage. The two buckled columns are removed from the model, while the corresponding dynamic overload area is extended to cover the area above these two columns. This configuration

simulates the elastic buckling failure of the two columns adjacent to the removed one. If this additional analysis yielded a higher collapse load than the one calculated in the 3D model $[M + G]$ analysis (6.3 kPa), the structure would be able to withstand the three column removals and would not experience an overall collapse (Figure 13).

Figure 13(a),(b) displays the axial forces in the columns adjacent to the three removed ones in the E-W and N-S directions, respectively. Both curves exhibit the horizontal branch which is the classic characteristic of buckling and negative eigenvalues appear in the stiffness matrix (the other classic characteristic of buckling). The adjacent columns on each side (E-W and N-S) start forming this horizontal branch at 5.2 kPa which is lower than 6.3 kPa. This means that these two columns will fail by buckling immediately after the first two columns that fail at 6.3 kPa (also by buckling).

Figure 13(c),(d) displays the deformed shapes of the structure in the E-W and N-S directions, respectively, at the last loading step of the analysis. The two adjacent columns fail in an elastic flexural-torsional buckling mode. Figure 13(c),(d) displays the torsional deformation of these columns and show no significant yielding in the beams above the column removals.

The conclusion of this additional analysis is that the removal of the first corner column is followed by elastic flexural-torsional buckling failure of its immediately adjacent columns and then by elastic flexural-torsional buckling failure of the next adjacent columns in the same floor. It is trivial to show that the failure of these five columns translates into overall collapse of the entire structure.

5. Concluding remarks

This paper has demonstrated the critical importance of using a 3D model of a structure in conjunction with an analysis involving material and geometric nonlinearities to determine loss-of-stability-induced collapse mechanisms and corresponding collapse loads. A 20-storey steel frame (FEMA-355C, 2000) has been selected for this purpose. The analysis results revealed that after the initial removal of a corner column, the two adjacent columns fail from elastic flexural-torsional buckling at a load level which is lower than the design load. The failure of these two columns is followed immediately afterwards by the failure of the next two adjacent columns from elastic flexural-torsional buckling. After the failure of these five columns, the entire structure collapses without the occurrence of any significant plastification in the structural system.

This work demonstrated the strong relationship between loss-of-stability phenomena and progressive collapse for certain common steel-framed structures such as the SAC-FEMA design used in this work. The criticality in selecting the appropriate numerical/computational

Figure 13. Additional 3D static load-control analysis involving removal of 3 columns. (a) and (b): axial force in the columns adjacent-to-the-removed ones as a function of the vertical load q . (c) and (d): deformed shapes of the structure in the E–W and N–S directions respectively, at the last loading step of the additional analysis.

technique to identify such collapse mechanisms has been presented using several analysis techniques including eigenvalue, material nonlinearities, material and geometric nonlinearities, as well as 2D and 3D modelling of the structural system. The next task will be to carry out nonlinear dynamic analysis to study dynamic loss-of-stability phenomena.

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