Vertical Incremental Dynamic Analysis for Assessing Progressive Collapse Resistance and Failure Modes of Structures

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Abstract: For unforeseen loads, although many design codes at present have recognized the importance of structural robustness for preventing progressive collapse, it has been still lack of guidelines and procedures since they don't give detailed specifications or design methods. As for strong earthquakes, although the present seismic design codes have particular requirements and analysis approaches for preventing progressive collapse, one can only get qualitative results, rather than quantitative ones.

To investigate on the failure modes of vertical progressive collapse as well as progressive collapse resistance of structures, the vertical nonlinear dynamic analysis (NDA) is firstly utilized to intuitively observe the response of the area adjoined to initial damage of structures after some elements are removed, in which the removing instant, removing duration and the location of element removing are considered to be varied. After that, a vertical incremental dynamic analysis (IDA) method is further developed based on the vertical NDA to capture the ultimate bridging-over capacity and impact resistant ability of the areas upon and below the removed elements. The progressive collapse failure modes are analyzed, and the progressive collapse resistance of the structure is assessed by the proposed methods, respectively.

Keywords: Vertical NDA, Vertical IDA, Progressive Collapse, Bridging-over Capacity, Failure Modes

1. Introduction

The robustness of a structure is the ability of the structure to withstand local damage that may arise by accidental actions without disproportional failure that is disproportionate to the triggering cause. Progressive collapse is such a disproportional failure, which refers to the condition when the failure of a local component (or localized region) leads to global system failure, and the final failure state of the structure is disproportionate to the triggering local failure.

The interests in the topic of structural robustness have dramatically increased as a result of the terrorist bomb attack on the Murrah Federal Building in Oklahoma City in 1995 (Corley, et al, 1998) and the attack on the World Trade Center in New York in 2001(NIST; 2005). Since then, many provisions for preventing progressive collapse of structures have been issued in the International Code Council (International Code, 2000) and the Interagency Security Committee (ISC, 2001). In these documents, the concept of progressive collapse is defined more clearly, static or dynamic analysis of progressive collapse of structures is proposed. Based on the above codes, more detailed guidelines for preventing progressive collapse of federal public buildings and military facilities are issued in "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects"(GSA, 2003) and "Design of Buildings to Resist Progressive Collapse" (UFC, 2005). The concepts, design methods and process, implementation

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steps for preventing structures from progressive collapse are more systematically expounded in GSA and in UFC.

Since the 1980s, some general requirements for enhancing the overall stability of the structure have been proposed in various Chinese design codes, standards and specifications of building structures. For instance, In clause 1.0.7 of "Unified Standard for Reliability-Based Design of Building Structures" (China, 1984), it was required that "the overall stability of the structure should be kept on while and after an unpredictable accident event happens", in which the overall stability of the structure was defined as "there is only local damage other than overall collapse of a structure if it is attacked by accident loads". For another example, in clause 3.1.6 of "Code for Design of Concrete Structures" (China, 2002), it was specified that "the structure should be overall stable, and the local damage should not result in global progressive collapse." Although there are many design codes in China which have recognized the importance of structural overall stability for preventing progressive collapse, they are still difficult to operate in engineering practice, since the detailed specifications and operable design methods are not given in these codes.

Despite of many significant theoretical, methodical, technological advances over the recent years, structural robustness for preventing progressive collapse is still an issue of controversy and poses difficulties with regard to its interpretation as well as regulation. The requirements are still not substantiated in further detail, nor has the engineering profession been able to agree on an unequivocal interpretation of robustness which facilitates its quantification.

To quantitatively assess the robustness of resisting local damage and the potential of progressive collapse in the design of building structures, a number of researchers have studied the approaches for progressive collapse analysis (PCA) based on the alternative load path (ALP) method (Kaewkulchai and Williamson 2004, Grierson et al 2005, Kim and Park 2008, Sasani and Nicki 2008, Kim and Kim 2009, Mohamed 2009, Talaat and Mosalam 2009, among others). With the ALP method, one or more load carrying members are assumed to fail and are removed from the structural model for the purposes of analysis. The remaining structure is then analyzed to determine if other member failures result. The analysis for progressive collapse based on the ALP method is carried out using four increasingly complex procedures (Marjanishvili 2004, Marjanishvili and Agnew 2006): linear-elastic static (LS), linear-elastic dynamic (LD), nonlinear static (NS), and nonlinear dynamic (ND). Linear-elastic static analysis (LSA) procedure is the simplest and easiest to perform. However, it is limited to relatively simple structures where both nonlinear effects and dynamic response effects can be easily and intuitively predicted. The linear-elastic dynamic analysis (LDA) procedure can account for internal dynamic loading effects coupled with the effects of higher modes of vibration. However, it cannot account for material and geometric nonlinearity, which could be significant in complex structures where structural yielding patterns cannot be easily identified. The advantage of the nonlinear static analysis (NSA) procedure (i.e., pushdown analysis) is its ability to account for nonlinear effects, and to determine elastic and failure limits of the structure. The disadvantages of this procedure are the inability to consider dynamic effects and the fact that the analysis can become time-consuming due to convergence issues. The nonlinear dynamic analysis (NDA) procedure is the most thorough method of progressive collapse analysis in which a primary load-bearing structural element is removed dynamically and the structural material is allowed to undergo nonlinear behavior. Furthermore, it can account for larger deformations and energy dissipation through material yielding, cracking, and fracture.

Although a nonlinear dynamic analysis procedure can be used when applying the alternate load path method, how to perform such an analysis still relies heavily on engineering judgment. Furthermore, the problem of column removal and its effects is very complex. Few researchers have considered the effects of the duration and the instant of element removing on damaged structures. In addition, how to evaluate the ultimate bridging-over capacity and impact resistant ability of the areas upon and below the removed elements still poses a difficult problem that has been explored by few researchers. In this paper, the vertical nonlinear dynamic analysis (NDA) is firstly utilized to intuitively observe the response of the area adjoined to initial damage of structures after some elements are removed, in which the removing instant, removing duration and the location of element removing are considered to be varied. And then, a vertical incremental dynamic analysis (IDA) method is developed based on the vertical NDA to capture the ultimate bridging-over capacity and impact resistant ability of the areas upon and below the removed elements. A RC frame structure is taken as a case study, its progressive collapse failure modes are analyzed, and the progressive collapse resistance of the structure is assessed by the proposed methods.

2. Vertical Nonlinear Dynamic Analysis for Progressive Collapse

Vertical nonlinear dynamic analysis for progressive collapse is used as a baseline step to determine the capacity of a structural system when one load carrying member is suddenly lost.

Progressive collapse of a damaged structure is a complicated dynamic effect; therefore, using a nonlinear dynamic method to predict the structural performance is more suitable. The loads that cause progressive collapse of a structure can be divided into two different types: (1) primary load and (2) secondary load. The primary load is the action that causes the structural element to fail. External abnormal loads, such as blast pressure due to explosion and vehicular impact, are the examples of the primary load. The secondary loads result from internal static and dynamic loads and are caused by sudden changes in the load path through the structure geometry caused by abrupt loss of one or many load carrying members.

It can be realistically assumed that the time lapse between primary loading and secondary loading may be very short and thus the two loads cannot be separated. However, in the situation where the definition of primary loading is not clearly understood, it may be best as a first step to analyze the structure under two separate loading conditions (primary and secondary loadings).

In order to simulate the phenomenon that one load carrying member is abruptly removed, the member forces should be suddenly removed after a certain time had elapsed while the gravity load remained unchanged, because once the member (usually a column in the first story) is suddenly removed, the stiffness matrix of the system also needs to be suddenly changed. This may cause difficulty in the analytical modeling process. To avoid this problem, all member forces are obtained first from the structural model subjected to the applied load; then the structure is re-modeled without a column with its member forces (P, V, and M) applied to the structure as lumped forces to maintain equilibrium position as shown in Figure 1; the structure becomes stable at time t_1 and the member force is suddenly removed at time t_2 to initiate progressive collapse. In this way the progressive collapse analysis starts from the moment that the structure is already deformed by the applied load, which reflects the loading situation quite realistically.

For nonlinear dynamic analysis neither guidelines of GSA and DoD recommend to use the dynamic amplification factor, therefore the load combination D + 0.25L was uniformly applied as vertical load in the entire span, as shown in Figure 2. Both Material and geometric nonlinearities are accounted for.

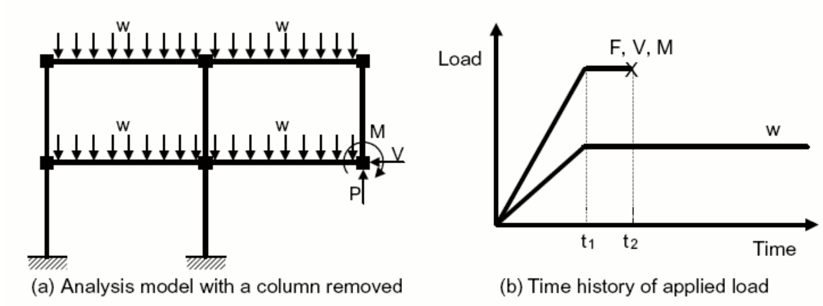


Figure 1. Modeling of sudden removal of a load carrying member

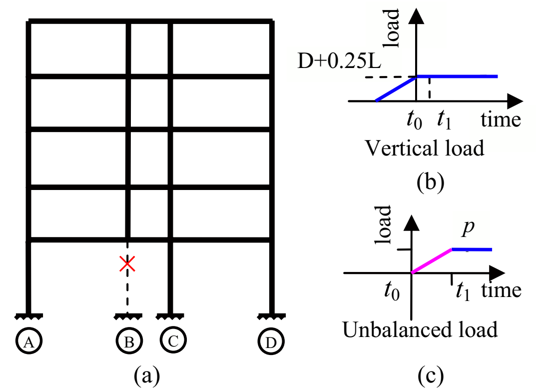


Figure 2. Loading scheme in vertical nonlinear dynamic analysis

The main steps of nonlinear dynamic collapse analysis are (a) apply the gravity load, then (b) remove the column from the unloaded structure, keeping the gravity load constant, and (c) use nonlinear dynamic analysis.

The effects of the duration of element removing are considered by applying the unbalanced force to the remaining structure in sub-steps. The failed column is removed within the time interval (t_0, t_1) as shown in Figure 2, if the duration of element removing is zero, it means that the column fails instantaneously, which corresponds to a rectangular shock load; otherwise, it means that the failure of the element takes a short time, and it corresponds to a shock load that has an ascending segment.

3. Vertical Incremental Dynamic Analysis for Progressive Collapse

The progressive collapse resistance, which is defined as the ultimate downward loading capacity of the column-removed building, is further estimated using the nonlinear incremental dynamic analysis (IDA) method after the vertical nonlinear dynamic analysis is conducted in this paper.

As is well known, the lateral incremental collapse of a structure can be made thorough investigations on by the sideway IDA analysis method. The incremental dynamic analysis (IDA) procedure was firstly proposed by Bertero in 1977, Vamvatsikos and Cornell set up a profound theoretical foundation for it in 2002 (Vamvatsikos and Cornell, 2002). This method was introduced in the guidelines of FAMA350 and

FAMA351 to determine the seismic capacity for preventing lateral incremental collapse of steel frame structures. The IDA process can entail appropriately scaling each strong earthquake ground motion record to cover the entire range of structural response, from elasticity, to yielding, and finally global dynamic instability.

Similar to lateral IDA, in this paper, a series of vertical nonlinear dynamic analyses under different dynamic secondary loadings are conducted. A step function multiplied by D + 0.25L is used to simulate the dynamic loading applied to the column-removed building. The magnitude of the step function is increased gradually till extremely large deflection occurs at the column-removed point. The P-Delta effect and large displacement are considered in the nonlinear dynamic analysis. The peak displacement response of each time history is collected to construct the load-displacement envelopes for the incremental dynamic analysis. The flowchart of the vertical IDA method is summarized in Figure 3. The state of dynamic instability of the structure or the divergence of the program is defined as collapse.

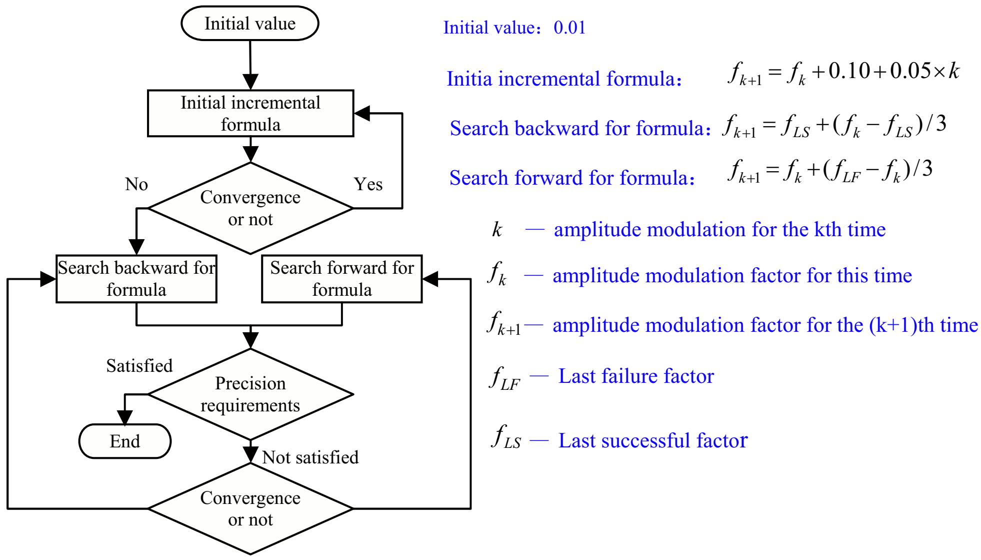
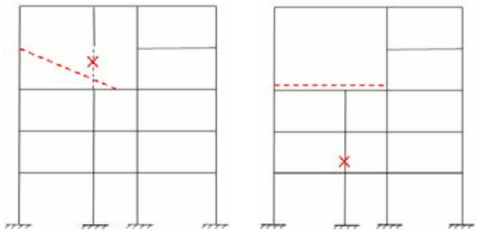


Figure 3. Flowchart of vertical IDA method

To capture the ultimate bridging-over capacity and impact resistant ability of the areas upon and below the removed elements in vertical IDA analysis of this paper, two element-removing scenarios are considered, as shown in Figure 4. The first case is regarding to the remaining structure above the removed element, while the second case is concerned with the remaining structure below the removed element. In the first case shown in Figure 4a, it's assumed that the left inner column in the fourth storey is suddenly lost due to an accidental load, and the beams supported by this column fails and is smashed to the surface of the third floor, then the problem is whether the third floor can survive it. In the second case shown in Figure 4b, it's supposed that the lost column and the beams supported by this column all fall down to the surface of the third floor, and the third floor can survive the impact load, so the load applied on the third floor increases a lot in a moment, and after that, the left column in the second floor is removed also due to accident loads. In this case, the load applying on the third floor is not (D+0.25L), instead it should be amplified by a factor α, where α is a dynamic amplification factor greater than 1.



(a) case 1 (b) case 2

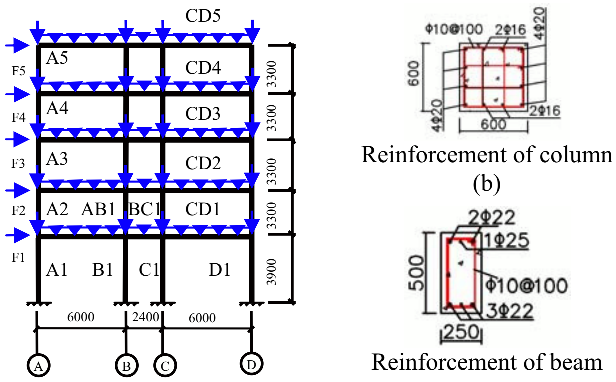
Figure 4. The two scenarios in vertical IDA method

For the vertical IDA analysis of the two actual scenarios in Figure 4, the main difference between them is the loading step. For the first case, we need to remove the lost column firstly, then apply the load α(D+0.25L) to the floor which is below the lost column. For the second case, we need apply the load α(D+0.25L) to the floor which is above the left column in the second floor, then remove this column, and finally apply the dynamic analysis to the damaged structure.

4. Case Study: Progressive Collapse Resistance and Failure Modes of RC Framed Structures

4.1. Design and Modeling of the Example Structure

A 3-bay and 5-storey RC frame is designed according to the current seismic design code of buildings (China, 2001). The elevation of the RC frame and the details of the members are shown in Figure 5, the vertical loads are listed in Table I.



(a)

Figure 5. Elevation and details of 5-storey and 3-bay RC frame

The OpenSees is chosen as the simulation platform. Concrete is modeled by Concrete01, confinement is specified implicitly by using the confined stress-strain relationships proposed by by Mander, Priestley and Park (1988); reinforcing steel is modeled by Steel02 with 1% strain hardening, the material properties used in the simulations are listed in Table II. Beams and columns are modeled by BeamWithHinges elements with co-rotational geometric transformation, two fibers in the cover region and ten fibers in the confined core are adequate to achieve a stable, converged subassembly response, and the length of plastic hinge length is defined as the height of the section of the beam.

Table I. List of vertical loads

Location	Uniformly distributed load (kN/m)		Concentrated load (kN)			
	Dead load	Live load	Edge node		Internal node	
			Dead load	Live load	Dead load	Live load
Head floor	12.09	1.10	121.0	7.0	149.6	10.6
Middle floor	9.35	4.40	102.3	28.1	122.9	42.5
Ground floor	9.35	4.40	105.4	28.1	126.0	42.5

Table II. Parameters of concrete material properties (C35)

		Peak stress	Peak strain	Ultimate stress	Ultimate strain
Unconfined	column	-29.76	-0.0018	0.0	-0.0046
Confined	column	-37.65	-0.0046	-12.50	-0.0271
Confined	beam	-36.20	-0.0045	-11.73	-0.0278

Note: The unit of stress is MPa.

4.2. VERTICAL NDA ANALYSIS

To evaluate the progressive collapse potential of the model structure, which was designed according to the conventional design code without considering progressive collapse, the vertical nonlinear dynamic analyses are carried out in this section.

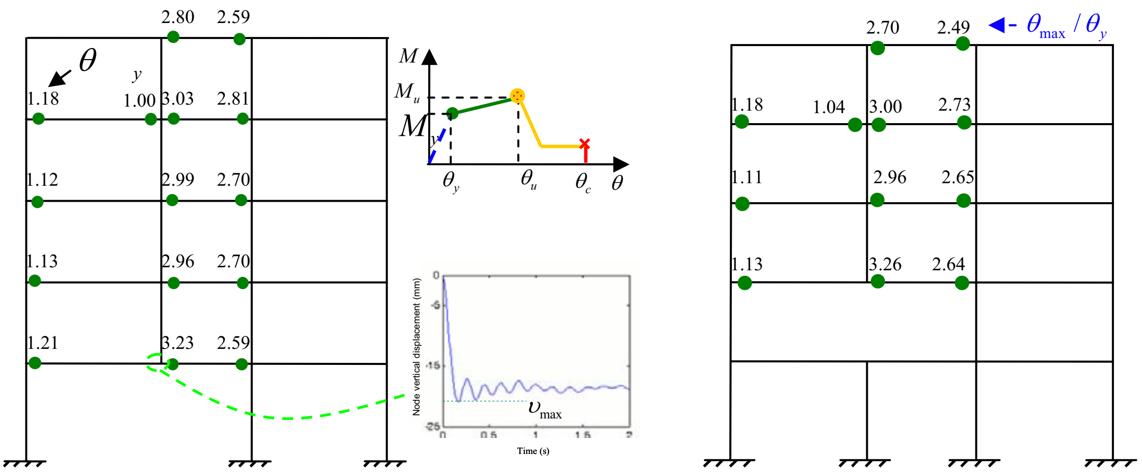
In the vertical nonlinear dynamic analysis of this paper, the yielding of the rebars in tension means that the cross-section undergoes the initial plastic stage, corresponding to the yielding state of the structure; the cross section rotation is defined as peak rotation when the strain of core concrete at the edge reaches its ultimate strain, whose corresponding state is defined as the ultimate state; the state of instability of the structure or the divergence of the program is defined as collapse.

4.2.1. *The overall response with respect to the development of plastic hinges*

The distributions of plastic hinges of the damaged structures assuming column B1 or column B2 removed are shown in Figure 6. It can be observed that the failure mode of the structure is a strong-column weak-beam (SCWB) type after the left internal column in the first floor is lost, and that the plastic hinges don't occur at the end of columns. The plastic hinges developed at the ends of beams (beam AB and beam BC) of the damaged bays on some instants: at time $t = T/4$, the plastic hinges occurred at all the ends of beam BC; at time $t = T/2$, they occurred at the remote ends of beams AB1, AB2, AB3, and at both ends of beam AB4; at time $t = T$ and $t = 1.25T$, the maximum angles of columns reached, and the maximum angles of beam BC is about 0.009rad, which is about three times as the yielding angle, and the maximum angles of beam AB is about 0.004rad, which is very close to the yielding angle.

4.2.2. *The local reactions with respect to internal forces of the adjacent members*

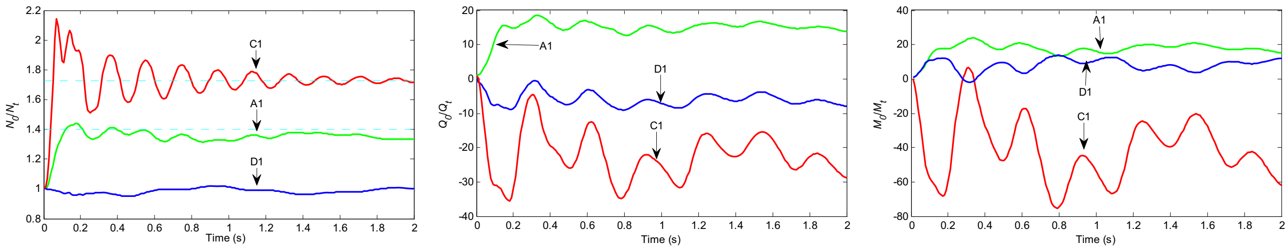
Vertical NDA analyses are applied to the damaged structure assuming column B1 removed. The influence on the remaining structure caused by the lost column B1 is analyzed primarily from the changes of internal forces of the columns which are on the same floor with the lost column, from the changes of internal forces of columns which are in the same column line with the lost column, and from the changes of internal forces of the beams supported by the lost column. The analyses results are shown in Figure 7.



(a) Distributions of plastic hinges of the damaged structures assuming column B1 lost

(b) Distributions of plastic hinges of the damaged structures assuming column B2 lost

Figure 6. Distributions of plastic hinges of the damaged structures



(a) (b) (c)

Figure 7. The internal forces of the remaining columns which are on the same floor with the lost column before and after the failure of column B1

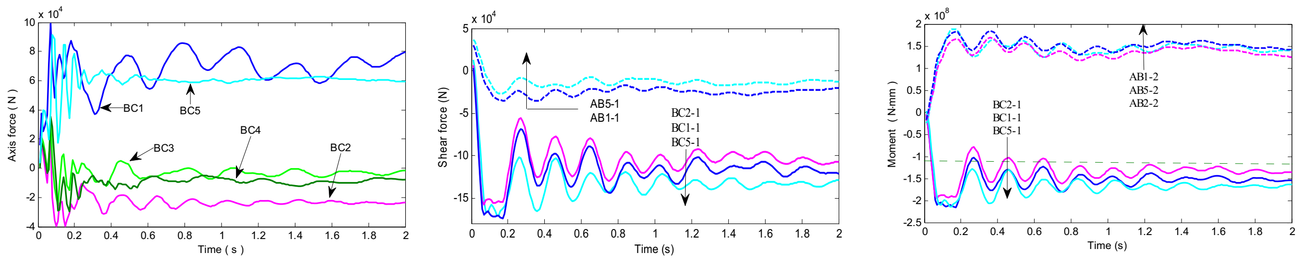
The ratio of the internal force γ in Figure 7 is calculated by

$$\gamma = \frac{R_0}{R_t} \tag{1}$$

where R_0 is the force of the column before the failure of column B1, R_t is the force of the column after the failure of column B1.

As shown in Figure 7, the loads originally resisted by the lost column suddenly transferred to the adjacent column C1, so that the axial force of column C1 soared to twice as much as its original axial force, this is because the loads originally resisted by the two columns B1 and C1 are redistributed to column C1 now. A part of the axial force of column C1 transferred to column A1 gradually, since the axial force of column C1 descend three times in the first vibration, and the axial force of column A1 reached to the peak value for the first time before the third decline of the axial force of column C1, and then the two columns vibrated synchronously. However, the axial force of column D1 has no change in the whole process. It can be observed from the shearing force diagram and bending moment diagram that the vibration of each column is consistent, the remaining columns on the ground floor vibrate synchronously, the amplification of shearing force is approximately half of the amplification of bending moment for each element; the internal force of column C1 is magnified the most in the reverse direction, and the vibration amplitude of column C1 has a similar fluctuations in the cosine wave, this is because the st□ucture also vibrate in the horizontal direction. Contrary to the axial force, the moment and shearing force of column D1 vibrate and increase obviously, but the direction of vibration is contrary to the column A1 and column C1, this fact precisely proves that column D1 plays as a support which absorbs the unbalanced force of the structure. Therefore, it can be concluded that the main response of the damaged structure is the vertical vibration in the damaged bays after column B1 is removed; at same time, the damaged structure also vibrate in the horizontal direction, this is mainly due to the geometric asymmetry of the structure itself after column B1 is removed.

The changing trends of the internal force of the beams which are originally supported by the lost column are shown in Figure 8. It can be observed that, the moment of the beams increased a lot after the failure of the column B1 (it is 200-500 times as much as the bending moment of the beams which is under the gravity load), this is because: (1) the releasing of the bending moment of column B1; (2) the changing of the geometry of the structure, i.e., the span of the beam increases suddenly after the failure of the column B1, which leads to the increased bending moment of the beam; (3) the concentrated load which originally resisted by column B1 now become a external load applied on beam ABC, this produces greater extra bending moment in the beam (the bending moment of the beam changed from zero to a large value). The latter two of the three reasons are the main reasons. There is a platform in the diagram of shearing force and bending moment of beam BC, This is because the axial vibration of remaining columns in column line B stabilizes until several cycles (see Figure 9), which leads to the vibrations of the shearing forces of the beams which are on the same floor with the lost column, so the axial force of adjacent columns (mainly column C1) arrived at peak value for several times during the first period (see Figure 7). The bending moments of beam AB, BC are nearly the same, it is because they must satisfy the continuous condition since the bending moment of column B2 is almost zero; but the shearing force of beam BC is almost three times that of beam AB, it is because the location of concentrated load is near the node C.

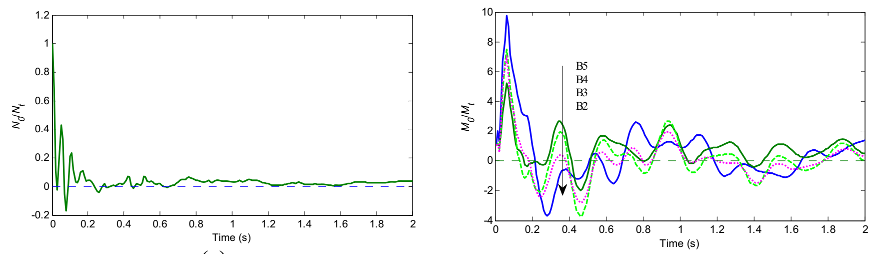


(a) (b) (c)

Note: BC-1 represent the left side of the beam BC, and BC-2 represent the right side of the beam BC.

Figure 8. The moments of the remaining beams which are on the same floor with the lost column before and after the failure of Column B1

The internal forces of columns in column line B on each floor are shown in Figure 9. The variations of axial force of columns in column line B for each floor is the same, so only the axial force diagram of column B5 is shown in Figure 9. The variations of shearing force and moment of columns in column line B on each floor are also similar to each other, although the shearing force is much smaller than the bending moment, so only the bending moment diagrams of columns in column line B are shown in Figure 9. It is observed from Figure 9 that, the axial forces of the remaining columns in column line B release quickly to zero after the failure of the column B1; the period of axial vibration of columns is shorter than that of the bending vibration of beams, the column vibrates several cycles before the beam finished the first vibration cycle, so the axial forces of column C1 arrived at peak value several times during the first period. This can be explained that, structural vibration must firstly satisfy the force balance conditions, then satisfy the boundary constraint condition. This means that, the structure should first resist the load caused by the shear vibration of beams (the force equilibrium conditions), then it achieves stable state by the bending vibration of beams (the deformation coordination conditions); Although the bending moment of each column in column line B is magnified many times when column B1 is removed, they vibrate near zero in the last, what's more, the variation range of the bending moment of columns is small, the columns vibrate synchronously except that column B2 vibrate in the opposite direction after the third period.



(a) (b)

Figure 9. The internal forces of the remaining columns in column line B before and after the failure of Column B1

4.3. VERTICAL IDA ANALYSIS

The two actual scenarios in Figure 4 are analyzed by vertical IDA analysis, in which the different loading steps are applied to the structure accordingly.

4.3.1. *Vertical IDA analysis for the substructure above the removed column*

The ultimate ability of the damaged structure assuming column B1 removed is analyzed by vertical IDA, which is corresponding to the first scenario. The resulted IDA curve and the corresponding pushdown curve are shown in Figure 10.

The vertical displacement shown in Figure 10 is the absolute of real displacement. It is observed that, the IDA curve is under the pushdown curve all the time, the limit load and the limit displacement from vertical IDA are both smaller than that obtained by pushdown analysis. For the same load factor, the displacements obtained by the two methods are very different from each other at the lower level, but the gap between them reduced gradually with the increasing load factor; Similarly, for the same displacement, the load factor obtained by the two methods are very different from each other at the lower level, but the gap between them reduced gradually with the increased displacement. The distributions of plastic hinges of the damaged structures just before it goes into instable state are shown in Figure 11.

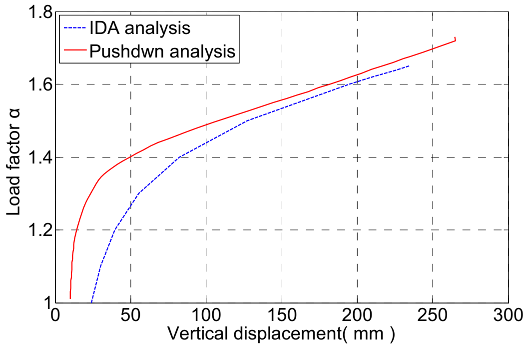
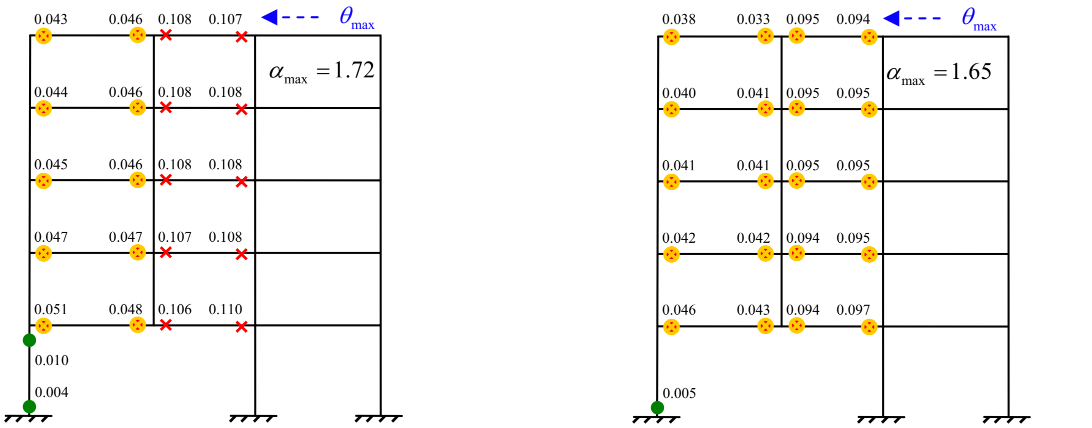


Figure 10. Comparison of IDA curve and pushdown curve



(a) The failure mode of the structure obtained by pushdown analysis (b) The failure mode of the structure obtained by IDA

Figure 11. Plastic rotations of the structure in the instable state

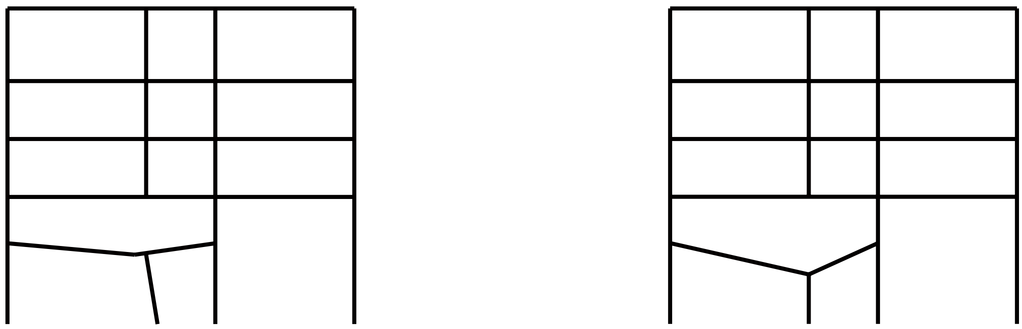
The load factor is used to evaluate the ultimate ability of the damaged structure. The load factor for the model structure is 1.65. It needs to be greater than 2 to ensure that the structure won't collapse if there are two or more floors failed. So, if there is one floor of the structure collapsed, then the structure will collapse when a column below the collapsed structure is removed, and the range of collapse is between the floor which has collapsed before and the floor which is supported by the lost column. For example, if the third floor has collapsed, and the column on the ground floor is removed, then the floors between the ground floor and third floor will collapse. A mega-frame structural system is recommended in the design for high-rise buildings, because it is useful to improve the ultimate ability of the structure if the structure is segmented into several parts in the vertical direction.

4.3.2. *Vertical IDA analysis for the substructure below the removed column*

The dynamic effects are considered by applying 2(D+0.25L) to the damaged bays which are just below the lost column according to the provisions of UFC. However, it is found in this paper that, the vertical load can achieve a considerable value if there is no vertical component below the lost column failed, the maximum dynamic load that the substructure which is below the lost column can resist is obtained in the vertical IDA analysis.

Vertical IDA is applied to the structure assuming column B2 lost instantaneously, the maximum dynamic load that the substructure which is below the lost column B2 can resist is analyzed. The loading scheme of the vertical IDA is as follows: firstly, gravity load is applied to the structure; then the column B2 is removed; finally, the load α(D+0.25L) is applied to the beams AB1and BC1 in a dynamic way.

It is observed from Figure 12 that, the failure of the substructure is caused by the instability of the column which is under pressure. The load factors obtained by pushdown and vertical IDA are both more than 60 times the design load (which largely exceeds the value stated in UFC), so the ability of the designed structure is enough to prevent from progressive collapse.



(a) The failure mode of the substructure below the removed column obtained by pushdown analysis

(b) The failure mode of the substructure below the removed column obtained by vertical IDA

Figure 12. The failure modes of the substructure below the removed column

It is seen that the ultimate loading capacities are developed at the deflections approximately to that obtained from the nonlinear static analysis. This is due to the fact that identical plastic hinge models are adopted for the nonlinear static and nonlinear dynamic analyses.

4.4. THE CONCLUSIONS OF THE CASE STUDY

The conclusions can be drawn from the case study as follows:

1. Through the response of the damaged structure assuming one column removed by vertical NDA, it is found that the structural damage only happens in the damaged bays, where plastic hinges occur at the ends of the beams, and the plastic deformations are rather small, there isn't any plastic hinge which occurs in columns, so the design structure will not collapse.
2. The internal forces of the remaining columns in column line B is unloaded to zero quickly after column B1 is removed, this means that the columns in column line B have lost their ability to resist the applied load, so the two adjacent beams supported by those columns become one large-span beam, the motion of each layer is consistent with each other.
3. Vertical IDA analysis is applied to the damaged structure, the ultimate load factor of 1.65 is obtained, it means that, if one floor of the structure has collapsed, then the structure will collapse when a column below the collapsed floor is removed, and the damage range is between the collapsed floor and the floor which is supported by the removed column.

4. Through the vertical IDA analysis applied to the damaged designed structure, it is found that there is considerable space for the increasing external load, the structure will collapse due to the instability of the column which is under pressure, and the ultimate load factor obtained is 63.5. It means that, the structure has the ability to resist the dynamic impact caused by the collapse of upper structures as long as the critical member below the lost column does not fail.

5. Conclusions

The vertical nonlinear dynamic analysis (NDA) is used as a baseline procedure to assess the potential of progressive collapse of a damaged structure based on the alternative load path approach in which one or more load carrying members are assumed to be removed suddenly. And then, a vertical IDA is further developed to evaluate the ultimate bridging-over capacity and impact resistant ability of the areas upon and below the removed elements. Applying the two approaches to a RC frame structure as a case study demonstrates the applicability of the two methods. Nonlinear incremental dynamic analysis is a promising method for estimating the progressive collapse resistance of building structures. Nevertheless, it is time-consuming to perform the incremental dynamic analysis for the maximum loading capacity. As a consequence, a more efficient and simplified approach may be needed to develop to predict the progressive collapse resistance and robustness of structures precisely in the future. Furthermore, the various uncertainties should be accounted for in vertical NDA and IDA in order to develop reliability-based and robustness-based design procedures for progressive collapse limit state design.

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