

Progressive collapse in multi-storey steel frames: the effect of varying column spacing

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ABSTRACT: Following the collapse of the Ronan Point apartment tower, provisions to minimise the potential for progressive collapse were introduced in design codes for the first time. In recent years, the increased threat of terrorism has highlighted the importance of such robustness requirements. This paper presents a methodology to assess the extent of damage to a multi-storey structure following localised collapse. This is accomplished through the design of an algorithm, based on the notional element removal method. By systematically considering the effects of damage to all members in a structure, this algorithm can be used as both a design and an analysis tool to identify whether a structure is unduly sensitive to the effects of localised damage. This paper describes the key features of this analysis program. Additionally, the results of a study to determine the effect of column spacing on the response of a damaged structure are presented.

1 INTRODUCTION

On May 16, 1968, a small gas explosion triggered the partial collapse of the Ronan Point apartment tower. This event focused the intellectual debate on progressive collapse, and following the report of the Commission of Inquiry a number of countries introduced provisions to minimise the potential for progressive collapse. For the first time in the UK, it was required that *“structure[s] shall be designed and executed in such a way that [they] will not be damaged by events to an extent disproportionate to the original cause”* (CEN 2006).



Figure 1. Ronan Point collapse.

The more recent attacks on the Murrah Federal Building (1995) and the World Trade Centre twin towers (2001) have highlighted the increased threat of terrorism worldwide. As a result, numerous publications on progressive collapse and extreme loading have appeared in the literature over recent years.

As building designers cannot possibly design for every hazard that a building may be subjected to in its lifetime, a general design approach is required to account for the risks associated with low-probability high-consequence events. This can be achieved through the provision of robustness: where the authors consider robustness to be the ability of a structure to survive a certain degree of local damage without causing damage, to an extent disproportionate to the original cause. Current design codes and guidelines (ASCE 2005, CEN 2006, DoD 2009, GSA 2003) suggest three recommended approaches to provide a structure with an acceptable level of robustness. These are improved interconnection or continuity; key element design; and the notional element removal method.

A methodology to assess the extent of damage to a multi-storey structure following localised collapse has been developed by the authors (Janssens and O'Dwyer 2010). This has been accomplished through the design of an algorithm based on the no-

tional element removal method, and its implementation as a computer program. This program models the progression of collapse through a structure, following the loss of one (or more) primary load carrying members. The dynamic effects associated with sudden member loss are included, and material nonlinearities are modelled using a plastic hinge approach. Additionally, the structural model is updated at each time step to account for geometric nonlinearities arising as a result of significant deformations in the structure. By systematically considering the effects of damage to all members in a structure, this algorithm can be used as both a design and an analysis tool to identify whether a structure is unduly sensitive to the effects of localised damage. The key features of this program are described in section 3 of this paper.

As part of this research, the analysis program developed is being employed to study the increased displacements and internal forces which may arise in steel-framed structures, following the sudden loss of a key element. By performing a series of parametric studies, the authors intend to quantify the increased displacements and internal forces which may arise following localised failure. The sensitivity of the response to variations in the material properties and the overall structural geometry will be assessed. The critical structural components in a collapse sequence will also be identified. This information may then be utilised by designers to ensure a structure possesses sufficient robustness to guarantee safety of its occupants if an extreme event were to occur. This paper presents the results of a preliminary study to determine the effect of column spacing on the response of a damaged structure.

2 DESIGN METHODS TO IMPROVE ROBUSTNESS

There are, in general, three possible approaches to designing robust structures: improved interconnection or continuity; key element design; and the notional element removal method.

2.1 Improved interconnection or continuity

The overall robustness of a structure can be increased by adopting general prescriptive measures which improve integrity throughout the structure: usually in the form of requirements for minimum joint resistance, continuity and tying between the members. This approach has the advantage that it can be implemented without the need for any additional analysis. This is a significant benefit when dealing with unforeseen loading conditions, and therefore this approach is incorporated into most major codes and guidelines (ASCE 2005, CEN 2006, DoD 2009, GSA 2003).

This method is often adopted in the form of minimum tying force requirements (CEN 2006). These

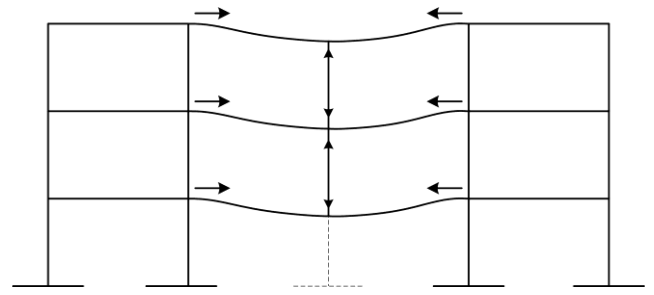


Figure 2. Horizontal ties bridge local failure by catenary action, while vertical ties redistribute loads among the floors above.

requirements are based on the underlying philosophy that if all members are connected by joints with a specified capacity, the selected structure will have adequate strength to resist collapse, by allowing redistribution of the gravity loads following a local failure. In general, both horizontal and vertical ties should be considered, the capacities of which are determined separately to the design loads.

The provision of horizontal ties is based on the concept that, following the loss of a support, the remaining structure will support the loads through catenary action (Alexander 2004). While vertical ties are intended to redistribute the loads throughout the remaining floors (Figure 2). However, recent research by Liu et al. (2005) demonstrated that for a simple steel frame the peak tying force substantially exceeds that required by BS5950:1 (2000) and EN1991-1-7 (2006). Furthermore, the rotations experienced by the connections as the structure absorbs the damage are likely to be significant. In fact, it has been shown that, for steel framed buildings, industry standard beam-column connections possess insufficient ductility to accommodate the displacements required to mobilise catenary action (Byfield and Paramasivam 2007). Hence, the peak tying force and the rotational capacity of connections require careful consideration if this approach is adopted.

It should also be noted that the provision of continuity can be counter-productive in some cases. When an 1800 kg TNT equivalent truck bomb exploded outside the Murrah Building the resulting blast destroyed one of the ground floor columns (Osteraas 2006). The ensuing progressive collapse destroyed nearly half of the building and was enhanced by the continuity of the reinforced concrete frame. In this case, the extent of collapse could have been reduced had the reinforcement not been continuous throughout. The partial collapse of Paris Roissy Charles de Gaulle Airport's Terminal 2E (Starossek 2009) illustrates the usefulness of such structural segmentation. In this case, the failure of a roof section initiated the collapse sequence, in which only 24 m of the 680 m long structure collapsed. The progression of collapse beyond this portion of the structure was prevented (unintentionally) by a movement joint at one end, and a weak joint at the other.

2.2 Key element design

The key element design approach involves identifying vulnerable members of a structure and designing them to resist a specified extreme load. As a result, the structure is provided with additional strength at areas that are believed to be prone to accidental loads (e.g. exterior columns at risk from vehicular collision), or in key elements that are crucial to the overall structural stability. These members should be able to develop their full resistance against an anticipated load without failure of either the member itself or its connections. By activating the full resistance available in the key members, this approach maximizes their ability to deal with unforeseen hazards without having to redistribute loads.

One of the issues with designing to prevent disproportionate collapse is that the loading events in question are outside the scope of normal design. Due to the unforeseen nature of these events, we cannot accurately predict their magnitude and location. Yet, EN 1991-1-7 (2006) requires key elements be designed to resist a uniformly distributed load of 34 kN/m^2 : a value derived from the peak pressure in a gas explosion (Alexander 2004). However clearly loads greater than this will result in failure of the element. Therefore, this approach may be of limited benefit in resisting collapse. In respect of this, the key element design approach was initially intended to be used only if a structure cannot sustain notional removal of the element in question (Ellingwood and Leyendecker 1978). But this is not necessarily clear in modern design codes.

2.3 The notional element removal method

The notional element removal method was initially recommended following significant research during the 1970's (Allen 1972, Burnett 1975, Taylor 1975, Ellingwood and Leyendecker 1978). This approach focuses on the behaviour of a structural system following the occurrence of an extreme event, and requires the structure to redistribute the loads following loss of a primary load bearing member. The basic procedure followed in any alternative path analysis involves removal of one, or more, primary structural components. The altered structure is then analysed to determine if the initiating damage propagates. This method promotes the use of regular structural configurations that exhibit ductility and energy absorption properties, which are desirable features of a robust structure.

One of the main advantages of this technique is that it is a threat independent approach, and therefore, is valid for any hazard that may cause failure. This avoids one of the main difficulties faced by engineers in designing structures to resist progressive collapse, attempting to quantify an otherwise unknown loading event. The design guidelines pro-

duced by the Department of Defence (2009) and the General Services Administration (2003) both recommend the use of this technique. In general, there are three alternative analytical approaches, of increasing complexity: linear static, nonlinear static, and nonlinear dynamic analysis.

The increased accuracy associated with more complex nonlinear and dynamic analysis procedures comes at a significant computational expense. However, the notional element removal method does not intend to precisely model the progressive failure process. Instead, the purpose of this approach is to assist engineers in designing more robust structures. Therefore, it is possible to use the simpler procedures but an experienced engineer with considerable knowledge and experience in structural modelling is essential to ensure validity of the results.

3 MODELLING PROGRESSIVE COLLAPSE

An analysis program capable of following the sequence of failures that occur during a progressive collapse has been developed by the authors. This program implements the finite element method, and is based on the notional element removal technique. Figure 3 outlines the algorithm adopted for this purpose. This algorithm removes individual elements from the structural model and computes the associated response. By systematically considering the effects of damage to all members in a structure, this algorithm could be used as both a design and an analysis tool to identify whether a structure is unduly sensitive to the effects of localised damage. Finally, this program models the structure to be analysed in two-dimensions. Therefore, three-dimensional effects which may exist in a 'real' structure are not considered. The following sections outline the key features of this analysis tool.

3.1 Initiating event

The notional element removal method requires a primary load-carrying member to be removed from the structure, in order to simulate localised failure. When performing a dynamic analysis the time period over which the initial element removal occurs will influence the computed response. The guidelines produced by the General Services Administration (2003) recommend that the time period for removal of an element is less than $1/10$ of the natural period of that element. This is based on the fact that if an explosive device is detonated in contact with or in close proximity to a structural member, that member can be removed almost instantaneously by brisance (Cormie et al. 2009). This recommendation has been adopted for the program developed.

However, collapse of this nature is usually initiated by unforeseen loading conditions. This presents

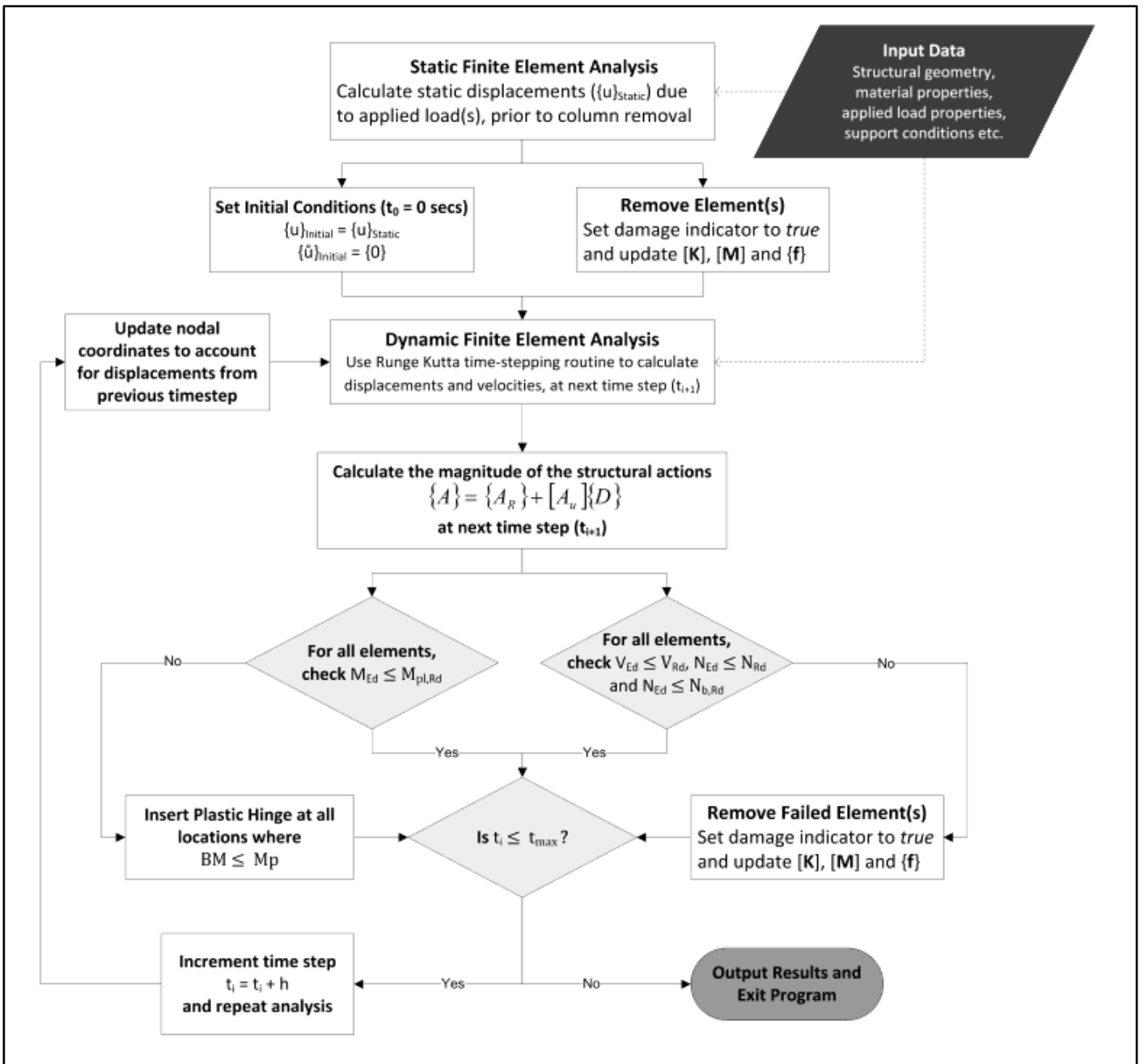


Figure 3. Flowchart describing progressive collapse algorithm.

the designer with the difficulty of predicting the extent of initial damage caused by an unknown event.

The initial damage observed will be dependent on a large number of factors: including the magnitude and location of the exposure; capacity of the structural elements and connections; distance between members; ceiling height; and distribution of stiffness throughout the structure. For this reason, it is advisable that for a comprehensive progressive collapse analysis a wide range of initiating events should be considered, including the removal of multiple elements and the removal of elements at various locations throughout the structure.

3.2 Geometric nonlinearities

As a result of the large deformations likely to occur following localised damage, geometric nonlinearities

are likely to have a significant effect on the internal forces in the structure. These nonlinearities are included in the program by updating the nodal coordinates at the end of each time-step. Hence, the stiffness and mass matrices, as well as the restraining force vector, are updated to include any nonlinearities. This allows the program to model P-delta effects, so that secondary moments due to the lateral deformation of the columns, or the entire structure, can be calculated. Similarly, by including the deflected shape of beam and floor members catenary effects are accounted for.

3.3 Material nonlinearities

Additionally, inelastic behaviour of the structure can lead to increased rotation in the members and connections, and hence will influence the structural re-

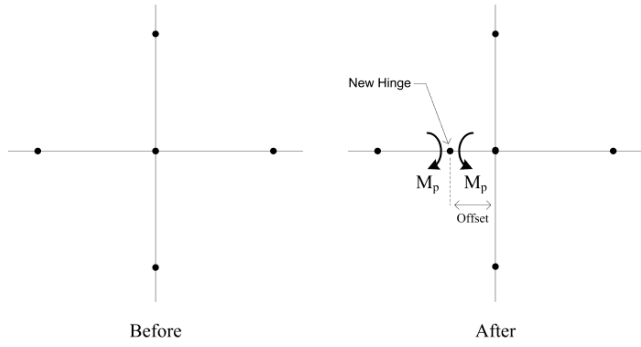


Figure 4. Approach used when inserting plastic hinges.

sponse. Such material nonlinearities are incorporated into the analysis using a plastic hinge approach.

At each time step, the algorithm checks all members of the structure for plastic yielding. This is achieved by checking if the bending moments in each element exceed its plastic moment capacity, at any point. If plastic yielding occurs, a plastic hinge is inserted at that point (Figure 4).

A plastic hinge is inserted in an element by introducing an additional degree of freedom (if necessary) at the relevant location. The rotational stiffness of this point is set to zero, and a moment bi-action, equal to the plastic moment capacity (M_p) of the element, is applied as shown. This allows plastic rotation to occur at the hinge, without affecting the moment at the hinge. Any permanent plastic deformation is also recorded so that it can be accounted for if the loads are later reversed.

The rotation at the plastic hinge locations is closely monitored as the analysis progresses. If the rotation starts the decrease, the plastic hinge is removed from the model and a permanent plastic deformation is applied to the element.

3.4 Dynamic effects

The sudden removal of a structural component results in an immediate change in the structural geometry. As a result of this, gravitational energy is released and the internal strain energy, and kinetic energy, of the structure can be expected to alter rapidly. Therefore dynamic effects are important when attempting to accurately represent the associated structural behaviour.

Dynamic effects can be indirectly considered by assuming an equivalent static load based on a constant amplification factor, typically taken equal to 2.0 (DoD 2009, GSA 2003). However, this approach has been proven to underestimate the response in some cases (Izzuddin and Nethercot 2009) and should be used with caution.

A more comprehensive approach involves dynamically removing a member from the structure. This program models dynamic effects, via a Runge-Kutta time-stepping routine which solves the dynamic equation of motion:

$$[M]\{\ddot{x}\} + [C]\{\dot{x}\} + [K]\{x\} = \{f\} \quad (1)$$

where $[M]$, $[C]$ and $[K]$ are the mass, damping and stiffness matrices of the structure, respectively. $\{x\}$ describes the displacement of the structure and $\{f\}$ describes the external forces acting on the structure.

The initial conditions for the structure are determined by calculating the static response of the structure to the unfactored design loads, prior to removal of the structural element(s). The initial displacements for the dynamic analysis are set equal to the static displacements, and the initial velocities are set to zero. The algorithm then iterates through the time steps, calculating the displacements and velocities. The values of the structural actions along the length of each member are also computed using appropriate shape functions. At present, this program does not include the effects of structural damping.

3.5 Identifying member failure

When modelling progressive collapse, it is necessary to be able to track the series of local failures that occur in response to the initiating event. All elements are checked for failure in compression, tension, shear and buckling at the end of each time-step. If failure occurs, the structural matrices are updated to exclude these members.

However, the case of failure due to the formation of an unstable mechanism is more complex. The formation of such a mechanism results in instability in the structural matrices: characterised by a row/column of zeros, or multiple rows/columns that are multiples of each other. A routine has been written to detect such a mechanism and remove the unstable portion of the structure. It should be noted that for large displacements a mechanism could form that is in fact stable: for example where the applied loads are resisted through catenary action (see Figure 2).

4 CASE STUDY

The following case study demonstrates the application of the progressive collapse program described in the preceding section. The behaviour of the two-storey frame (shown in Figure 5) is analysed, with a uniformly distributed load of 48.6 kN/m applied to the beams. The section sizes selected for the individual elements are shown in Table 1. These sections have been chosen to ensure the frame (with column spacing equal to 11.5 m) will survive the loss of a column without any progression of failures (excluding plastic yielding). Hence, the section sizes are considerably larger than required to resist normal loading. All sections are class 1 cross-sections (CEN 2006). Therefore, the rotational capacity of the elements is not limited by local buckling.

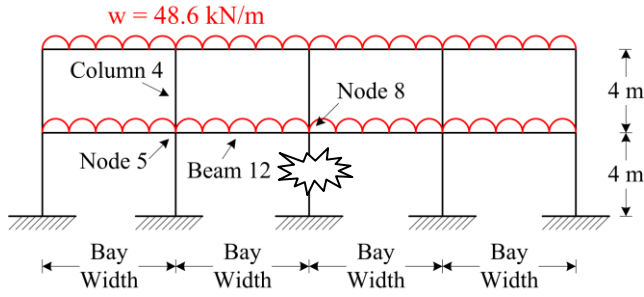


Figure 5. Frame for analysis.

For the purpose of this case study, the connections are modelled as fully fixed and are assumed to be stronger than the members they are connecting. Also, ‘real’ connections may be limited in the degree of rotation that they can undergo before the connection fails completely. This has been intentionally omitted from this analysis, as the authors wish to determine the maximum rotations and forces at the connections.

The response of this structure is computed for the chosen initiating event: removal of the central ground floor column. This is in line with the Department of Defence (2009) and General Services Administration (2003) recommendations, and is consistent with the extent of local failure caused by a minor gas explosion or vehicular collision.

Once the static displacements are computed, and the selected member is removed from the structural model, the time-stepping routine is applied, allowing the loads to dynamically redistribute throughout the structure. Simultaneously, the structure is monitored for opening/closing of plastic hinges and for member failure (as described in section 3.5). The analysis has been performed for a period of 0.4 seconds, which for the structure chosen is adequate time for the response to stabilise.

4.1 Response for Column Spacing = 10,5m

There are many possible failure sequences which may arise for the given initiating event. Figures 6(a)-(d) show the progression of failures predicted by this analysis program, for the limit states described in the previous sections and for bay width 10,5 m.

At $t = 0$ s, the failure sequence is initiated by removing the central ground floor column. As a result, the vertical displacement of node 8 increases rapidly and the loads start to redistribute throughout the structure. This increased deformation results in increased bending moments in the beams: particularly in the central two bays of the structure, where plastic

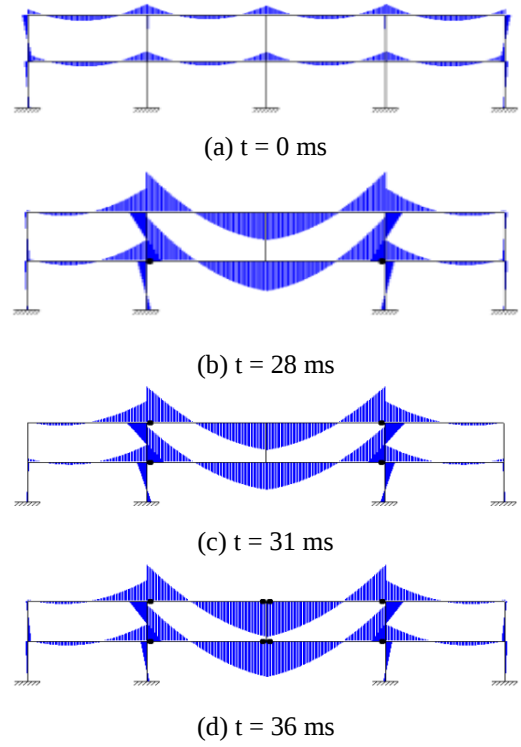


Figure 6. Response for Column Spacing = 10,5 m.

hinges begin to form at their ends (Figure 6(b)-(c)). Following the formation of these hinges, further bending occurs in the centre of the beams and additional plastic hinges form (Figure 6(d)).

Studying the configuration of plastic hinges in Figure 6(d), an unstable beam mechanism appears to have formed. However, this is not the case. As a result of the significant mid-span displacements, the applied loads are no longer resisted in bending, and instead are resisted by second order catenary forces. These catenary forces vary over time, as the structure continues to redistribute dynamically, and may ‘drag-down’ the remaining structure. Therefore, it is important to quantify the magnitude of these forces, and ensure the connections can resist them.

4.2 Effect of Column Spacing

A study has been undertaken to investigate the sensitivity of the deformations and internal forces to the grid dimensions selected. This is achieved by varying the column spacing of the frame shown in Figure 5. The selected bay widths are 9,0 m; 10,0 m; 10,5 m; 11,0 m and 11,5 m. The results of this study are presented in this section.

Figure 7 shows the vertical displacement at node 8 as time progresses. Once the initial plastic defor-

Table 1: Section sizes used in the analysis

Section Size	f_y (MPa)	E (GPa)	L (m)	A (cm ²)	I (cm ⁴)	M_p (kNm)	N_c/N_t (kN)	V_c (kN)
305x305x137 UC	275	210	4	174	10 700	632,5	4 785,0	784,3
838x292x176 UB	275	210	-	224	246 021	1 873,0	6 160,0	1 963,0

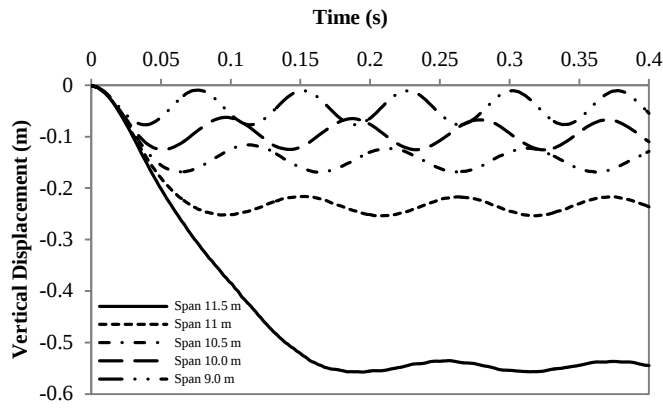


Figure 7. Vertical displacement of node 8

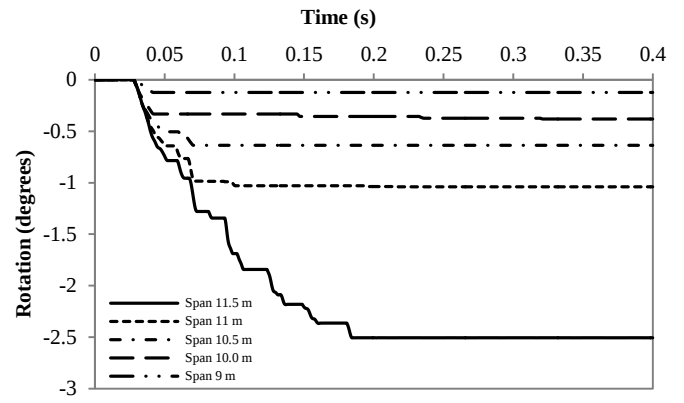


Figure 8. Rotation of connection at node 5

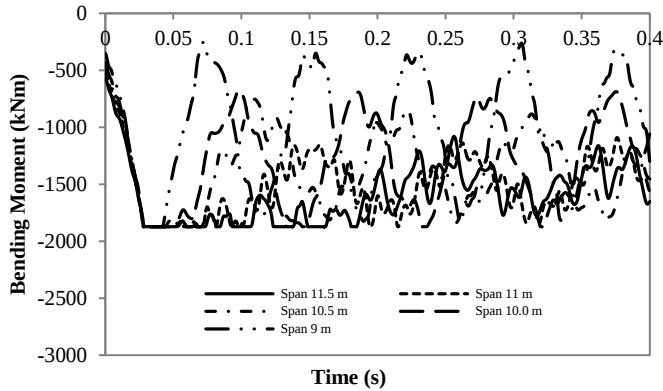


Figure 9. Bending moment at left end of beam 12

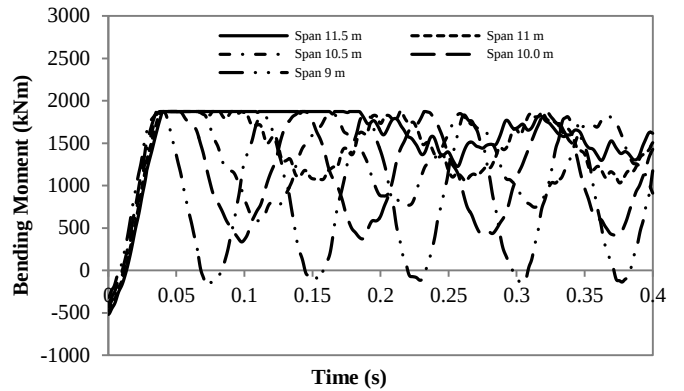


Figure 10. Bending moment at right end of beam 12

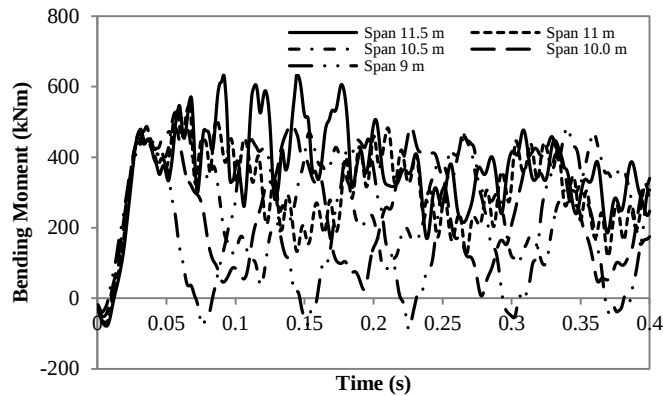


Figure 11. Bending moment at bottom of column 4

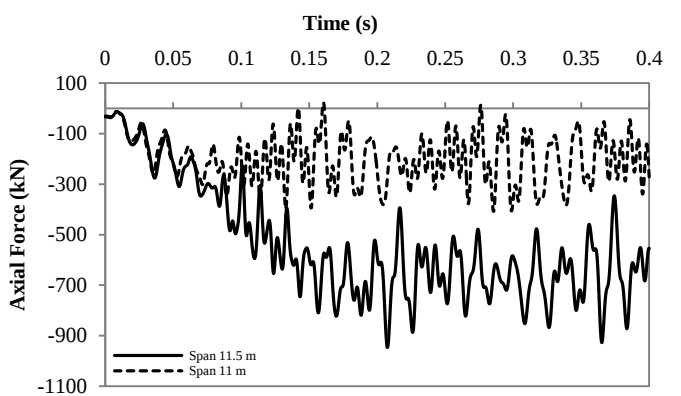


Figure 12. Axial Force in beam 12

mations have taken place, these can be seen to vibrate dynamically with approximately equal period. As the column spacing increases, the peak displacement observed at node 8 also increases. This agrees with the predicted behaviour. However, the internal forces and bending moments are not as straightforward to predict. Figure 8 plots the rotation observed in the connection at node 5: where the connection rotation is taken as the rotation of the beam at the connection, relative to the rotation of the column at the same point. The connection rotation can be seen to increase with increasing beam span, following a similar relationship to that seen in Figure 7.

Figure 9 and Figure 10 graph the bending moments experienced at the ends of beam 12. The bending moments in this element are initially similar for the five frames. At $t = 0.05$ s, the configuration

of plastic hinges shown in Figure 6(d) has formed in all of the structures. For a column span of 9.0 m the plastic hinges close relatively soon after their formation. As a result of this short 'catenary phase', the connection rotation observed for this structure is small in comparison with that for the other frames. As the column spacing increases, the duration of the 'catenary phase' increases accordingly (as the structure is closer to its capacity) and hence an increase in the connection rotation is observed.

Figure 11 shows the bending moments experienced at the top of column 4. These moments are significant, with the peak values reaching more than two-thirds of their plastic moment capacity. For larger column spans, the mean value of the bending moment can be seen to continue increasing throughout the 'catenary phase'.

Finally, Figure 8 describes the axial forces experienced by beam 11 for column spacing 11,0 m and 11,5 m (only two configurations are plotted for clarity). The presence of tensile catenary action is evident in both cases, resulting in a rapid increase in the axial force following formation of the plastic hinge configuration shown in Figure 6(d). It can be seen that the magnitude of the peak tensile catenary force is dependent on the duration of the 'catenary phase'.

5 CONCLUSIONS

This paper has presented a methodology to analyse the vulnerability of a structure to progressive collapse. The computer program developed can be used both as a design and an analysis tool, providing a simple method of assessing both existing and new buildings. By systematically considering the effects of damage to all members in a structure, this algorithm can be used as both a design and an analysis tool to identify whether a structure is unduly sensitive to the effects of localised damage. Software of this nature is a key tool for engineers when ensuring a structure possesses adequate excess capacity to resist the effects of unforeseen loading conditions.

This paper presents the results of a preliminary study to determine the effect of column spacing on the response of a damaged structure. The effect of column spacing on the displacements and internal forces following localised failure has been studied. The results show significant bending moments in the columns, as well as large rotations and axial forces once the bending capacity in the beams has been exceeded.

This case study is part of a comprehensive study of the increased displacements and internal forces which may arise following localised failure, being undertaken by the authors. As part of this research, the analysis program developed is being employed to study the increased displacements and internal forces which may arise in steel-framed structures, following the sudden loss of a key element. The sensitivity of the response to variations in the material properties and the overall structural geometry will be assessed. The critical structural components in a collapse sequence will also be identified. This information may be utilised by designers to ensure a structure possesses sufficient robustness to guarantee safety of its occupants if an extreme event were to occur.

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