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CHALLENGE JOURNAL OF STRUCTURAL MECHANICS 4 (3) (2018) 82–88

Challenge Journal of Structural Mechanics

Research Article

Re-examination of steel frame office buildings in preventing collapse when subject to intense fires

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ABSTRACT

The purpose of the paper is to investigate the extent to which present-day design of steel framed buildings is susceptible to total collapse when subjected to extreme fire events. We select a 50 storey structure in which 2 and 4 adjacent storeys located at different above-ground heights are, in separate scenarios engulfed in raging fires. A total of 8 scenarios are analyzed, employing Newtonian mechanics and realistic energy dissipating properties of H-shaped columns and normal concrete floor slabs possessing secondary (shrinkage and temperature) reinforcement alone. While the present Canadian building code is the basis for our column designs, other standards provide very similar specifications. Although fire proofing is required in virtually all high rise building construction, we are excluding such materials in order to simplify the analyses, but clearly do not advocate its omission – quite the opposite in fact. As well, attributes such as floor beams, partitions and furnishings of every description, all of which would in practice participate in absorbing the kinetic energy of a crush-down upper block are excluded. Despite such a vast array of conservative assumptions, it is shown that partial collapse may occur during crush-down, however, in no case will total collapse be the consequence. These results should provide some comfort to code writers that their requirements should indeed prevent the most catastrophic of failures due to fires.

ARTICLE INFO

Article history:
Received 7 December 2017
Revised 20 March 2018
Accepted 2 June 2018

Keywords:
Hi-rise steel frame buildings
Fire events
Energy dissipation elements
H-shaped columns
Secondary reinforced concrete floors
Newtonian mechanic principles
Collapse states

1. Introduction

Prior to the horrendous events of 9/11 that resulted in thousands of deaths and the total destruction of three high rise steel framed buildings in the World Trade Center complex in New York City, the issue of fires being the cause of collapse of such structures was ignored for good reason. An excellent earlier history of fire safety inferred that existing fire protection of the supporting structure together with life safety systems were indeed sufficient. Even the authors of the FEMA 403 report (2002) concluded that no significant changes were warranted in building codes and design practice. The circumstances of that day in 2001 were deemed to be an extraordinary combination of extreme events that could not have been anticipated.

Surprisingly, perhaps, is the huge number of fire events that occur in high rise buildings every year that are generally unreported because of the rarity of associated collapse events. For example, John Hall Jr. has published reports for the National Fire Protection Association in the U.S. (2001, 2013) that provide statistics showing, on average, 10,000 significant fires having occurred annually in buildings 7 storeys or more in periods 1985-1998 and 2007-2011 respectively, with occupancy classifications being: apartments, hotels and motels, hospitals and care facilities, and offices. Although the latter 5 year period reveals somewhat fewer fire events than the earlier study, that data, together with the events of 9/11 have inspired the structural engineering profession to devote greater efforts to reduce the risks of fire, civilian deaths and injuries and property damage. In that regard,

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ISSN: 2149-8024 / DOI: https://doi.org/10.20528/cjsmec.2018.03.001

a very comprehensive, but difficult to verify computer model known as the Fire Dynamics Simulator (FDS) which purportedly "gave good agreement with the fire spread as observed at the windows" of WTC 1 and 2 (McAllister et al., 2013), but when later improvements were made to FDS, attempts to confirm its validation for reconstructing the magnitude of the WTC fires was described as weak (Quintiere and Williams, 2014).

Engineers and scientists who have a rudimentary knowledge of structural design and thermal properties of materials should not be expected to accept on faith the results of computer analyses, especially when they are at odds with historical data, as were the circumstances involving the demise of the WTC towers. In an earlier paper on the collapse of WTC7 (Korol et al.,2015), we undertook a fire analysis that accounted for fire loadings and temperature-time plots to ascertain if that structure could have succumbed by fire alone. However, in this article, our purpose is to undertake a simplified analysis that accepts worst case conditions on a generic steel framed 50 storey office building weakened by fire loadings, employing conventional principles of engineering design and Newton's laws, and to which gravity loads are applied. In this regard, a series of 8 scenarios will be examined involving adjacent 2 and 4 storey fires at levels ranging in 10 storey increments from top to bottom - the objective being to assess the extent to which partial or total collapse would be the result.

2. Description of the Postulated Structure

We begin by assuming a rectangular-shaped 50 storey hi-rise office building, having equal storey heights of 4m for which non-core areas (our focus) occupy an area of 3,920 m². For purposes of simplicity, we assume a regular grid consisting or 80 equal sized columns in a given storey, a segment of which is noted in Fig. 1, with tributary areas of 49 m² and sizes commensurate with gravity loading based on load and resistance factors consistent with the most recent design code for steel structures used in Canada (CSA, 2014). It's assumed that W-shapes are employed for the columns with size changes occurring in three storey segments. A commonly-used series in such applications is W360 which has a wide range of weights and dimensions compatible as columns in compression load transfer, with upper column footprints contained within the cross section profiles consistent with those below. This form of column stacking provides for simplified and cost efficient detailing for which splice and filler plates alone are needed for full moment and shear transfer compatibility.

It will be assumed that the steel is construction grade, having a nominal yield stress value, F_y of 345 MPa, which will be reduced to a value defined as F_y^* to account for compressive loading of slender elements in compression (flanges and web) in accordance with the Canadian design code mentioned earlier. As well, there is the matter of selecting column sizes to minimize fabrication costs (i.e. choosing sections that are continuous for 3 storeys). For Scenario 1, we assume a constant size for columns occupying the uppermost storeys for which fires will rage, i.e. the 40th and 41st levels. A design computation for sizing these columns must therefore be based on prescribed loadings on various floors, for which we chose the 40th for our example that also must take into account column slenderness. A detailed computation for this case is presented in Appendix A.

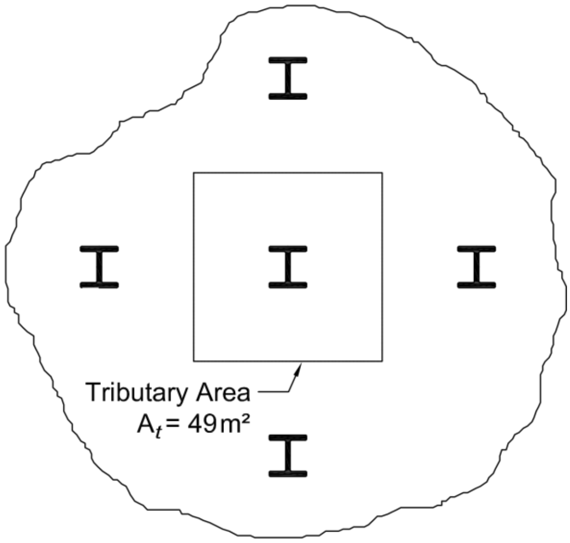

Fig. 1. Non-core column arrangement.

Meanwhile, the floors at all levels and the factored gravity loading appropriate at that level are presumed to consist of normal strength concrete having an average thickness of 102 mm (4") and containing wire mesh, ostensibly cast onto ribbed steel deck, meant to resist shrinkage and temperature cracks. Although the floors will obviously require floor beams and girders to support floor loadings, our analysis will ignore the twisting and bending energy absorbed during a potential global collapse event. In this way, we are assured that actual collapse states will be less severe than what we predict from our analysis.

3. Assumptions Associated with Collapse Initiation

In the design of any structure, a reasonable margin of safety is needed that accounts both for unanticipated overload conditions, and a possible strength deficiency of the material below its nominal value. In general, a high overload value, generally above 3 is the norm when lateral loads from winds and earthquake are present.

However, for our case, we will assume that the core of the structure will contain the elevator shaft, stairwells and mechanical rooms with a reinforced concrete perimeter wall which, together with robust steel columns provides the structural system needed to resist lateral loading. In the non-core area therefore, we assume that resistance to gravity loading alone controls the design of the columns based on limit states. The safety factor then represents a ratio of the factored gravity loads divided by realistic load values existing under the situation of a fire event. The resistance factor for the columns, therefore, is based on both the nominal steel strength and an effective length factor for equally loaded columns at given floor levels. Even discounting the bending restraint provided by floor beams and girders, a factor of safety of approximately 2 is provided based on industry standards and noted by Wikipedia which states "Design factors for specific applications are often mandated by law, policy, or industry standards. Buildings commonly use a factor of safety of 2.0 for each structural member. The value for buildings is relatively low because the loads are well understood and most structures are redundant." General details are noted under "Wikipedia" in our References section. A further breakdown of loads and resistance information is presented in Appendix A for the case described as Scenario 1. As noted, fires raging throughout a given storey will reduce column strength equally to all columns, but for computational purposes, the equivalent of 39 of the 80 columns fail totally, thus offering no further resistance. Consequently, we make the assumption that 41 remain with full load capacity and which will offer resistance at the instant that collapse of the upper block is initiated. This value represents an average, since column sizes are limited and can only provide approximate factors of safety for given storeys. Overall, however, this number is on the conservative side in virtually all design situations and as such is the value which we shall ascribe to our design example.

Such a value will be presumed to mean that the equivalent of slightly more than 50% of the columns in a storey that is fully ablaze are able to resist the loads above it until progressive collapse begins. When one more column succumbs, those remaining will be deemed to no longer withstand the loading above, and will fail with post-bucking resistance offered by the 41 unscathed columns, and act to slow down the motion of the block of storeys above. However, their post-buckling resistance will be shown to be insufficient in preventing crush-down onto the floor below.

Fig. 2 shows a profile view of such a structure depicting four scenarios in one sketch. Scenario 1 is indicative of fires in storeys 41 and 40 while the rest of the building is free from fires of any kind. Scenario 2, illustrates a case where the fires are only occurring in storeys 31 and 30, etc. to Scenario 4, in which fires exist only in storeys 11 and 10.

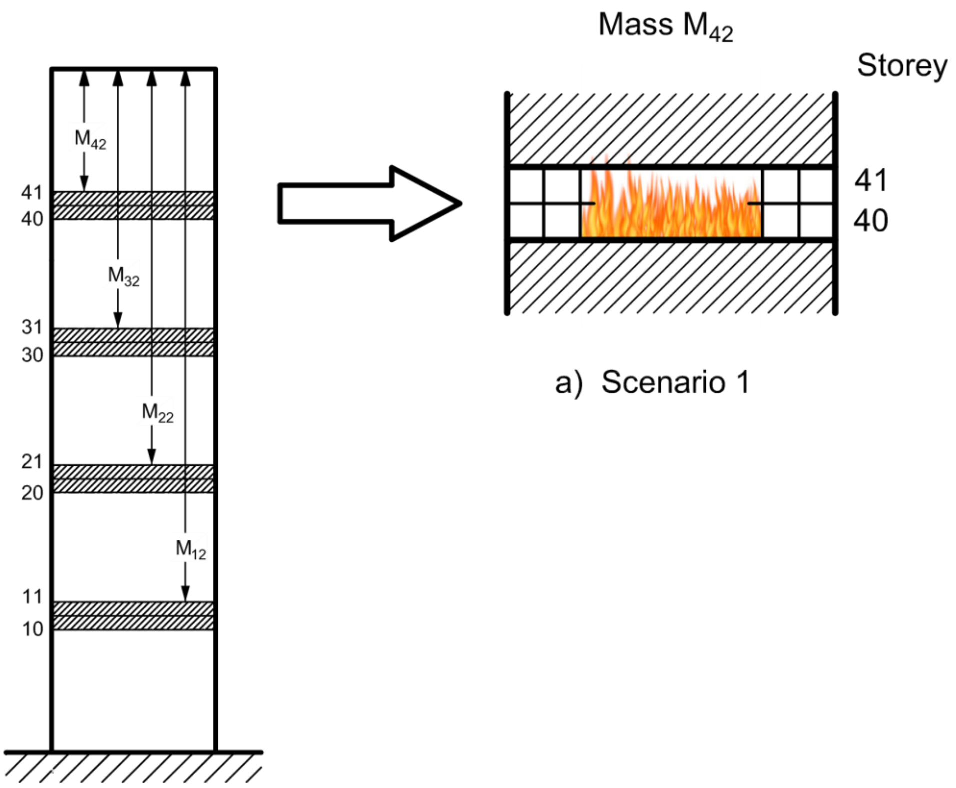

Fig. 2. Scenarios from 1 to 4 (raging fires on each of 2 storeys).

4. Dissipative Elements Resisting Collapse

In considering the columns which are deemed to be the major contributors to resisting storey-to-storey gravity loads, we assume therefore that 41 columns within a given storey are totally capable of providing post-buckling resistance to axial loads as opposed to the other 39 which are assumed to have been completely compromised by the fires raging in the storeys noted (i.e. 41 and 40 as noted in Fig. 2(a) for Scenario 1). An issue to be resolved is what value of axial resistance is appropriate when a column fails, but retains post buckling strength resistance. Bazant and Zhou (2002) chose to employ only bending resistance together with rotation capacity as being the measure of such energy absorption. As noted in an earlier paper (Korol and Sivakumaran, 2014), the vast majority of columns such as those employed in office buildings offer much greater energy dissipative values under axial loads than values computed employing standard plastic hinges under lateral loads only. Based on a set of experiments undertaken at McMaster University, axial energy dissipation in the post-bucking range needs to be included. We do so indirectly by employing a factor defined as α, which is used to multiply the Plastic Bending energy part alone to account for both types of resistance. Indeed, our experiments on the equivalent of pin-end-ended columns showed values of α to be about 3.5 times the value obtained when that same member is subjected to a lateral load causing minor axis plastic bending alone.

We assume for storeys experiencing raging fires that the concrete in their floor areas offer no resistance through pulverization because of having experienced intense heat. However, other floors that have not been exposed to the fires are presumed to be resistant to break-up during the assorted impacts that will occur as storeys crush down onto lower levels. In this regard, our earlier work on the energy required to pulverize concrete slabs that contains shrinkage steel suggest that penetration loadings from beams and girders crashing onto them, will be resisted in accordance with the degree of pulverization, i.e. the extent to which floors are broken up into both very small and large sized particles through penetration loadings. An earlier paper (Sivakumaran et al., 2014) reported on our experimental findings involving concrete slabs with shrinkage reinforcement only, and arrived at an estimate of 4900 J/kg as an average value for the break-up under penetration loadings. We will use this figure for normal strength concrete having a density of 2400 kg/m^3, together with an average depth of floor slab of 101.4 mm (4”).

5. Fire Condition Scenarios

5.1. Scenario 1

Being very conservative in estimating resistances offered in storeys experiencing raging fires, we ignore any dissipative energy the presumed concrete floors would normally offer. Consequently, the only resistance assumed to contribute towards energy dissipation by steel members will involve 41 columns i.e. these being *almost* sufficient to support the loading above the storeys that are on fire, and noted as mass M_{42} in Fig. 2 for Scenario 1. Since the initial energy state has zero velocity the following equation therefore applies to this case:

$$½(M_{42})\,(v_{41}^{f})^2 = M_{42}\,g\,h - 41 \cdot 0.9\pi\, Z_y\, F_y^{*}\, \alpha\,, \tag{1}$$

in which v_{41}^{f} is the final velocity of the upper block at impact with the 41st floor and Z_y is the plastic section modulus for bending about the section’s minor axis (CISC handbook, 2016). Note that we have assumed that plastic hinges form only at the mid-height of storeys, and do *not* form at floor levels, again under-estimating column absorptive energy capacity in the interest of being conservative. The 0.9π factor is an estimate of the angle rotated at mid- height as a given column is crushed, before encountering a floor’s crushed debris. Inserting the values noted in row 1 of Table 1, together with h = 4m gives the result of v_{41}^{f} = 7.71 m/sec.

Table 1. Column and total mass data for pertinent storeys.

Storey i	W-Shape	Z_y (mm^3)	F_y^* (MPa)	$\sum i^{51}(m_i + m_{cols}) \cdot 10^3\ \text{kg} = M_{i+1}$
41	W360 x 162	1520	299.9	19483
40	“	“	“	21553
39	W360 x 179	1680	302.2	23529
38	“	“	“	25505
37	“	“	“	27480
31	W360 x 262	2680	305.2	39072
30	W360 x 287	2960	306.2	41082
29	“	“	“	43091
28	“	“	“	45096
27	W360 x 314	3240	306.3	47120
21	W360 x 382	4030	308.0	58750
20	“	“	“	60790
19	“	“	“	62790
18	W360 x 421	4490	310.4	64883
17	“	“	“	66936
16	“	“	“	68985
15	W369 x 463	4980	310.8	70989
11	W360 x 509	5550	311.6	78742
10	“	“	“	80823
9	W360 x 551	6050	312.0	82918
8	“	“	“	85072
7	“	“	“	87106
6	W360 x 592	6570	312.2	89214
5	“	“	“	91321
4	“	“	“	93428

Continuing with the analysis in the 40th storey, we employ the conservation of momentum principle to ascertain the initial velocity of the block crushing down into the 40th storey and which gives a value of v_{40}^{i} equal to 10/11 times 7.71 to give the initial velocity of 7.01 m/sec. in the 40th storey. The equation of motion then becomes:

$$½(M_{41})\left(v_{40}^{f}\right)^{2} = ½(M_{41})\,(7.01)^{2} + (M_{41})\,g\,h - 41 \cdot 0.9\pi\,Z_{y}\,F_{y}^{*}\,\alpha\,. \quad (2)$$

Employing the values noted in the second row of Table 1 results in a value of v_{40}^{f} of 10.51 m/sec, followed by a reduced value of v_{39}^{i} of 9.63 m/sec. We are now dealing with a storey that has not experienced fires during this scenario and hence we need to account for all 80 columns and, if they are insufficient to stop the motion, pulverization of the 39th floor comes into play. The governing equations for such a case is:

$$(v_{39}^{f})^{2} = (v_{39}^{i})^{2} + 2gh - \frac{2\left[80\cdot 0.9\pi\,Z_{y}\,F_{y}^{*}\,\alpha - X\,(4.674\cdot 10^{9})\right]}{M_{40}}\,. \quad (3)$$

The numerator of the last term in Eq. (3) is the energy absorbed by concrete floor pulverization. Total floor destruction is indicated by $X = 1$, in which case the final velocity within that storey > 0 with motion continuing into the storey below, whereas a negative value of v^{f} would indicate that the motion stops with only part of the floor area energy dissipation needed by that floor's breakup. In our case, setting $v_{39}^{f} = 0$, and solving for X results in a value of 0.34 means that 34% of that floor is pulverized. We conclude, therefore, that the motion stops at the 39th floor. This result is given in the first row of Table 2.

5.2. Scenarios 2, 3 and 4

Similar calculations were performed for the other three scenarios that involved 2 storey fires. The pertinent data for Scenario 2 involved storeys 31, 30 and 29 in Table 1, while row 2 of Table 2 noted that motion stopped at the floor of storey 29, this time with 70% of the floor broken up. In the case of Scenario 3, it will be noted in Table 2 that crush-down extended an extra storey to level 18, with 38% pulverization, while Scenario 4 resulted in the most severe damage state with 4 storeys crushed and 77% of floor 8 suffering breakup. It is evident therefore that raging fires occurring at low rather than high levels in a building will result in a partial collapse that involves the greatest amount of damage to the structure.

5.3. Scenario 5

It is of interest to determine the extent to which a greater degree of structural damage occurs when 4 storeys are engulfed in fires. Such might be the case for circumstances when a building is struck by a jumbo-jet aircraft, for example, as we witnessed in New York City some years ago. In this particular scenario, fires are presumed to rage in storeys 41, 40, 39 and 38, and is shown in Fig. 3. For this Scenario, we will have equations similar to those noted for Scenario 1, but which require us to employ the data and do calculations for two additional levels involving stories 38 and 37. Employing Newton's laws of conservation of energy and the linear momentum principle as before, and with data obtained from Table 1, we determine that crush-down will destroy 5 storeys and result in 64% of the floor in storey 37 having been compromised before the motion stops. This means that about twice as much of the floor below the fire is subject to breakup as compared with Scenario 1 that involved two storey fires.

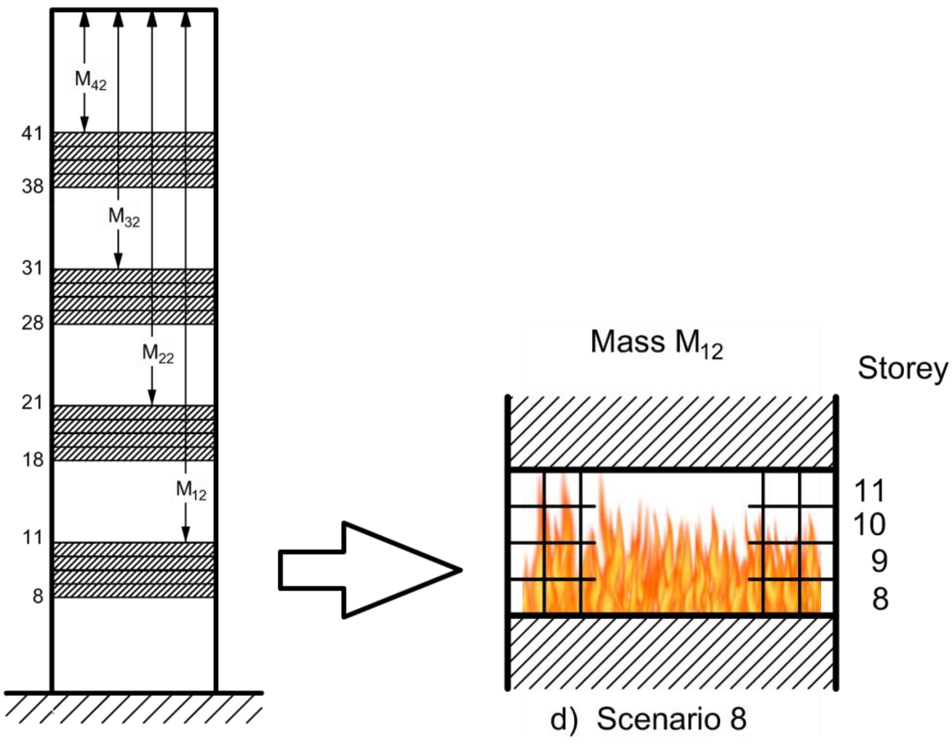

Fig. 3. Scenarios from 5 to 8 (raging fires on each of 4 storeys).

Table 2. Results of storey fires and computed states of collapse (based on steel yield stress = 345 MPa and S.F. = 2).

Scenario	Fires in storeys	Storeys crushed	Motion stopped in floor	Pulverization when motion stops
1	41,40	41, 40, 39	39	34%
2	31,30	31, 30, 29	29	70%
3	21,20	21,20,19, 18	18	38%
4	11,10	11, 10, 9, 8	8	77%
5	41, 40, 39, 38	41, 40 39, 38, 37	37	64%
6	31, 30, 29, 28	31, 30, 29, 28, 27, 26	26	46%
7	21, 20, 19, 18	21, 20, 19, 18, 17, 16, 15	15	49%
8	11, 10, 9, 8	11, 10, 9, 8, 7, 6, 5,4	4	63%

5.4. Scenarios 6, 7 and 8

These last three scenarios involve 4 storey fires at progressively lower levels akin to scenarios 2, 3 and 4. As observed from the last three rows of Table 2, an additional one or two storeys crush down occurs when compared to their scenario counterparts, 2, 3 and 4. Fig. 3 shows the storeys that are pertinent, i.e. 31, 30, 29 and 28 for Scenario 6, and so on, with our highlighting the last one as "d) Scenario 8". It represents the worst case in terms of damage to the structure since there are four additional storeys, 7, 6, 5 and 4 that crush down with the latter floor's area being pulverized by 63% before the motion stops.

6. Conclusions

As we noted in the Introduction, even the authors of the FEMA report pertaining to the collapse of the twin towers, purportedly due to fires primarily, did not advance a need for major changes to the applicable building codes at that time (FEMA, 2002). However, others, such as Bazant and Zhou (2002), Bazant and Le (2011) and NIST (2005, 2011) indicate otherwise. Our objective in this article has been to employ the present day building code that is applicable to high rise buildings and arrive at column sizes that meet the requirements of the CSA Standard (CAN/CSA S16-14) for non-core areas. These areas are less robust than are core areas, and hence represent a greater potential risk of collapse due to gravity driven loadings when extreme fire loads are present than would the overall structure experience.

Our focus has been to utilize principles that are easily understood, i.e. Newton's laws and employ assumptions commensurate with energy absorption elements that include plastic hinge buckling at mid height storeys that are subject to progressive collapse, and to concrete floor slabs that will breakup when struck by falling steel floor beams and girders. In the interest of being conservative, we have excluded any twisting and bending of these floor support members, the fasteners that yield and/or fracture, the arrays of non-structural partitions, desks, bookcases, filing cabinets etc. that are present in occupied office spaces.

Despite our neglecting so many such crush resisting components, we found that for cases where fires raged on two adjacent floors (4 scenarios including top, bottom and intermediate cases) that the one giving rise to a maximum amount of structural damage was Scenario 4. Fires in storeys 10 and 11 resulted in collapse of 2 storeys below, i.e. 9 and 8. For storey 8, the motion of the upper block stopped when 77% of a floor broke up.

When four adjacent floors experience extremely hot fires, crush-down failures do cause more than two storeys below to fail. In the case of Scenario 8 that involved fires on floors 8 to 11 inclusive, storeys 7 down to 4 suffered collapse, with the motion stopping with 63% of the 4th floor subject to breakup.

Since complete collapse in crush-down for even the most severe case of fires on four adjacent floors conceived in our study did not happen, we have to conclude that fire events of any reasonable description will not cause a steel framed building to collapse – partial collapse of some storeys? Yes, unfortunately, but overall collapse? No.

In that regard, code writers might wish to caution owners of high rise buildings that occupants ought to be warned about the possibility, slight as it might be, of some floors immediately below those experiencing *raging* fires to collapse, but not to panic. Storeys distant from those so engulfed would likely provide a temporary safe haven. Of course, smoke inhalation is another issue that such standards might need to embrace. Clearly, other means of protection which were beyond our study such as proper exits, sprinkler systems, alarms etc. are vitally important in saving lives.

Appendix A. Design Parameters Associated with Scenario 1

The following subsections provide the basis for the design of the columns in storeys 40 and 41.

a) For design purposes we must select what are known as specified values for L and D which for office buildings the values prescribed are taken as 2.4 kPa and 3.6 kPa respectively for floors. Employing load factors of 1.5 for L and 1.25 for D that include roof loadings assumed to be equivalent to that for a typical floor, and, estimating the self weight of columns above the 40th (10 levels), we get a factored load of 80 [8.1 x 49 x 11 x 10^3 + 105 x 4 x 10 x 9.81] = 353 x 10^6 N. Meanwhile the factored resistance for a single column of size W360x162 is computed to be 5560 kN for an unsupported length of 4m, the computation of which is given in Appendix 2. It follows, then, that for 80 columns prior to the fires the factored resistance is 5560 x 80 x 10^3 = 444.8 x 10^6 N, or about 26% higher than the factored load. (Had we selected a lighter section, it would have required filler plates to accommodate depth dimension discrepancy with the 360W179 section selected immediately below). The top two rows of Table 1 indicate the W-shape selected and the Z_y and F_y^* associated with the storeys that relate to the fire events pertaining to Scenario 1.

b) The computation of mass M_1 (block above storey 41) arising from *expected* live and dead loads during a fire, or in general, normal occupancy, involves prescribing a value lower than that proposed for the design. As such, we select 0.5 L, for the former, with dead loading equal to that specified, i.e. D in storeys 42 to the roof inclusive, i.e a total of 10 levels. We then calculate a value of load applied that when converted to mass units, works out to be 19,180 x 10^3 kg. Accounting for the self weight of the columns for 9 levels, gives an adjusted total tally of 19,483 x 10^3 kg (i.e. an average of 105 kg/m) and noted in the top row of Table 1. The 40th storey will, of course be subject to an additional level of loading for which the tally becomes 21,553 x 10^3 kg, noted in the second row. As pointed out previously, an adequate factor of safety (of about 2), results in our selecting an H-shape designated as 360W162 for these two levels, with a computed resistance in mass units of (5560 x 80 x 10^3)/9.81 = 45,340 x 10^3 kg, a value which is 2.1 times that of the expected applied loading. This size is reasonable for gravity forces only, since the core areas are deemed to provide much of the lateral resistance to withstand anticipated maximum wind or earthquake loads. However, it should be noted that expansion of structural members due to temperature effects which can result in additional stresses due to restraining forces from floor systems will have unspecified consequences in reducing the factor of safety to a minor extent, perhaps lowering it to just under 2.

Appendix B. Factored Resistance of Columns (Storey 40 & 41)

While F_y is presumed to be 345 MPa, a reduction factor is required to account a column's effective length and radius of gyration values. The formula that is applicable by the Canadian code is based on the parameter $\lambda = KL/r\ (F_y/\pi^2 E)^{1/2}$ in which KL = 4000 mm, $r_{\min} = r_y$ = 95 mm, and E = 200,000 MPa. For a W360x162 section, λ = 0.557. The empirical factor, which the code adopted is computed as $[1+0.557^{2.68}]^{-0.7463}$ = 0.8685, that when multiplied by F_y gives the value of F_y^*, or 299.6 MPa, as noted in Table 1.

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