Fall November, 2011

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J. Wind Eng. Ind. Aerodyn. 99 (2011) 1162–1173

Contents lists available at SciVerse ScienceDirect

Journal of Wind Engineering and Industrial Aerodynamics

journal homepage: www.elsevier.com/locate/jweia

Evaluation of wind load integration in disproportionate collapse analysis of steel moment frames for column loss

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ARTICLE INFO

Article history:
Received 17 December 2010
Received in revised form
16 September 2011
Accepted 16 September 2011

Keywords:
Wind loading
Lateral load
Robustness
Disproportionate collapse
Alternate path method
Damaged steel frames
Column loss

ABSTRACT

The design of steel structures, in most cases, depends majorly on the level of wind loads which are prescribed by codes and regulations and are used in the structural analysis due to the fact that steel structures being light and ductile systems are strongly affected from a slight difference in the values of wind loading. During the last decades, disproportionate collapse analysis has become of major interest mainly due to the increasing number of failures occurring in that pattern. Commonly accepted guidelines and methods of analysis have been produced, the most dominating of which being the Department of Defense Facilities criteria or DoD. In the DoD, as well as in other criteria, the event of a column loss is suggested as the modeling scenario which has to be sustained by a structural system in order to be robust. However, all the guidelines so far have disconnected the column loss analysis from wind loads and have only performed it for gravity loading. This paper presents the dynamic time history disproportionate collapse analysis of steel frames, including various levels of wind loading. Interesting aspects are discussed through the parametric analysis of five different numerical examples of moment resisting frames.

1. Introduction

Disproportionate or progressive collapse has developed during the last decades as a common pattern of structural failures which has appeared in many incidents such as the failure of the 22-storey building in Ronan point in 1968 or the World Trade Center disaster in New York in 2001 (ASCE, 2009). The main characteristic of a disproportionate collapse is the disanalogous extent of consequences produced by a triggering event. Often, the consequences are not only restricted to structural components but can also extend to societal, financial, environmental or other fields.

Until today all structural systems are designed according to regulations and standards which concern normal loading conditions. That is why the calculation of the resistance of a structure to disproportionate collapse is a very tedious task since the phenomenon is strongly related to abnormal conditions and rare events. Such events that have been identified so far by the researchers are gas explosions, terrorist attacks, accidental impact of vehicles, unpredicted fire, extreme wind and many others (National Institute of Standards and Technology, 2007; Deodatis, 1997; Papadopoulos and Deodatis, 2006; Baker et al., 2008; Ellingwood et al., 2007; Ellingwood and Leyendecker, 1978; Ellingwood and Dusenberry, 2005; Izzuddin et al., 2007; Starossek, 2009; Gerasimidis et al., 2008, 2009; Dubina et al., 2010; Frangopol and Curley, 1987; Kwasniewski, 2010; Foley et al., 2007; Kim and Kim, 2009; Gerasimidis and Baniotopoulos, 2011; Gerasimidis, 2011). The uncertainties related to such type of abnormal events are apparently significant and therefore relevant loading simulations are very hard to be produced. Even harder, the load combinations between normal loading conditions described in the codes with these abnormal loading conditions are very vague and there are only probability-based limit states of design that have been presented so far by researchers.

In that direction, a very comprehensive document regarding an overview of the load combinations relative to abnormal events which could lead to disproportionate collapse is the report by Ellingwood which describes possible design strategies of national standards comparing ASCE Standard 7 (ASCE 7, 2007), the Eurocode 1 (EN Eurocode 1, 2006) and the National Building Code of Canada (National Research Council of Canada, 1995). All the aforementioned codes recommend the incorporation of dead, live, snow and wind loads in the combinations that should be applied in the design for disproportionate collapse analysis. In the same document, special attention is paid regarding the response of a structure as a system rather than member design.

Recently, the Department of Defense Unified Facilities Criteria [Unified Facilities Criteria (UFC), 2009] probably the most dominating document in disproportionate collapse analysis so far

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doi:10.1016/j.jweia.2011.09.009

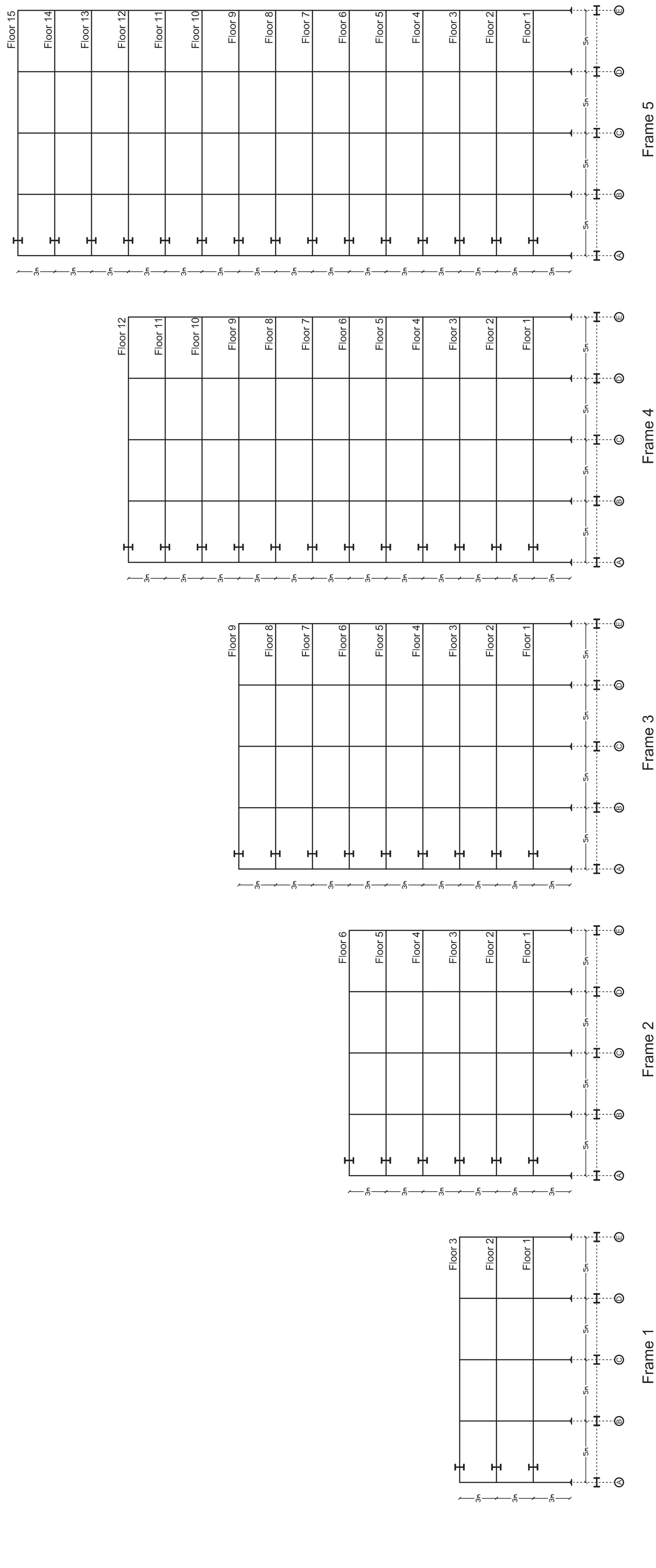

Fig. 1. Geometry of the three frames.

(with US General Services Administration, 2003), has introduced the alternate load path method of analysis as a direct method for the design of buildings regarding disproportionate collapse. The alternate path method is composed of three different methods of analysis: the linear static method, the nonlinear static method and the nonlinear dynamic method of analysis. Although there are major differences among the three methods, the load combinations suggested for all the analyses are commonly defined and consist of dead, live and snow loads. The wind loads are excluded from the design and only a small lateral load is applied on the structure to incorporate imperfection effects in the analysis. However, for steel structures, the loading due to the wind can be the major design parameter mainly due to the fact that steel structures are light and ductile structures and are strongly affected by lateral loads (Stathopoulos and Baniotopoulos, 2007; Glanville and Kwok, 1995; Das Nirmal and McDonald James, 1990; Petrov Albert, 1998; Ballio and Mazzolani, 1983; Stathopoulos et al., 2008).

The alternate path method tunes the structure in order to develop alternate load paths in the event of local damage by keeping the main structural components meeting specific criteria. Damage in the structure is introduced by the removal of specific members of the structure; in the case of steel moment frames, these members are the columns; although the engineering concept of column removal is far from realistic, it is a useful method to simulate a local damage in the structure.

This paper presents the parametric study of five reference steel moment frames of various heights for the case of both corner column removals at their first floor. The loads applied on the structures initially consist of the loads prescribed by the DoD [Unified Facilities Criteria (UFC), 2009] which include dead, live, snow and a small lateral load for the imperfection of the structure. Following these analyses, the same dead, live and snow loads are applied but this time combined with six different levels as parts of the design wind load. The ultimate goal of the paper is to investigate and demonstrate the wind effect on the response of a structure after a column loss, something that is not accounted today at all by the available guidelines. In this sense a comparable criterion used here is the elastic maximum instantaneous values of displacements or forces following the column removal. The results are presented at the end of the paper and various aspects are discussed.

2. Advanced structural models

For the purposes of this work, five different reference steel moment frames were selected for disproportionate collapse analysis including wind loading. The frames are plane and orthogonal with floor heights equal to 3 m and bay widths equal to 5 m. As shown in Fig. 1, the first frame is a 3 floors high steel moment frame, the second has 6 floors, the third 9 floors, the forth 12 and the fifth 15 floors while all five have four bays with a total width of 20 m. The frames were intentionally selected as varying in height in order to investigate possible divergencies in the effect of wind loading depending on the height of the buildings. Fig. 1 shows the geometrical characteristics of the three moment frames.

The design of the frames was carried out according to the Eurocodes (EN Eurocode 1, 2002, 2006) through a computer aided analysis. The material's yield stress was fixed at 235 MPa. The loads used in the design of the frames can be summarized as follows. For all the floors except from the roof, the imposed dead loads were fixed at 28 kN/m^2 and the live loads at 14 kN/m^2. For the roof, the dead loads are 28 kN/m^2, the live loads 5 kN/m^2 and the snow loads 3.45 kN/m^2. The wind loads used in the design will be described in more detail in following sections.

The frame sections used for the design were standard commercial European steel sections and they are summarized in Table 1.

3. Frame wind loads according to IBC 2006

The wind loads applied on the frames have followed the calculation procedure of IBC 2006 (International Code Council, 2006) which applies Chapter 6 of ASCE 7 (ASCE 7, 2007). The analytical procedure of wind load calculation for buildings has been followed. Throughout the procedure it must be noted that frames 1 and 2 which are 3 floors and 6 floors high (9 m and 18 m respectively), are considered as low-rise buildings while frames three, four and five with a total height of 27 m, 36 m and 45 m respectively are not considered as low-rise. This distinction affects the calculation of several coefficients.

Table 2
External pressure coefficients GC_{pf} for frames 1 and 2.

Frames	Building surface			
	1	2	3	4
Frame 1	0.4	−0.69	−0.37	−0.29
Frame 2	0.4	−0.69	−0.37	−0.29

Table 1
Frame sections for the three frames.

Floor	Fr.#1			Fr.#2			Fr.#3			Fr.#4			Fr.#5		
	Corner columns	Middle columns	Beams	Corner columns	Middle columns	Beams	Corner columns	Middle columns	Beams	Corner columns	Middle columns	Beams	Corner columns	Middle columns	Beams
*Floor*15													*HEB*200	*HEB*200	*IPE*500
*Floor*14													*HEB*200	*HEB*200	*IPE*500
*Floor*13													*HEB*200	*HEB*200	*IPE*500
*Floor*12										*HEB*280	*HEB*280	*IPE*360	*HEB*280	*HEB*280	*IPE*500
*Floor*11										*HEB*280	*HEB*280	*IPE*360	*HEB*280	*HEB*280	*IPE*500
*Floor*10										*HEB*280	*HEB*280	*IPE*360	*HEB*280	*HEB*280	*IPE*500
*Floor*9							*HEB*220	*HEB*220	*IPE*360	*HEB*320	*HEB*320	*IPE*450	*HEB*340	*HEB*340	*IPE*500
*Floor*8							*HEB*220	*HEB*220	*IPE*400	*HEB*320	*HEB*320	*IPE*450	*HEB*340	*HEB*340	*IPE*500
*Floor*7							*HEB*220	*HEB*220	*IPE*400	*HEB*320	*HEB*320	*IPE*450	*HEB*340	*HEB*340	*IPE*500
*Floor*6				*HEB*220	*HEB*220	*IPE*360	*HEB*280	*HEB*280	*IPE*450	*HEB*340	*HEB*340	*IPE*500	*HEB*450	*HEB*450	*IPE*500
*Floor*5				*HEB*220	*HEB*220	*IPE*400	*HEB*280	*HEB*280	*IPE*450	*HEB*340	*HEB*340	*IPE*500	*HEB*450	*HEB*450	*IPE*500
*Floor*4				*HEB*220	*HEB*220	*IPE*400	*HEB*280	*HEB*280	*IPE*450	*HEB*340	*HEB*340	*IPE*500	*HEB*450	*HEB*450	*IPE*500
*Floor*3	*HEB*280	*HEB*280	*IPE*360	*HEB*320	*HEB*320	*IPE*450	*HEB*400	*HEB*400	*IPE*500	*HEB*500	*HEB*500	*IPE*500	*HEB*650	*HEB*650	*IPE*500
*Floor*2	*HEB*280	*HEB*280	*IPE*400	*HEB*320	*HEB*320	*IPE*450	*HEB*400	*HEB*400	*IPE*500	*HEB*500	*HEB*500	*IPE*500	*HEB*650	*HEB*650	*IPE*550
*Floor*1	*HEB*280	*HEB*280	*IPE*500	*HEB*320	*HEB*320	*IPE*550	*HEB*400	*HEB*400	*IPE*550	*HEB*500	*HEB*500	*IPE*550	*HEB*650	*HEB*650	*IPE*550

3.1. Basic wind speed and wind directionality factor

The basic wind speed V used for the determination of the design loads has been fixed at 49 m/s and the wind directionality factor K_d has been fixed at 0.85 as for buildings.

3.2. Importance factor

All five frames are assumed to be buildings which fall into the general occupancy category III as defined by the IBC 2006. Therefore, the importance factor is set at 1.15.

3.3. Exposure category

It is assumed that the frames are in exposure category B ground surface conditions. For frames 1 and 2 as they are considered low-rise buildings the coefficients GC_{pf} are given in Table 2. It is also assumed that the frames are intermediate frames and therefore the corresponding building surfaces according to IBC 2006 are 1, 2, 3 and 4.

The velocity pressure exposure coefficient K_z is set at 1.56 for all five frames.

3.4. Topographic factor

It is assumed that $K_{zt}=1$.

3.5. Gust effect factor

The fundamental frequencies of the five frames can be seen in Table 3 and they are all above 1 Hz. Therefore, the gust effect factor G is set at 0.85.

3.6. Enclosure classification

The five frames are categorized as enclosed buildings.

Table 3
Fundamental frequencies of the five frames.

Frames	Period (s)	Frequency (cycle/s)
Frame 1	0.162	6.18
Frame 2	0.242	4.12
Frame 3	0.313	3.19
Frame 4	0.379	2.64
Frame 5	0.459	2.18

Table 4
Design wind loads for the frames 3, 4 and 5 in kN/m.

Frames	Windward	Leeward	Roof
Floor 15	9.79	−6.88	−6.88
Floor 14	9.79	−6.88	
Floor 13	9.44	−6.64	
Floor 12	9.01	−6.34	−6.34
Floor 11	8.57	−6.03	
Floor 10	8.31	−5.85	
Floor 9	8.05	−5.67	−5.85
Floor 8	7.71	−5.42	
Floor 7	7.36	−5.18	
Floor 6	7.01	−4.94	
Floor 5	6.58	−4.63	
Floor 4	6.06	−4.27	
Floor 3	5.72	−4.02	
Floor 2	5.37	−3.78	
Floor 1	4.94	−3.47	

3.7. Internal pressure coefficient

As for enclosed buildings the internal pressure coefficients GC_{pi} are set at 0.18 and −0.18 for toward and away surfaces respectively.

3.8. External pressure coefficients

For frames 1 and 2, see Table 2. For frames 3, 4 and 5 the external pressure coefficients for the walls are set at $C_p=0.8$ for the windward wall and $C_p=-0.5$ for the leeward wall. For the roof, it is assumed that $C_p=-0.5$.

3.9. Velocity pressure

The velocity pressure q_z is calculated as follows:

$$q_z=0.613K_zK_{zt}K_dV^2I$$

3.10. Design wind load

Finally, the design wind load p for frames 1 and 2 is calculated as follows:

$$p=q_h[(GC_{pf}-GC_{pi})]$$

and for frames 3,4,5

$$p=qGC_p-q_i(GC_{pi})$$

The results can be found in Tables 4 and 5.

4. Alternate path method of analysis

4.1. Basics

It has been common engineering ground that the assessment of disproportionate collapse resistance of a structural system

Table 5
Design wind loads for the frames 1 and 2 in kN/m.

Frames	Building Surface areas			
	1	2	3	4
Floor 6	1.88	−7.45	−4.71	−4.02
Floor 5	1.79			−3.83
Floor 4	1.68			−3.60
Floor 3	1.55	−6.13	−3.88	−3.31
Floor 2	1.55			−3.31
Floor 1	1.55			−3.31

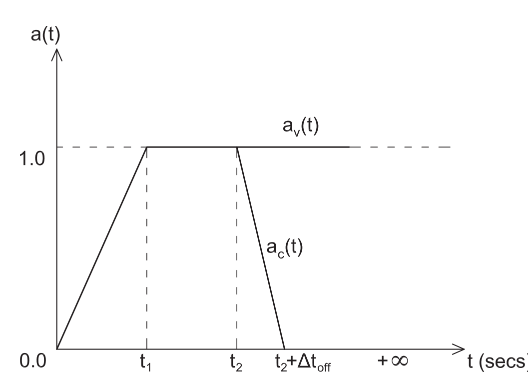

Fig. 2. Time history analysis functions.

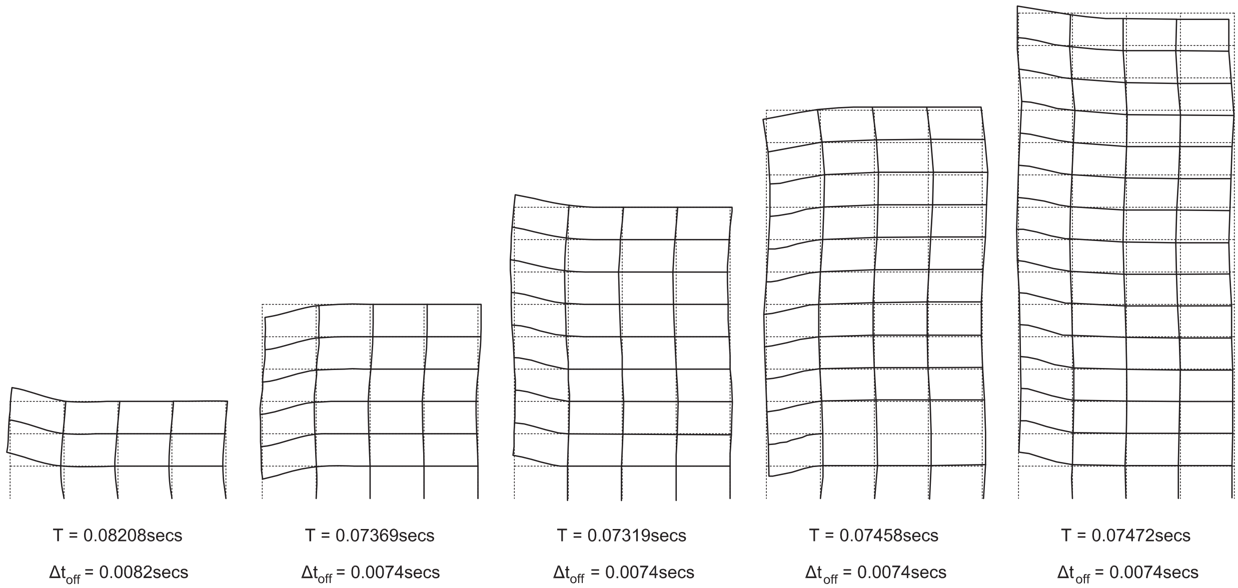

Fig. 3. Periods associated with corner column removals and respective Δt_{off}.

Table 6
Load combinations used for the analyses.

LC	Dead	Live	Snow	Wind
1	1.2	0.5	0	0
2	1.2	0.5	0	0.2
3	1.2	0.5	0	0.4
4	1.2	0.5	0	0.6
5	1.2	0.5	0	0.8
6	1.2	0.5	0	1.0

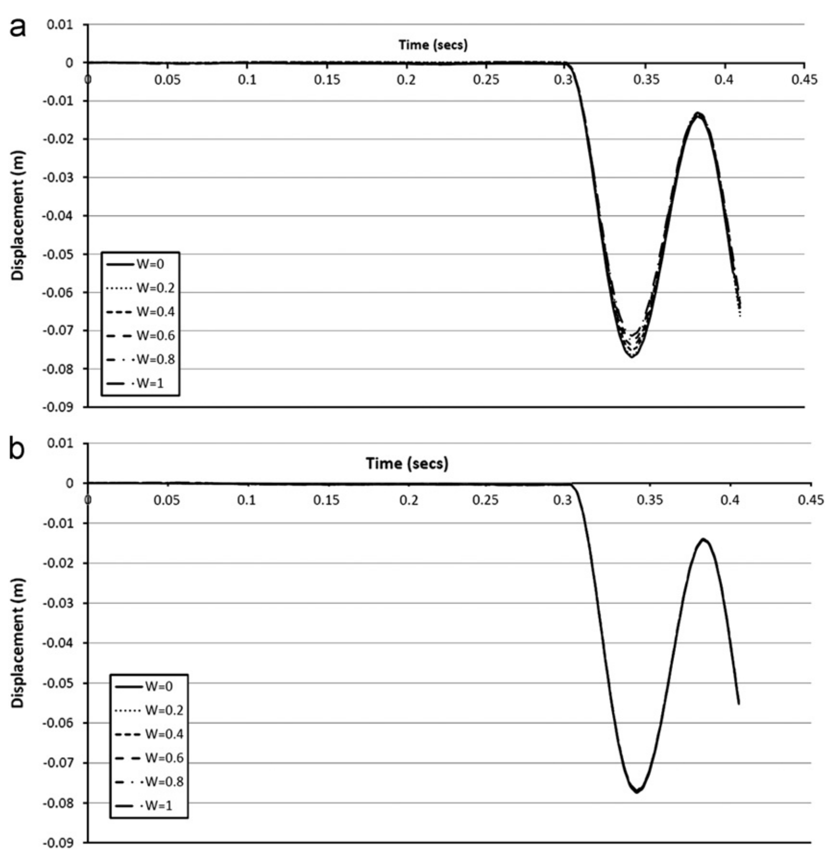

Fig. 4. Frame 1. Displacement at top node of removed column. (a) Windward corner column loss, (b) Leeward corner column loss.

includes two main categories of structural design provisions. The first is the specific local resistance method and the second is the alternate path method of analysis. The latter has been received by the structural engineering community as the closest to engineering intuition and that is why most of the research efforts have followed it.

The method is based on the engineering concept of element removal as a simulation tool of the triggering event that could cause disproportionate collapse. Although not very realistic, since an element is very unlikely to be completely damaged or even in case of an abnormal event a series of elements could experience partial damage, the notional element removal analysis provides very useful conclusions regarding the behavior of a structure in the event of local damage. For the case of steel frames, the method prescribes the removal of the columns of the structure one by one in turn and the respective check of the structure's ability to sustain the introduced damage.

For the case of the five frames, the two corner columns of the first floor have been selected to be removed one by one. It has been noticed (Foley et al., 2007; Kim and Kim, 2009) that the corner columns of a frame are the most critical in disproportionate collapse analysis. Therefore, for the five frames of the present study the windward corner column of the first floor is firstly removed and the leeward corner column of the first floor is removed as a different column removal scenario.

4.2. Time history dynamic method of analysis

The structural analysis involved in the alternate method as described in detail in Unified Facilities Criteria (UFC) (2009) includes three different methods of analyzing a structure for the event of disproportionate collapse. Firstly, a simplistic linear static method is described, secondly a nonlinear static method and lastly a dynamic method of analysis is presented. It has to be noted that in most events of disproportionate collapse analysis, the initial damage of the structure occurs in a rather dynamic pattern as a short-lived and very high load and therefore the dynamic method of analysis is considered by many researchers as the most accurate method of analysis.

The work presented in this paper includes the application of the dynamic method of analysis incorporating time history analysis for the removal of the column. As presented in Foley et al. (2007), the removal of the column from the structural model can be simulated as turning off the column from the structure in an appropriate amount of time in order to take into account the dynamic effect of the failure.

The time history functions used for the removal of the column are shown in Fig. 2. Firstly, the element is removed from the model and the element's forces are applied in the opposite direction to the remaining structure. These forces are can be described by the function $a_c(t)$ and the rest of the loads are described by the function $a_v(t)$. In Fig. 2, the loading is applied on the structure in the first $t_1 secs$ and then the removal of the column starts at $t_1 secs$ and lasts for Δt_{off} which defines the column removal interval of the column. Δt_{off} is suggested by many researchers and the DoD to be set as the one-tenth of the structure's period associated with the response mode of the element removal. For the case of the five frames the response modes associated with the removal of the corner columns are shown in Fig. 3, as well as their periods.

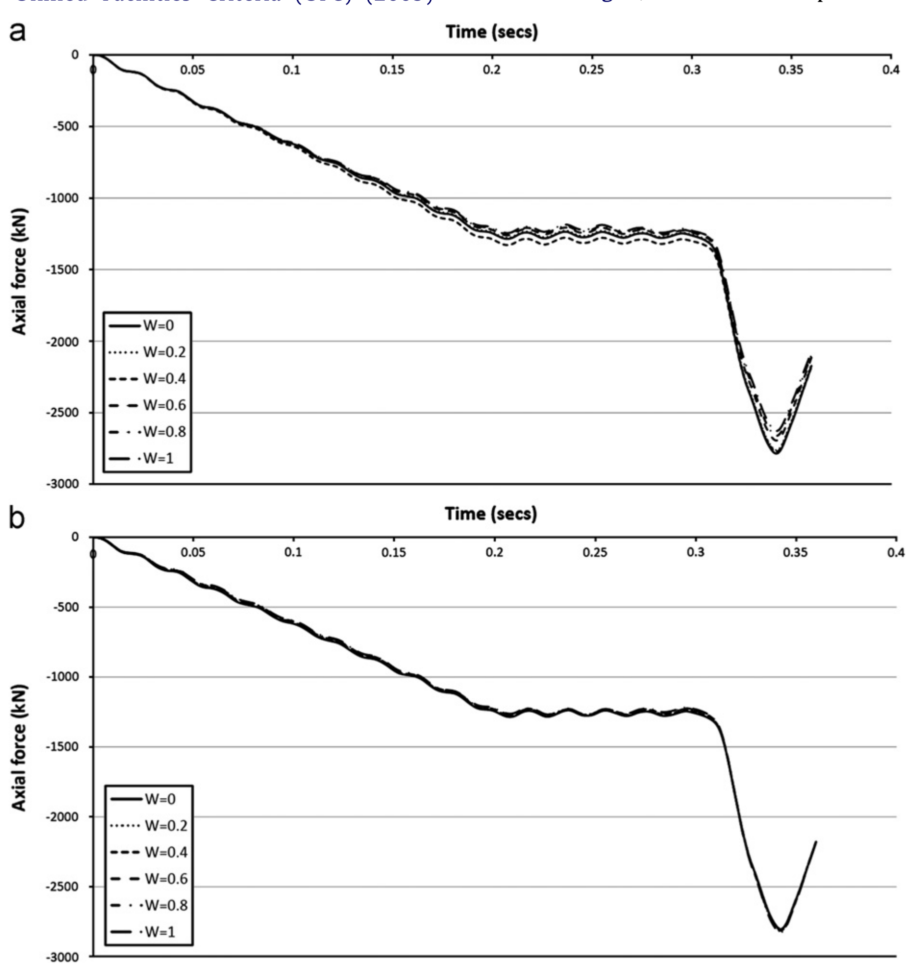

Fig. 5. Frame 1. Axial force of adjacent column. (a) Windward corner column loss, (b) Leeward corner column loss.

4.3. Computational algorithm of the analysis

For the purposes of the dynamic analyses, among the plethora of the available methods, the β-Newmark method was selected. Numerical damping was not included in the analysis in order not to affect the results of the instantaneous maximum response of forces and displacements which occur shortly after the complete removal of the column and are the main interest of the analysis performed (further explanation on the method can be found in Gerasimidis and Baniotopoulos, 2011).

Gerasimidis and Baniotopoulos have shown in Gerasimidis and Baniotopoulos (2011) that regardless the computational algorithm used for the dynamic analyses, the time step size of the algorithm must be sufficiently small in order to capture the phenomenon. An appropriate value of time step size has found to be $\Delta t_{off}/300$, where Δt_{off} is the time interval of the column removal.

4.4. Load combinations—wind loads participation

According to Unified Facilities Criteria (UFC) (2009), the load combinations associated with the event of progressive collapse include three different load cases: dead, live and snow. Additionally, a very small lateral load is applied on every floor of the structure as the incorporation of imperfections of the structure. For the purposes of this paper, the integration of wind loading is presented with six different levels of application. The load combinations applied on the three frames are shown in Table 6. Intentionally, all the loadings except from wind have been kept as fixed values leaving the wind loading as the only variable. This way, better conclusions on the effect of wind loading to the results will be accomplished.

5. Analysis results

In order to monitor the response of the structure two different response measure have been selected to be presented. The first regards the vertical displacement that occurs at the top joint of the column that has been removed of the structure; this way the effect of the column removal is highlighted. Secondly, the axial force of the adjacent column to the column removed is presented as a function of time.

The results for the displacement of the top node of the removed column are presented in Figs. 4, 6, 8, 10 and 12 of frames 1, 2, 3, 4 and 5 respectively. For example for frame 3, Fig. 8(b) shows the displacement of the node as a function of time for all the six different load combinations. A first comment that can be extracted by the graphs is that the effect of the column removal for all the three frames as well as for all the six different load combination is dramatic and majorly affects the displacement. However, it is remarkable that for frame 1 the displacement is slightly affected and furthermore as the wind loads are increased, the displacements decrease, relieving the frame. It should be noted for the windward column removal, the maximum deflection for zero wind loading, i.e. $w=0$, is 7.7 cm while for $w=1$ the same deflection is 7.1 cm. Although the difference is not significant, the fact that the deflection is decreasing shows the effect of wind loading to the response. For

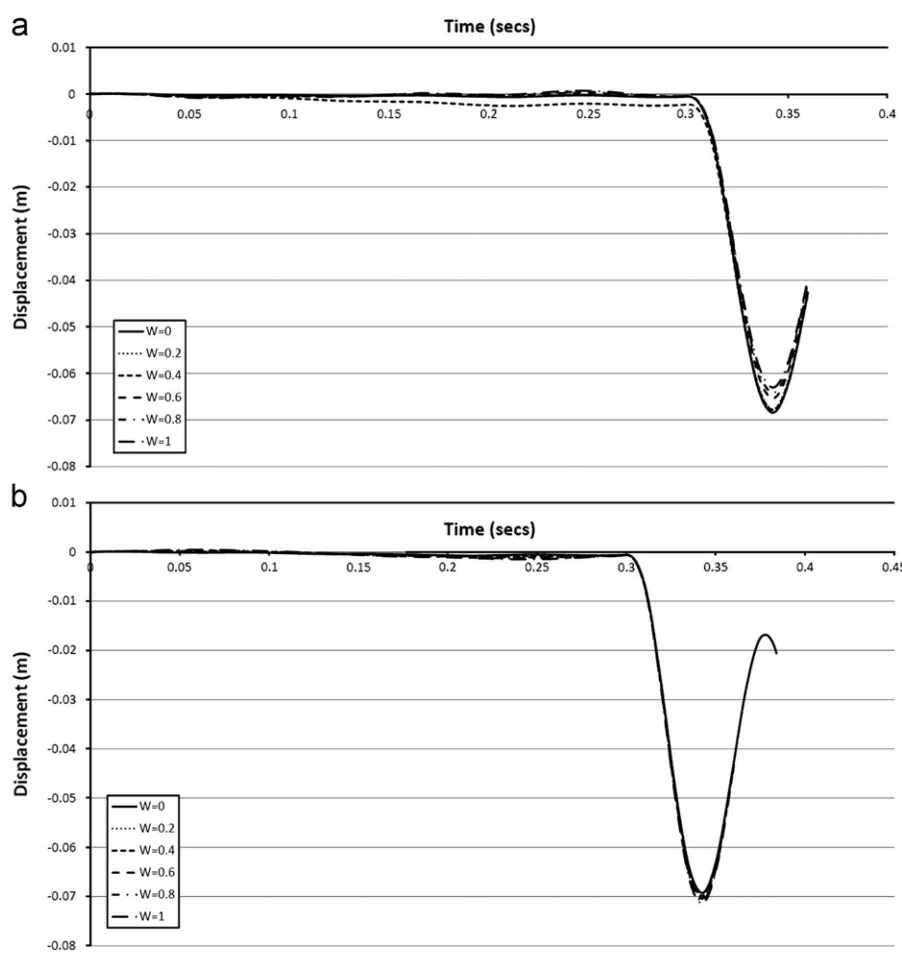

Fig. 6. Frame 2. Displacement at top node of removed column. (a) Windward corner column loss, (b) Leeward corner column loss.

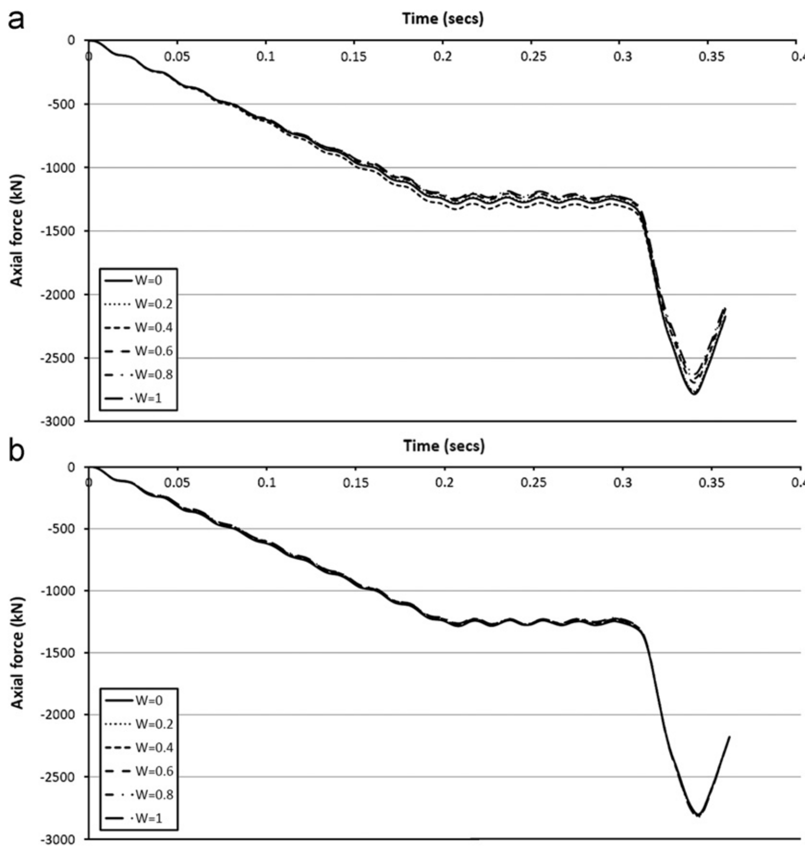

Fig. 7. Frame 2. Axial force of adjacent column. (a) Windward corner column loss, (b) Leeward corner column loss.

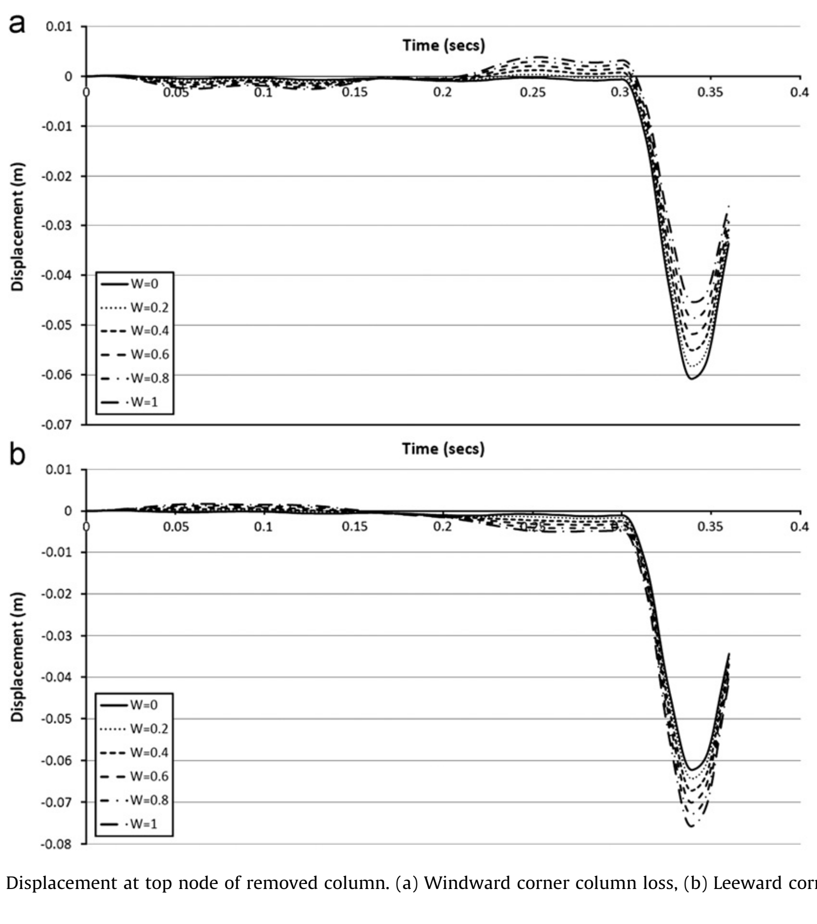

Fig. 8. Frame 3. Displacement at top node of removed column. (a) Windward corner column loss, (b) Leeward corner column loss.

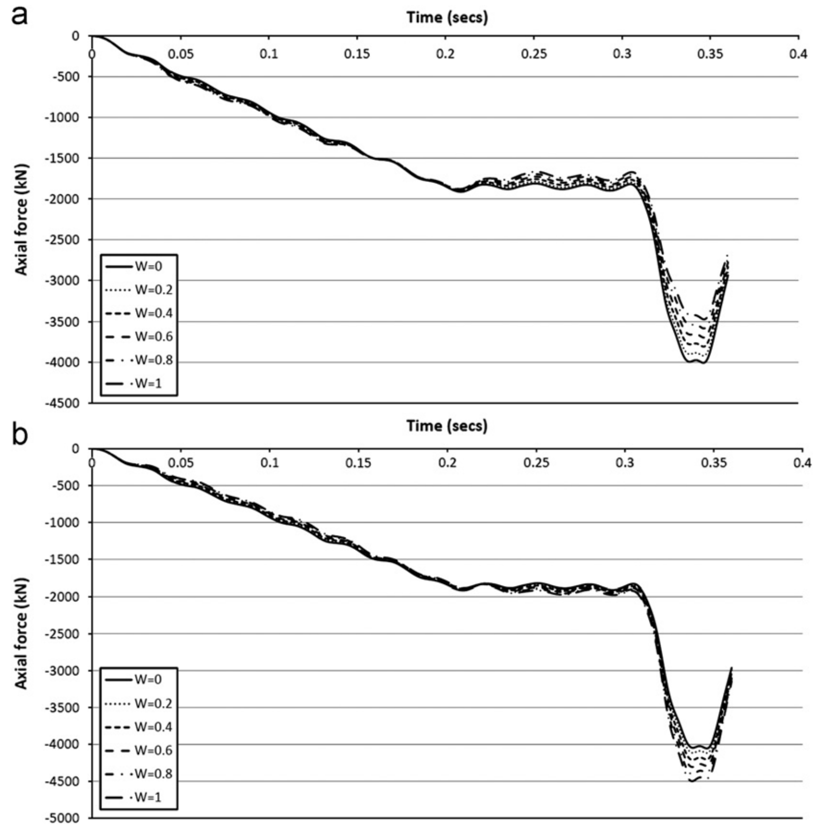

Fig. 9. Frame 3. Axial force of adjacent column. (a) Windward corner column loss, (b) Leeward corner column loss.

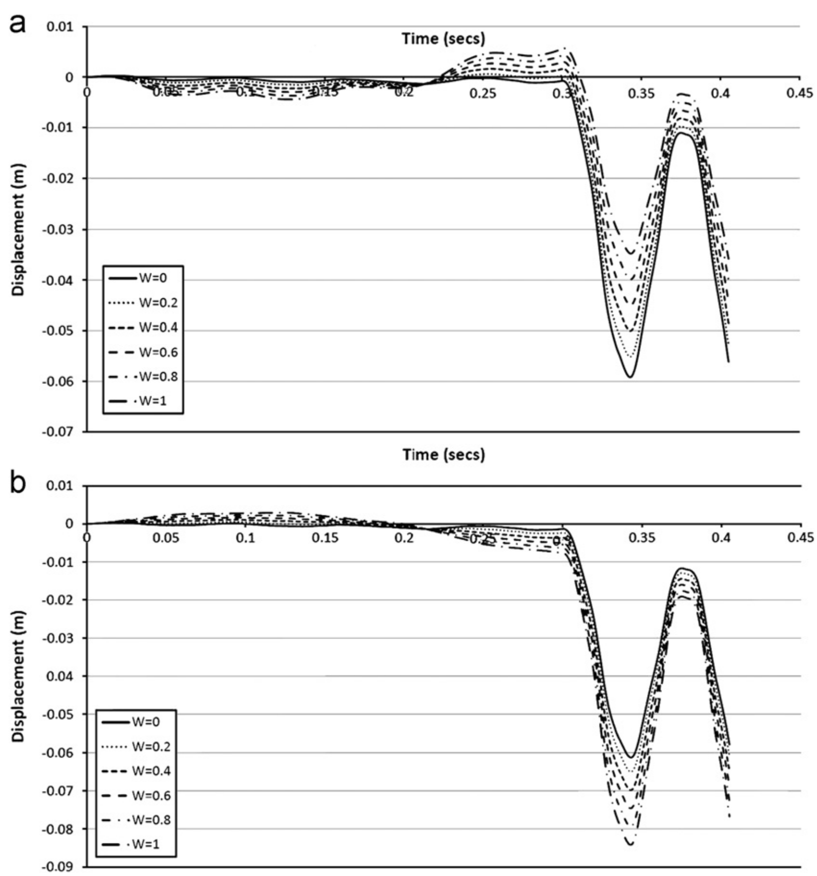

Fig. 10. Frame 4. Displacement at the top node of removed column. (a) Windward corner column loss, (b) Leeward corner column loss.

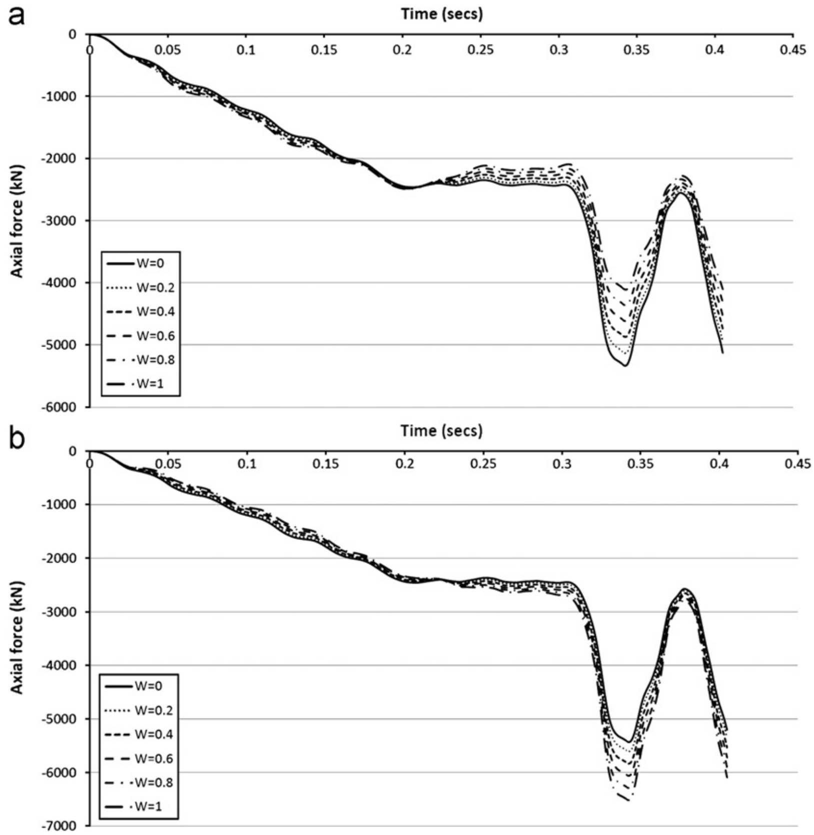

Fig. 11. Frame 4. Axial force of adjacent column. (a) Windward corner column loss, (b) Leeward corner column loss.

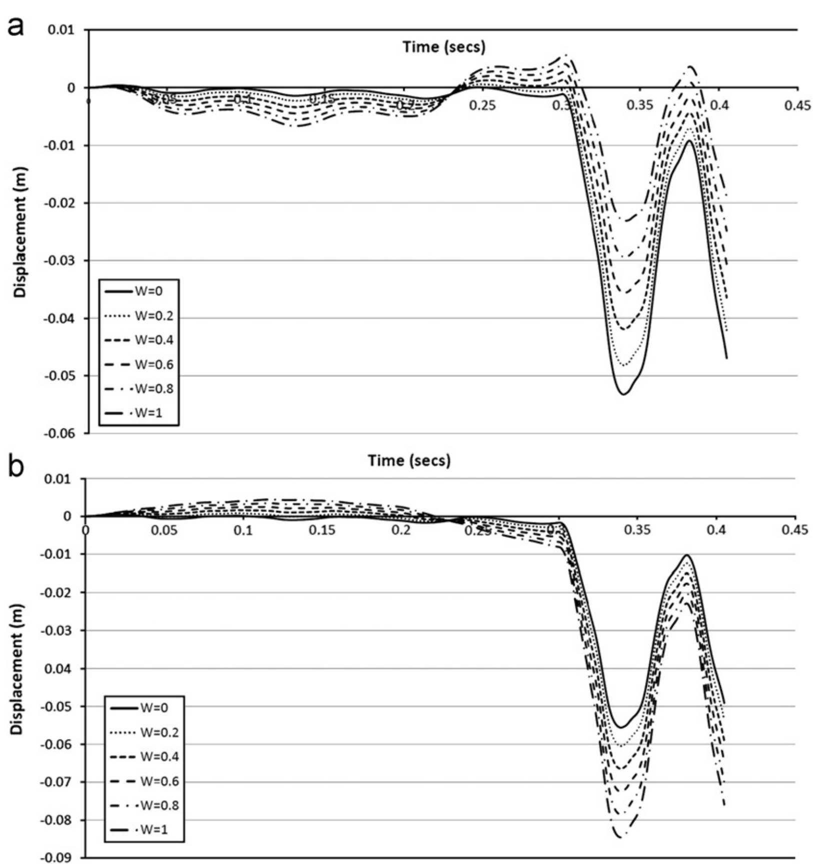

Fig. 12. Frame 5. Displacement at the top node of removed column. (a) Windward corner column loss, (b) Leeward corner column loss.

the leeward column removal, the effect of the wind loads is negligible and that is why the graph of the deflections [Fig. 4(b)] is almost identical for all the loading cases. The explanation of this finding can be that the lateral wind loading for a three floor building does not play an important role for the response of a steel moment frame, for the event of a column removal.

Particularly, for frame 2 and more demonstratively for frame 3, the maximum deflections are significantly affected by wind loading. For frame 3 and for the case of leeward column removal the deflection of the top node changes from 6.23 cm for the case of $w=0$, to 7.6 cm for the case of $w=1$; this is an increase of approx. 22%. The same increase for frame 2 is 3% while for frame 1 it is negligible. From these values, a first rather expected conclusion can be drawn that the higher the steel frame, the bigger is the impact of the wind loads in the response of the structure.

The axial loads of the adjacent column to the removed one follow the same pattern as the deflections; the results are depicted in Figs. 5, 7, 9, 11 and 13 for frames 1, 2, 3, 4 and 5, respectively. For frame 1, the increase of the axial load between the case $w=0$ and the case $w=1$ is negligible, for frame 2 the increase is 1%, while for frame 3 it reaches 11%. It can be easily assumed that taller buildings would tend to present higher relevant increases in their responses.

As height increases the effect of the wind loading becomes much more important. It must be noticed that the increase in the deflection between the case of no wind and the case of full wind respective of the node above the leeward column (when removed) is almost 0% for frame 1, 4% for frame 2, 20% for frame 3, 28% for frame 4 and almost 35% for frame 5. The same pattern follow the results of the axial forces of the adjacent column to the removed one, showing the higher the building the more crucial it is to include wind in the disproportionate collapse analysis. Even the inclusion of 20% of the design wind for the case of frame five increases the deflection of the node almost 10% (Fig. 13).

It must be also noticed that the effect of the wind loading is smaller for the case of the leeward column removal but at the same time the values of the displacements are higher for that case. This becomes clearer for the high frames; for example for frame 5 the effect of the wind loading is almost identical for the case of windward and leeward column removals but the values of the deflection are 5.2 cm for the windward column removal and 8.3 cm for the case of leeward removal. This occurs due to the fact that the columns resisting larger axial loads cause more deformation in the structure when they are suddenly removed.

6. Concluding remarks

The effect of wind loading in the response of steel moment frames for the event of a column loss was analyzed in this paper. The absence of wind in load combinations recommended in guidelines regarding disproportionate collapse can be either a conservative approach or not, depending on the structural system.

This paper applied time history analysis at the problem of disproportionate collapse analysis for steel moment resisting frames. The alternate load path of the DoD was loosely followed for the load combinations of the structure, incorporating wind loading which is absent by relevant documents describing disproportionate collapse analysis. The time history analysis was used in order to simulate the

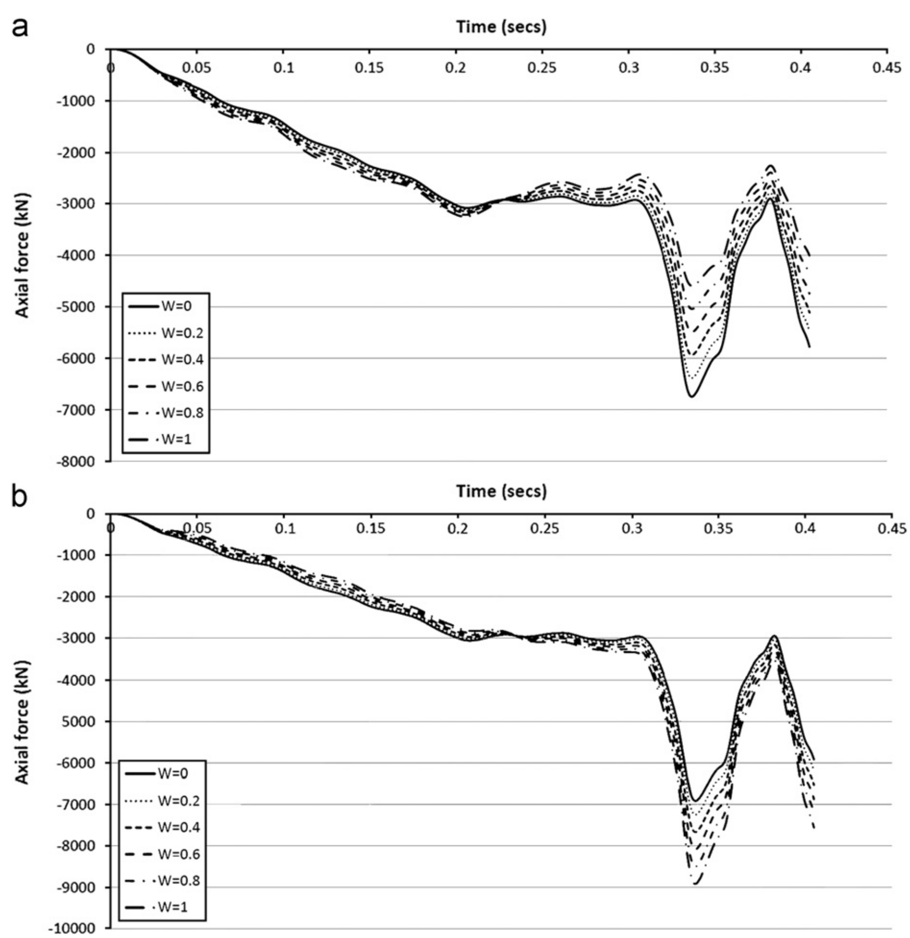

Fig. 13. Frame 5. Axial force of adjacent column. (a) Windward corner column loss, (b) Leeward corner column loss.

concept of column removal, a commonly accepted modeling scenario for inducing damage to a structural system.

The parametric analyses included five reference frames varying in height and six different load combinations varying the wind loading. From the results it was shown that the wind loading does not significantly affect the response of the structure for low-rise buildings such as frames 1 and 2. Therefore the absence of wind loading does not have any impact for the evaluation of disproportionate collapse analysis measures. However, for taller buildings such as frames 3,4 and 5, it was found that wind loads can affect the structure's response as high as 35% and must therefore be included in the analyses.

The analyses have shown that a clear distinction between low-rise and medium-to-high-rise steel frames is needed when analyzing for disproportionate collapse. The distinction is needed in order to capture the effect of wind loading on structures which can be very significant for high-rise steel buildings.

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