An Integrated Adaptive Environment for Fire and Explosion Analysis of Steel Frames - Part II: Verification and Application

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ABSTRACT

The companion paper presents a new adaptive method for integrated fire and explosion analysis of steel frames. This paper verifies the new developments, implemented within the nonlinear analysis program ADAPTIC, particularly against the results of experiments on steel members and frames subjected to fire. The benefits of adaptive nonlinear analysis are then highlighted by means of an example which demonstrates its computational superiority over conventional nonlinear analysis. Finally, parametric studies on column and frame configurations subject to the successive actions of explosion and fire are undertaken using the developed integrated analysis tool. The results of these preliminary studies indicate that the damage induced by moderate explosion loading can significantly influence the fire resistance of steel structures. Collectively, the capabilities and efficiency of the developed environment emphasise the important role that it can play in further studies aimed at generating design guidance.

KEYWORDS

Steel frames; Fire and explosion analysis; Adaptive nonlinear analysis.

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1. INTRODUCTION

A new nonlinear analysis method is presented in the companion paper^[1] for steel frames subject to fire and explosion loading, where emphasis is placed on adaptivity of the nonlinear analysis method and the integration of fire and explosion loading within the same analysis. In the latter context, the developed method can be applied to investigate scenarios of explosion followed by fire, thus enabling the assessment of the influence of explosion damage on the fire resistance of steel members and frames.

The components of the developed adaptive method have been implemented in the nonlinear analysis program ADAPTIC^[2], which is used in this paper to undertake several verification and application studies. The verification part is mostly concerned with the elevated temperature models of steel, and hence the fire analysis capability within ADAPTIC; the capability for explosion analysis within ADAPTIC, particularly in relation to the accuracy of the rate-sensitive material models of steel, was verified in previous studies^[3,4]. Several experiments on steel members and frames subjected to fire are used for the verification process, where particular consideration is given to establishing the relative accuracy and efficiency of the bilinear and elliptic material models^[1] of steel at elevated temperature.

The paper then demonstrates the superiority of the adaptive method over conventional nonlinear analysis of steel frames subject to fire and explosion, both in terms of modelling effort and computational demand. Finally, parametric studies on column and frame configurations, subject to the successive actions of explosion and fire, are undertaken using the integrated analysis capability within ADAPTIC. The results of these preliminary studies indicate that even moderate explosion loading can significantly influence the fire resistance of steel structures.

2. VERIFICATION OF FIRE ANALYSIS CAPABILITY

This Section is concerned with the verification of the fire analysis capability within ADAPTIC^[2], particularly in relation to the developed models of steel at elevated temperature presented in the

companion paper^[1]. Comparisons against the results of several experiments are presented hereafter, where conventional (non-adaptive) analysis is employed based on a pre-defined mesh of elasto-plastic cubic elements. The application of the proposed adaptive method for fire and explosion analysis, with particular reference to accuracy and efficiency in comparison with the conventional method, is discussed in Section 3. All CPU times reported hereafter are for ADAPTIC v2.8.0 running on a Silicon Graphics R4000 workstation with 112Mb of memory.

2.1. Beam Subject to Uniform Temperature

Rubert and Schaumann^[5] carried out several elevated temperature tests on simply supported beams loaded at midspan, as illustrated in Figure 1, where the tests were aimed at studying the temperature-dependent behaviour of steel beams under simultaneous external and thermal loading. The beams, having an IPE 80 cross-section (DIN 1025-1), were subjected to a constant midspan load and then heated uniformly along their entire length. The load utilisation factor, which is the ratio of the applied load to the ultimate load carrying capacity at room temperature, was varied in the tests.

The various tested beams are simulated with ADAPTIC using 10 elasto-plastic cubic elements, utilising both the bilinear and elliptic material models^[1]. The variations of Young's modulus (E), the proportional limit (σ_r), the yield strength (σ_y), and the strain-hardening parameter (μ) with temperature are given in Tables 1.a and 1.b for the bilinear and elliptic model, respectively. The parameters of the elliptic model are taken from Rubert and Schaumann^[5], whereas the parameters of the bilinear model are chosen such that the resulting stress-strain relationship at elevated temperatures provides a good fit to that represented by the elliptic model. A constant coefficient of thermal expansion of $14 \times 10^{-6} \text{ } ^\circ\text{C}^{-1}$ is adopted with both material models.

Young's modulus (E)		Yield strength (σ_y)		Strain-hardening (μ)	
Temperature (°C)	Value (GPa)	Temperature (°C)	Value (MPa)	Temperature (°C)	Value
20	210.0	20	399.0	20	0.00
100	210.0	300	399.0	300	0.00
700	84.0	700	59.9	400	0.032
1100	0.0	1000	0.0	900	0.00

Table 1.a. Material properties for the bilinear model.

Young's modulus (E)		Proportional limit (σ_r)		Yield strength (σ_y)	
Temperature (°C)	Value (GPa)	Temperature (°C)	Value (MPa)	Temperature (°C)	Value (MPa)
20	210.0	20	399.0	20	399.0
100	210.0	167	399.0	400	399.0
300	168.0	300	239.4	700	79.8
500	126.0	500	199.5	800	53.2
700	84.0	650	39.9	900	26.6
1100	0.0	1000	0.0	1000	0.0

Table 1.b. Material properties for the elliptic model

The variation of the midspan deflection with temperature is shown in Figure 2 for load utilisation factors of 0.20, 0.50, 0.70 and 0.85, as obtained from the tests and from ADAPTIC using the bilinear and elliptic models. The predicted analytical results agree well with the test results, particularly for a utilisation factor of 0.5, for which both the bilinear and elliptical modes agree closely with the test results over the full duration of elevated temperatures. It is also observed, for the four load utilisation factors, that the elliptic model provides a better prediction of the detailed response prior to the initiation of large displacements in comparison with the bilinear model.

However, the bilinear model still provides a realistic prediction of the overall behaviour, particularly in the range of large displacements achieved around the limiting temperature. In this range, it is evident that the neglect of strain-hardening effects by the elliptic model seems to affect adversely its prediction, and thus a more sophisticated elliptic model should include strain-hardening of steel particularly at the lower temperatures. With a reasonable prediction of the overall response, the bilinear model also achieves around 50% reduction in computational demand over the elliptic model; the CPU demand for the four cases of load utilisation factor is (5.5s, 4.4s, 3.9s, 2.5s) using the bilinear model and (10.5s, 6.6s, 6.8s, 6.6s) using the elliptic model. Accordingly, it is proposed that the bilinear model is better suited for computationally intensive parametric studies.

2.2. Beam with Temperature Gradient

Cooke^[6] carried out an experiment on an unrestrained I-section steel beam, shown in Figure 3, where only the bottom flange was heated, causing a temperature gradient over the cross-section. Since the beam is unloaded and unrestrained, this experiment was principally aimed at investigating the effect of thermal bowing at elevated temperatures. Thermo-couples were used at 12 internal stations over the beam length to measure temperatures, with 5 thermo-couples employed over the section depth at each station. The temperature distribution was observed as uniform over the width of the section but nonlinear over the section depth, as illustrated in Figure 3. The beam was heated over a period of 50 minutes, where maximum temperatures of 1058 °C, 450 °C and 294 °C were achieved in the bottom flange, web mid-height and top flange, respectively.

The beam is modelled with ADAPTIC using 13 elasto-plastic cubic elements, such that an element node is located at every station of measured temperatures, with the temperatures at the two supports assumed identical to those of the nearest internal station. The nonlinear temperature variation over the cross-section is modelled using quadratic functions, constructed using the centroidal and flange temperatures, which are different for the various internal stations. An equivalent linear distribution of temperature (i.e. constant temperature gradient over the cross-section) is also considered, where the equivalent centroidal temperature (t_c) and temperature gradient ($\Delta t/\Delta y$) are obtained from the

equivalence of axial force and bending moment under strain restraints, assuming a constant coefficient of thermal expansion:

$$t_c = \frac{1}{A} \int_A t \, dA, \quad \Delta t / \Delta y = \frac{1}{I_y} \int_A t \, y \, dA \quad (1)$$

Both the bilinear and elliptic models are considered for modelling this test, where the corresponding properties representing a best fit of the Eurocode 3 model^[7] are presented in tables 2.a and 2.b, respectively. The variation of thermal strain with temperature is assumed to follow the Eurocode 3 recommendations for the elliptic model, including the effect of phase transformation, whereas a constant coefficient of thermal expansion of $15 \times 10^{-6} \, ^\circ\text{C}^{-1}$ is assumed for the bilinear model.

Young's modulus (E)		Yield strength (σ_y)		Strain-hardening (μ)	
Temperature ($^\circ\text{C}$)	Value (GPa)	Temperature ($^\circ\text{C}$)	Value (MPa)	Temperature ($^\circ\text{C}$)	Value
20	210.0	20	275.0	20	0.00
300	210.0	300	275.0	300	0.00
600	42.0	600	55.0	600	0.04
1200	0.0	1200	0.0	1200	0.00

Table 2.a. Material properties for the bilinear model.

Young's modulus (E)		Proportional limit (σ_r)		Yield strength (σ_y)	
Temperature ($^\circ\text{C}$)	Value (GPa)	Temperature ($^\circ\text{C}$)	Value (MPa)	Temperature ($^\circ\text{C}$)	Value (MPa)
20	210.0	20	275.0	20	275.0
100	210.0	100	275.0	400	275.0
500	126.0	400	115.5	700	63.2
700	27.3	500	99.0	800	30.2
800	18.9	700	20.6	900	16.5
1200	0.0	1200	0.0	1200	0.0

Table 2.b. Material properties for the elliptic model

A comparison between the experimental results and the results of ADAPTIC using the bilinear and elliptic models, assuming a quadratic temperature variation over the cross-section, is presented in Figure 4, where a very good agreement is observed over the full duration of elevated temperatures. Cooke's^[6] analytical prediction is based on simplified expressions which are derived assuming equivalent linear temperature distribution over the cross-section, and which neglect the effects of elevated temperature on the elastic modulus, the initiation of plasticity and the influence of phase transformation on the thermal strains, a constant coefficient of thermal expansion of $14 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ being assumed. Cooke attributed the depression phenomenon observed at around 30 minutes in the test results to phase transformation. He also suggested that the subsequent discrepancy between his analytical and experimental results is not effected by assuming an elastic material response, but is principally due to the additional assumption of a constant elastic modulus which does not vary with temperature.

In order to investigate the influence of the various assumptions made by Cooke, the beam is re-analysed with ADAPTIC starting with the full representation using the elliptic model. This full model is then modified to incorporate Cooke's assumptions one at a time, where the comparative results are provided in Figure 5. The influence of phase transformation is first investigated through using an average coefficient of thermal expansion of $15 \times 10^{-6} \text{ }^{\circ}\text{C}^{-1}$ instead of the Eurocode 3 recommendation for thermal strains, where a negligible effect is observed in comparison with the full model. Next, the assumption of elastic behaviour, allowing for a varying elastic modulus with temperature, is considered, where the final displacement is overestimated by 10%. The assumption of elastic behaviour, neglecting the variation of elastic modulus with temperature, is then investigated, where the final displacement is overestimated by 38%. Finally, the influence of approximating the nonlinear temperature distribution over the cross-section by a linear function (i.e. constant temperature gradient) is considered, allowing for the effects of plasticity and variation of the elastic modulus with temperature, in which case the final displacement is overestimated by 40%.

It is worth noting that if the final analysis based on a linear temperature distribution is undertaken assuming an elastic behaviour with a constant elastic modulus, an identical result is obtained to that of the quadratic temperature distribution. From the results of Figure 5, this indicates that the

assumption of an elastic behaviour with a constant elastic modulus has overall a smaller, yet opposite, effect for the linear temperature distribution in comparison with the quadratic distribution.

2.3. Scaled Steel Frame Tests

A series of tests was performed by Rubert and Schaumann^[5,8] on 1/6-1/4 scale plane steel frames at elevated temperatures. Three frame configurations, shown in Figure 6, were considered: configuration EHR had two orthogonal members rigidly connected with pinned ends; configuration EGR represented rigidly connected portal frames with pinned bases; and configuration ZSR consisted of two-bay continuous portal frames with pinned bases. All members of the EHR and EGR series were heated uniformly using electrical devices, whereas only the left bay was fully heated for the ZSR series with the remaining two members left at ambient temperatures. The dimensions and loading used for each individual test, as well as the measured yield strength at ambient temperature (σ_y^0), are given in Table 3.

The above tests are simulated with ADAPTIC using 10 cubic elements for each structural member, and employing mainly the elliptic material model. The elastic Young's modulus of steel at ambient temperature is taken as 210 GPa, and the coefficient of thermal expansion assumes a constant value of $14 \times 10^{-6} \text{ }^\circ\text{C}^{-1}$. The adopted reduction factors for Young's modulus, the proportional limit (σ_r) and the yield strength (σ_y) at elevated temperature are as proposed by Rubert and Schaumann^[5], following the piecewise linear curves given in Table 1.b, but scaled for σ_r and σ_y to reflect the different yield strength at ambient temperature for each series as given in Table 3.

The critical temperatures at which large displacements are initiated are listed in Table 3 as predicted by ADAPTIC as well as reported by Rubert and Schaumann^[8]. Results from ADAPTIC show good agreement with the test results for all three configurations, with a discrepancy of less than 10% of the test results. In most of the cases, the predicted results are on the safe side, as illustrated in Figure 7.

System	L (cm)	h (cm)	σ_y^0 (kN/cm ²)	F1 (kN)	F2 (kN)	Critical Temperature	
						Test (°C)	ADAPTIC (°C)
EHR1	119	117	39.5	56	14	600	620
EHR2	124	117	39.5	84	21	530	542
EHR3	124	117	38.2	112	28	475	452
EHR4	125	150	38.9	20	5	562	529
EHR5	125	150	38.9	24	6	460	471
EHR6	125	150	38.9	27	6.7	523	417
EGR1b	122	117	38.2	65	2.5	533	489
EGR1c	122	117	38.2	65	2.5	515	489
EGR2	122	117	38.5	40	1.6	612	599
EGR3	122	117	38.5	77	3.0	388	387
EGR4	122	117	41.2	77	3.0	424	417
EGR5	122	117	41.2	88	3.4	335	293
EGR6	122	117	41.2	88	3.4	350	293
EGR7	122	117	32.0	68.5	2.6	454	426
EGR8	122	117	38.5	77	3.0	464	423
EGR9	122	117	41.2	88	3.4	38.7 min	N/A
EGR10	120	113.5	43.2	82	3.1	609	593
ZSR1	120	118	35.5	74.0	2.85	547	514
ZSR2	120	118	38.0	84.5	3.25	479	464
ZSR3	120	118	43.2	68.5	2.64	574	579

Table 3. Parameters and results for the frame tests

Rubert and Schaumann^[8] also reported the variation of displacements with temperature for three of the tests, namely EHR3, EGR1 and ZSR1. These results are compared against the predictions of ADAPTIC in Figures 8.a-c, respectively, where both the bilinear and elliptic material models are employed. The comparison against the results of test ZSR1 is particularly good, with the analytical predictions, based on both the bilinear and elliptic models, matching the test results closely up to the

critical temperature. The discrepancies observed in the comparison against the results of the other two tests are attributed to the uncertainty in material properties at elevated temperatures. It is worth noting that the predictions of the bilinear model compare favourably with those of the elliptic model for all three tests, particularly in terms of critical temperature. The reasonable accuracy of the bilinear model is further combined with greater efficiency; CPU times of (9.5s, 13.5s, 31.2s) are required by the bilinear model for tests EHR3, EGR1 and ZSR1, respectively, whereas the corresponding CPU times required by the elliptic model are (18.5s, 21.7s, 35.8s).

3. VERIFICATION OF ADAPTIVE ANALYSIS

A three-storey frame is considered under the action of explosion and fire loading in the first floor side compartment, as illustrated in Figure 9. The explosion loading (p), which precedes fire loading, is represented by a uniformly distributed overpressure applied in the outward direction to the compartment beams and columns, varying according to a triangular pulse with a peak value of ($p_{\text{peak}}=125 \text{ kN/m}$), a rise time of 120 msec and a total duration of 150 msec. Elevated temperatures due to fire loading are governed by a single temperature parameter (T), which is monotonically increased from the ambient temperature of 20°C . Assuming that the side column is fire protected and that the lower beam is not influenced by fire, the extreme fibre temperatures in the compartment members are given in Table 4 in terms of the temperature parameter (T). Note that all temperatures start from 20°C , with the temperature distribution taken to be uniform over the member length and linear over the cross-section depth. The bilinear model is adopted for modelling elevated temperature and strain-rate effects, with the elevated temperature parameters as given in Section 2.1 and the rate-sensitive parameters ($S = 31.19\text{MPa}$) and ($\dot{\epsilon}_* = 4.65 \times 10^{-3} \text{ sec}^{-1}$) as recommended by Izzuddin and Fang^[4].

Side column		Top beam		Internal column	Lower beam
Internal flange	External flange	Lower flange	Upper flange	Both flanges	Both flanges
$0.5T+10$	$0.25T+15$	T	$0.75T+5$	T	20

Table 4. Extreme fibre temperatures in fire compartment members (°C).

The frame is considered under two loading scenarios: 1) fire loading only, and 2) explosion followed by fire. For both cases, nonlinear analysis is undertaken using the conventional method and the proposed adaptive method. With the conventional approach, a pre-defined mesh of 10 elasto-plastic cubic elements per member is employed. With the adaptive approach, analysis is started with 1 elastic element per member, each of which could potentially subdivide into 10 elasto-plastic elements when and where necessary. The variation of the structural response with the temperature parameter (T), for both loading scenarios, is shown in Figures 10.a-c, depicting the horizontal displacements at the mid-height of the compartment side and internal columns, and the vertical displacement of the top floor left edge, respectively. These results demonstrate excellent agreement between the predictions of adaptive and conventional analysis for both loading scenarios.

This example illustrates the considerable influence of explosion on the fire resistance of steel frames, even when the extent of structural damage due to explosion is relatively small. For the considered overpressures, damage due to explosion is concentrated in the side column, which sustains an initial permanent deflection of approximately 125 mm at mid-height, as depicted in Figures 10.a and 11.b. For both loading scenarios, elevated temperatures initiate buckling in the internal column at T of approximately 475 °C. However, the explosion/fire scenario is associated with a much reduced overall fire resistance of ($T \approx 642$ °C) in comparison with that of the fire only scenario ($T \approx 894$ °C), representing a reduction of 28%. This reduction is mainly attributed to deterioration in the vertical resistance of the side column due to explosion damage, leading to redistribution of vertical loading to the internal column and an earlier overall failure of the system. The deflected shapes for the two

loading scenarios are shown in Figures 11.a-c, where the thicker lines represent locations where elasto-plastic cubic elements have been adaptively introduced according to the criteria given in the companion paper^[1].

In addition to modelling benefits, the proposed adaptive method provides a superior computational performance to that of the conventional method for both loading scenarios. Consideration of the CPU time demand over the analysis domain, as depicted in Figure 12, demonstrates the considerable computational savings of adaptive analysis, which range between 85% and 95%, depending on the desired termination point of the nonlinear analysis. It is also worth noting that even greater computational savings should be expected with adaptive analysis for larger structures subjected to localised explosion and/or fire loading, in which the relative need for expensive elasto-plastic elements would be greatly reduced.

4. APPLICATION STUDIES

The previous Section illustrates that explosion loading can have a considerable effect on the fire resistance of steel structures. The proposed integrated adaptive environment can be applied to quantify the complex interaction between explosion and fire effects in determining the overall resistance of steel framed structures. Two series of parametric studies are undertaken hereafter to illustrate the applicability of the proposed integrated environment, and to draw some preliminary conclusions regarding the mechanisms by which explosion loading can influence fire resistance. The first series considers a particular configuration of an isolated column where the column length, axial loading, explosion loading and temperature distribution are varied parametrically. The second series considers the steel frame of the previous Section, where the explosion loading and the initial gravity loading are the main parameters.

4.1. Partially-Restrained Steel Column

A partially-restrained column with a UC254×254×89 standard section, as shown in Figure 13, is considered under the actions of initial vertical loading (F), followed by an overpressure (p) due to

explosion loading and subsequent elevated temperatures due to fire loading. The material response is represented by the bilinear model, including the strain-rate and elevated temperature effects. At ambient temperature, Young's modulus is taken as 205 GPa, and the yield stress is taken as 275 MPa, with the parameters of rate-sensitivity taken as^[4] ($S = 31.19 \text{ MPa}$) and ($\dot{\epsilon}_* = 4.65 \times 10^{-3} \text{ sec}^{-1}$). The material properties at elevated temperatures are given in Table 5, considering the temperature entity (T) to be incremental over ambient temperature.

	E (GPa)			σ_y (MPa)			μ			ϵ_{th}		
T(°C)	100	700	1200	400	800	1200	400	600	1200	750	860	1200
Value	205	27	0	275	30	0	0	0.02	0	0.011	0.011	0.0178

Table 5. Material properties of steel column at elevated temperatures

The compressive axial force (F) is applied at the top of the column as an initial load. The explosion loading, represented by a uniformly distributed overpressure (p), is applied on the left side of the column causing bending about its major axis. Furthermore, an initial imperfection of (L/1000) is assumed at the column mid-height in the direction of the applied explosion loading. The variation of the normalised overpressure with time is shown in Figure 14, which is a typical overpressure time-history for a gas-air mixture in a vented confined explosion^[9].

Two column lengths (L=3, 5m) and two levels of axial loading ($F=0.3F_p$, $0.68F_p$) are considered in this study, where ($F_p=3.1 \times 10^3 \text{ kN}$) is the plastic axial capacity. The blast resistance of the resulting four combinations is given in Table 6, expressed in terms of the maximum peak overpressure ($p_{peak}=p_{max}$) that can be applied. These results show that the effect of reducing the column length and the level of axial loading improves the blast resistance of the column, as expected.

L=3m		L=5m	
F=0.3F _p	F=0.68F _p	F=0.3F _p	F=0.68F _p
849.8	520.7	290.5	146.8

Table 6. Blast resistance $p_{\max}$ (kN/m) of steel column

For each of the above four column configurations, three temperature distributions are considered over the cross-section i) uniform temperature (T), ii) linear distribution with positive gradient ($dT/dy=1.1T$ °C/m), and iii) linear distribution with negative gradient ($dT/dy=-1.1T$ °C/m). In the second case, the temperature on the left flange ($1.144T$) is higher than that of the right flange ($0.856T$), where T is the centroidal temperature. For all cases, it is assumed that the temperature distribution along the column length is uniform.

The fire resistance in the absence of explosion loading for the resulting twelve cases is given by Table 7 in terms of the critical centroidal temperature at failure. These results indicate that failure under the compressive axial force is associated with plastic rather than elastic buckling, since the critical temperatures for the two column lengths are similar given specific values for the remaining parameters. It is also observed that the linear temperature distribution is associated with a lower critical centroidal temperature than that of the uniform temperature distribution. This is expected, since a temperature gradient leads to non-uniform material response characteristics over the cross-section, which introduces a considerable effective eccentricity of the axial load, thus reducing the plastic buckling capacity of the column. Furthermore, the positive temperature gradient cases generally exhibit slightly lower critical temperatures than the cases of negative temperature gradient. This is attributed to the fact that initial imperfections, assumed to the right side of the column, increase the effective eccentricity of the axial load for the positive temperature gradient cases, given that the buckling failure mode for these cases is to the right side of the column.

Temperature distribution	L=3m		L=5m	
	F=0.3F _p	F=0.68F _p	F=0.3F _p	F=0.68F _p
Uniform	712.5	538.8	709.6	533.3
dT/dy=1.1T °C/m	651.0	499.9	634.9	495.5
dT/dy=-1.1T °C/m	653.3	506.3	642.7	504.1

Table 7. Critical centroidal temperature $T_{\max}$ (°C) for steel column under fire loading only

For each of the twelve combinations, the peak overpressure (p_{peak}) is varied over the range $[0, p_{\text{max}}]$ to establish the influence of explosion on the fire resistance of the column. The interaction between the normalised fire resistance ($T/T_{\max}$) and the normalised explosion loading ($p_{\text{peak}}/p_{\text{max}}$) is shown in Figures 15.a-d, where p_{max} and $T_{\max}$ are given in Tables 6 and 7, with each figure depicting the results for one of the four (L, F) cases subject to the three temperature distributions. The results indicate that the interaction between the fire resistance and explosion loading for the considered column configurations is not significant, since a considerable reduction in the fire resistance is only achieved for explosion loading close to the maximum blast resistance. For the short column ($L=3\text{m}$), a 10% reduction in the fire resistance is obtained for explosion loading in excess of 95% of the blast resistance. For the long column ($L=5\text{m}$), the same reduction in fire resistance is obtained for explosion loading in excess of 85% of the blast resistance, except for the more heavily loaded column with a positive temperature gradient ($F=0.68F_p$, $dT/dy=1.1T$ °C/m) where the corresponding value is 80% of the blast resistance. Interestingly, moderate explosion loading is shown to have a beneficial effect on the fire resistance of all the considered column configurations with a negative temperature gradient ($dT/dy=-1.1T$ °C/m), where an increase in the fire resistance of up to 5% is achieved. This is attributed to moderate permanent deflections following explosion loading, which have the effect of reducing the effective eccentricity of the axial load introduced by the non-uniform material response characteristics over the cross-section.

4.2. Three-Storey Steel Frame

The three-storey steel frame, considered in Section 3 for the verification of adaptive analysis (Figure 9), is investigated here under different intensities of explosion loading followed by fire loading. This study is carried out for two levels of initial vertical loading ($w=60, 75$ kN/m), with the normalised temperature and overpressure distributions as given in Section 3.

The blast resistance of the frame is established in terms of the peak overpressure ($p_{\text{peak}}=p_{\text{max}}$) which initiates overall structural failure. Similarly, the fire resistance, in the absence of explosion loading, is obtained in terms of the maximum temperature parameter ($T=T_{\text{max}}$) leading to overall failure. Values of (p_{max}) and (T_{max}) for the two levels of vertical loading are given in Table 8. With the response to fire loading in the absence of explosion discussed in Section 3, the response of the frame to the maximum explosion loading is depicted in Figures 16 and 17 for the case of vertical loading ($w=75$ kN/m).

$w=60$ kN/m		$w=75$ kN/m	
T_{max} (°C)	p_{max} (kN/m)	T_{max} (°C)	p_{max} (kN/m)
934	645	894	333

Table 8. Fire and blast resistance of steel frame (No fire/explosion interaction)

The interaction of fire resistance and explosion loading is established in a similar manner to the previous column example. The peak overpressure (p_{peak}) is varied over the range $[0, p_{\text{max}}]$, the extent of which depends on the vertical loading (w) as given in Table 8. For a given peak overpressure, the resistance to the subsequent fire loading is obtained in terms of the critical temperature (T) initiating overall failure. The resulting interaction between the normalised fire resistance (T/T_{max}) and the normalised explosion loading ($p_{\text{peak}}/p_{\text{max}}$) is depicted in Figure 18 for the two considered levels of vertical loading (w). Unlike the isolated column example, it is shown here that there is considerable interaction between explosion loading and the fire resistance of the steel frame. For the case ($w=75$

kN/m), a 40% reduction in the fire resistance is initiated for peak overpressures between 30% and 60% of the blast resistance ($p_{\max}$). Similarly, for the case ($w=60$ kN/m), a 30% reduction in the fire resistance is initiated for peak overpressures between 20% and 50% of the blast resistance ($p_{\max}$). In these respective ranges of peak overpressure, the side column is being subjected to increasing damage, leading to the redistribution of vertical loading to the internal column and the consequent reduction in overall fire resistance. Interestingly, the reduction in the fire resistance remains less than 50% until the explosion loading becomes severe, exceeding 92% and 98% of the overall blast resistance ($p_{\max}$) for ($w=60$ kN/m) and ($w=75$ kN/m), respectively. In the latter case, overall failure under the vertical loading ($w=75$ kN/m) is governed by the blast damage to the internal column; consequently, the extent of such damage is the main parameter influencing the residual overall fire resistance. In the former case ($w=60$ kN/m), however, overall failure under the reduced vertical loading ($w=60$ kN/m) requires considerable blast damage to the whole of the blast compartment, which is achieved only at very high overpressures (Table 8). This is reflected in the higher levels of explosion loading which are required to cause a further reduction in the overall fire resistance.

5. CONCLUSION

The companion paper^[1] presents a new adaptive environment for the integrated nonlinear analysis of steel frames subject to fire and explosion loading. This paper i) verifies the proposed environment, particularly in relation to its fire analysis capabilities, ii) demonstrates the accuracy and efficiency of adaptive nonlinear analysis for steel frames under fire and explosion, and iii) illustrates the applicability of the proposed method in parametric studies of components and structures subject to combined scenarios of explosion and fire.

Several verification examples of members and frames subject to elevated temperature show favourable agreement against the developed material models for steel. Whereas the elliptic model is shown to provide better comparisons than the bilinear model, particularly for small plastic strains, it is demonstrated that the simple bilinear model combines reasonable accuracy with greater efficiency.

The accuracy and efficiency of adaptive analysis is demonstrated on a steel frame subject to fire loading only as well as a combined scenario of explosion preceding fire loading. Comparisons show that the proposed adaptive method retains the accuracy of the conventional method while resulting in considerable modelling benefits and computational savings ranging between 85% and 95%. It is suggested that even more savings can be achieved with adaptive analysis for larger steel frames.

Finally, the application of the integrated adaptive environment is illustrated through parametric studies on an isolated column as well as a three-storey frame subject to explosion followed by fire. While it is observed that the interaction between explosion loading and fire resistance can be minor for the isolated column, it is shown that considerable interaction is present on the level of overall frame systems. For such systems, reductions of 40% in the fire resistance can be induced for explosion loading of around 40% of the blast resistance. Evidently, the proposed integrated environment will provide an essential tool for future parametric investigations, leading to design guidance on the interaction between explosion loading and the fire resistance of steel members and steel framed structures.

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Figure 1. Simply supported beam subject to elevated temperature^[5]

Figure 2. Response of simply supported beam to elevated temperature

Figure 3. Simply supported beam subject to temperature gradient^[6]

Figure 4. Thermal bowing of simply supported beam: comparison against test results

Figure 5. Thermal bowing of simply supported beam: significance of assumptions

Figure 6. Configurations of scaled steel frame tests^[8]

Figure 7. Critical temperatures for scaled steel frames

Figure 8.a. Comparison against test results: configuration EHR3

Figure 8.b. Comparison against test results: configuration EGR1

Figure 8.c. Comparison against test results: configuration ZSR1

Figure 9. Steel frame subject to explosion and fire loading

Figure 10.a. Horizontal displacement at mid-height of side column

Figure 10.b. Horizontal displacement at mid-height of internal column

Figure 10.c. Vertical displacement at left edge of top floor

Figure 11.a. Final deflected shape of steel frame under fire loading only

Figure 11.b. Deflected shape of steel frame after explosion loading

Figure 11.c. Final deflected shape of steel frame under explosion and fire loading

Figure 12. CPU time demand for seel frame analysis

Figure 13. Geometric configuration and loading for partially-restrained column

Figure 14. Variation of normalised overpressure with time

Figure 15.a. Influence of explosion on column fire resistance: $L=3$ m, $F=0.3F_p$

Figure 15.b. Influence of explosion on column fire resistance: $L=3$ m, $F=0.68F_p$

Figure 15.c. Influence of explosion on column fire resistance: $L=5$ m, $F=0.3F_p$

Figure 15.d. Influence of explosion on column fire resistance: $L=5$ m, $F=0.68F_p$

Figure 16. Response of steel frame to explosion loading: $p_{\text{peak}}=p_{\text{max}}$, $w=75$ kN/m

Figure 17. Final deflected shape of steel frame subject to explosion: $p_{\text{peak}}=p_{\text{max}}$, $w=75 \text{ kN/m}$

Figure 18. Influence of explosion loading on fire resistance of steel frame