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OPERATIONAL RESILIENCE AND PERFORMANCE-BASED FIRE DESIGN

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Abstract: Operational resilience is an increasingly important topic in today's discussions about building design, asset management, and business continuity. This is a trend that will not disappear as a changing environment and growing urbanization continue to increase the importance of designing for extreme events, whether they are malicious, accidental, or environmental in nature. The research presented herein will analyze how performance-based fire design can be used during the design of a building to enable operational resilience within it. By proactively and quantitatively determining the performance of a structural solution for a range of realistic design fires, the public can understand the implications of the fire for their tenant space and ultimately for the operation of their business. A methodology will be proposed for developing a range of criteria, in concert with the expected users of the building, to understand the required performance of the building. This can be quantified as immediate occupancy, short-term disruption, and long-term/permanent disruption. This is a novel approach to fire safety design that varies from what is typically seen in Canadian practice where a deterministic set of design fires seek to capture the worst case structural and life safety scenario. The overall aim of this approach is to lay out specific levels of performance for ranges of design fires so that the users of the building can appropriately prepare response plans for different fire events and remain operational following a fire. This methodology will be demonstrated through a case study of a realistic Canadian building design that has performance levels quantified for a realistic design fire to feed into a business continuity plan. The end result is a more sustainable building which enables operational resilience within it, and the approach can be extrapolated to all hazards affecting infrastructure.

1 Introduction to Resilience

In the context of the built environment, resilience can be defined as the ability for an operation to withstand *shocks* or *stresses* and return to normal operating capacity in a desired timeframe. A *stress* is a prolonged disturbance to the operation, such as labour shortage or reduced water supply, while a *shock* is a sudden and short disturbance such as an earthquake or a fire. It is important to note that resilience is a property of the *operation*, which in turn is enabled by the built environment in which it exists. The operation within a building may be commercial, residential, or institutional in nature, among others. When structures are designed, the engineer takes into account typical load types such as live load, wind load, and earthquake load. Increasingly, engineers are being asked to also account for loads due to hazards such as blast events, tsunamis/flooding, or even fire. When a hazard materializes the performance of that structure before, during, and after the event will have a direct influence on the resilience of that operation since the structural response will determine the extent of damage for that specific event and to what extent the structure can be occupied and/or repaired afterwards. The seismic community has previously quantified resilience for structural schemes by comparing functionality to time, as seen in Figure 1.

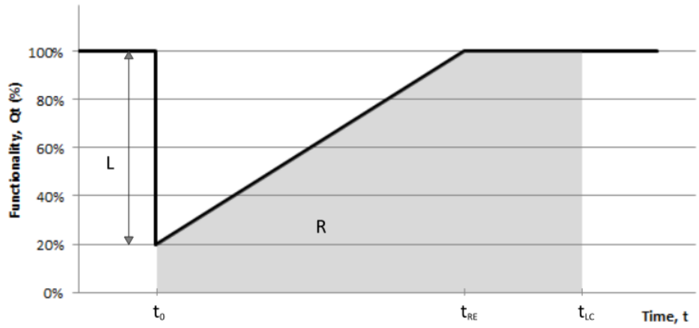

Figure 1: Simplified response plot for quantification of resilience, adapted from Cimellaro et al. (2010)

In the above plot, it can be seen following an event, the operation loses functionality equal to L. A recovery phase begins from the time of the event t_0 up until the time of full recovery t_{RE}. Resilience can be defined as the area under this functionality curve Q(t) as shown in equation [1] (Cimellaro et al., 2010), with more resilient operations recovering full functionality quicker. It is also measured up to a control time t_{LC} such that alternatives with different recovery times can be compared on the basis of resilience. A more detailed assessment of operational resilience will also consider ranges of functionality required at various times, including minimum operating capacity and minimum sustainable capacity (Hay, 2016).

[1] $R = \int_{t_0}^{t_{RE}} Q(t)dt$

This paper will propose a methodology for determining a series of structural performance criteria that can be used when assessing a structure, new or existing, at the fire limit state to quantify the performance of that structure for a range of fire scenarios. This range of performance across a series of realistic design fires can directly inform a business continuity plan for the broader operation in terms of responding to accidental fires. The emphasis throughout will be on steel and composite-steel structures as data and observations from experiments and accidental fires has enabled engineers to model these systems and verify the results against real behaviour.

2 The Fire Hazard

Fire is one hazard to which all operations are exposed. In 2013, the estimated direct losses for structural fires in Ontario was $636 million (Ontario Ministry of Community Safety & Correctional Services, 2016). Direct losses refer to the cost to repair or rebuild a structure that has experienced a fire. In addition to the direct losses, one must consider the indirect losses. These are the costs associated with business interruption from the fire as the operation goes through a recovery phase. Current literature suggests that the indirect losses are a function of the direct losses, as shown in equation [2] below and adapted from (Ramachandran, 1998):

[2] $IL = c(DL)^b$

where

IL = indirect loss
DL = direct loss
c,b = regression coefficient

For a business occupancy, the regression coefficients are c = 0.203 and b = 1.146 (Ramachandran, 1998). However, it must be noted that this regression analysis for indirect losses was based on 1976 data. To the best knowledge of the authors there appears to be a lack of recent studies that relates indirect fire losses to direct losses. The emphasis appears to have been on direct loss quantification as this directly impacts the insurance industry. It is expected, however, that the ratio between indirect losses and direct losses for fires is only going to increase as urbanization drives up the value of operations within buildings and the cost of the building itself dwindles in comparison to business costs. This is the impetus behind this research work, as project stakeholders are becoming increasingly risk aware and are asking their consultants to help mitigate risks associated with indirect losses. This is a novel approach to designing for fire safety which, it will be seen, typically just considers life safety and in some cases considers property protection for maximum realizable fire events.

3 Design for Fire

In most contemporary Canadian buildings, the fire safety strategy is still prescribed by the building code. The performance of these solutions is not quantified, and the emphasis of the building code is on life safety as communicated by the objective and functional statements of the acceptable solutions.

Globally, performance-based fire design (PBFD) is seeing increased usage for both new design and the assessment of existing structures. This allows designers to work with all project stakeholders to determine what the specific performance criteria of the building is, what analysis tools will be used, and what methods are acceptable for specifying realistic design fires. In this way, the designer is able to deviate from the prescriptive code solutions and deliver an alternative solution with equal or better performance. With performance-based design, property protection is seeing increased consideration since the performance of the structure itself can be quantified and it can be demonstrated if the structure stands or not.

However, property protection is just one goal that can be met with PBFD. Additionally, the authors believe there is opportunity for also assessing business protection. This acknowledges that the property itself exists to serve the purpose of an operation, and that with PBFD the impact of the fire on the operation can be understood and accounted for. Figure 2 illustrates this progression from prescriptive code solution to a performance-based approach enabling resilience.

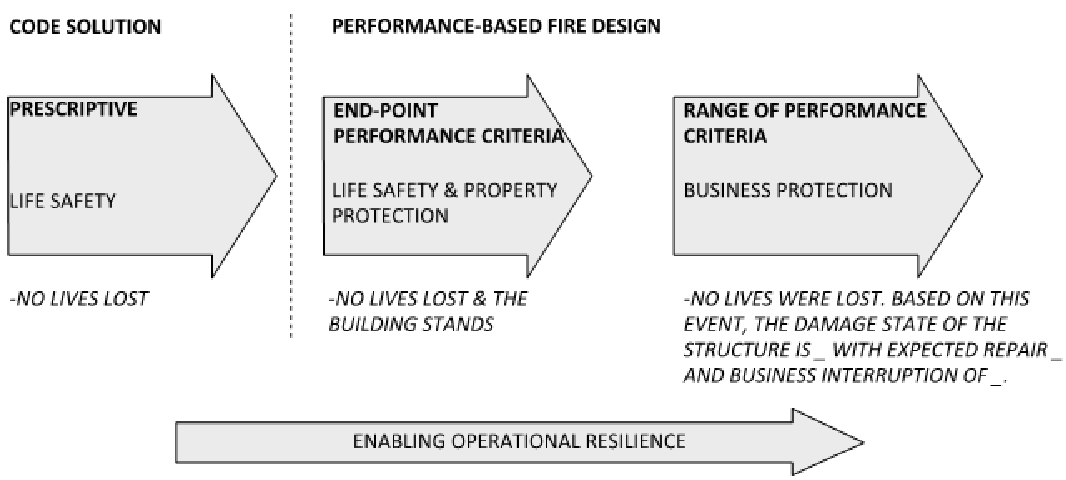

Figure 2: Progression of fire design as an enabler of operational resilience

4 Defining Performance Limits

A crucial aspect of designing for resilience is that it is completely dependent on the operation. This means the stakeholders must all be engaged and that the performance levels reflect the demands and needs of the specific operation. The SFPE Engineering Guide to Performance-Based Fire Protection (SFPE, 2007) is one document that lays out a process for performance-based design which has precedent for being used in Canada and has been accepted by the Authorities Having Jurisdiction (Bartlett, 2005). Inclusive is a step for developing performance criteria, which from a resilience perspective must be bounded ranges of threshold values to relate to a set of performance levels.

There is currently a lack of available research or data on what criteria constitute different levels of performance for steel during the design stage. By looking at the literature for indicators of what necessitates repair of a steel structure after a fire, we can deduce criteria from this for different levels of performance. In terms of performance levels, it is proposed to define three levels of performance, each having a different impact on the ability of the operation to recover following a given event. These performance levels are summarized in Table 1 below.

Table 1: Performance levels, descriptions and requirements

Performance Level		Generic Description	Client Specific Requirements*
Life Safety	**LOP-L**	• occupants can safely egress in time • damage state and repair scope unknown • fire effected area will likely require extensive repair and rebuilding	Long-term disruption • downtime is unknown, any essential services and operations to be temporarily relocated
Occupancy	**LOP-M**	• occupants can safely egress • fire effected zone requires repair, the scope of which may interfere with adjacent units	Short-term disruption • overall building has a downtime of 1 business day • fire compartment has a downtime of 5 business days
Operational	**LOP-H**	• occupants can safely egress • fire effected zone requires only aesthetic cleanup and repair, adjacent units can be occupied immediately following the fire	Immediate occupancy • overall building has a downtime of fire duration + hour • fire compartment has a downtime of 1 business day

* note, these requirements are given as an example and will be project and stakeholder dependent. Some requirements may not be readily quantifiable during analysis with current data available for verification, these represent various research needs in fire safety engineering.

4.1 Performance of Structural Members

Current literature suggests that following a fire event, any members displaying local buckling, bowing, twisting, or distortion will require repair and/or replacement (Ingham, 2009). Based on these, temperature limits are often used to define the extent of member distress in order to specify repair. The literature provides several such temperature limits for structural steel. First, at roughly 315ºC, it is suggested that local failures begin. The material itself will still retain all of its structural properties upon cooling up to a temperature of roughly 500°C (Ingham, 2009), and at 650ºC it is suggested global buckling can be expected (Tide, 1998). These temperatures are merely estimates based on material behaviour and testing, and the exact temperatures that a member may fail at are a function of gravity loading, thermal loading, restraint, and/or material properties.

4.2 Compartmentalization

The ability of the structural floor to keep the fire contained to the compartment of origin is crucial both for life safety and for mitigating the scope of repair required. In ambient design, it is typically assumed that partitions susceptible to cracking will begin to crack at a deflection L/360, where L is the span of the floor. It can be seen that partition performance is related to the curvature of the floor. Full-scale fire test observations suggest that structural stability of floors is maintained up to a value of L/20 (Wang, Burgess, Wald, & Gillie, 2013). These two deflection limits can provide the bounds for LOP-L and LOP-H. Within these bounds, a value must also exist for LOP-M, the limit at which partitions go from being distressed but repairable to a state where they must be fully replaced. This value has been selected as L/50 but will require further testing and modelling to verify.

4.3 Performance of Structural Connections

In addition to the local and gross member stability referred to in Section 5.1, the connections are important to consider since a beam expanding under thermal load may itself be acceptable but still imposes large forces at the connections. This is true both under heating and cooling phases of the fire. It is not economical nor feasible to design all connections for the actual force to be expected during a fire. Testing has shown that certain connection types of connections, if ductile, will deform and dissipate the connection forces calculated during analysis. (Selamet & Garlock, 2011) (Yu, Burgess, Davison, & Plank, 2009)

The concept of Demand Capacity Ratio (DCR) is proposed to be borrowed from contemporary Canadian blast design (CSA, 2012). With this methodology, one calculates the demand placed on a connection during the event and relates this back to the capacity under ambient conditions. This is proposed at the fire limit state in lieu of calculating actual deformations and stress relaxations at the connection as well as the complex thermal gradients in the connection relative to the gross member. Further testing and modelling is required to confirm if DCR is applicable for structural fire design, however it is beyond the scope of this initial discussion and is just one indicator of structural performance in fire.

4.4 Performance Level Summary

Sections 4.1 to 4.3 are summarized in Table 2 below. The deflection limits of Section 4.2 were translated into actual deflection values based on a floor span of 15m to be used in Section 5.

Table 2: Criteria for levels of performance

Performance Level		Deflection	Gross Member Steel Temperature	Connection DCR	
				Flexible	Rigid
LOP-L	*Life safety*	0.75m	650°C	4	1
LOP-M	*Occupancy*	0.3m	500°C	2	1
LOP-H	*Operational*	0.1m	315°C	1	1

5 Case Study of a Typical Canadian Composite Steel Building

A brief case study below is illustrated to show how PBFD can be applied to a structure to have the fire limit state quantified for a range of performance levels. A series of fire events will be considered, such that the stakeholders can have different types of responses planned ahead of time for varying sizes of fires. The building considered is a composite steel building currently under construction in Ontario. The fire safety strategy for the building is fully code compliant and follows all prescriptive requirements of the Ontario Building Code 2012.

A single bay will be considered to determine how varying sizes of compartment fires immediately below that bay affect the structure. The structural response will be quantified throughout the duration of the fire as well as the cooling phase. The floor plan of the building with the structural arrangement is shown in Figure 3 below.

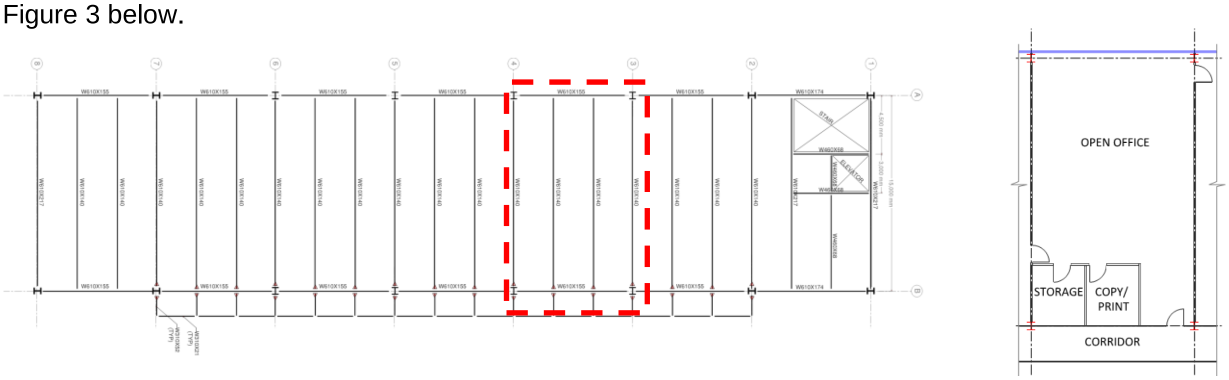

Figure 3: Structural plan of building floor (left) and architectural plan of study compartment (right)

Referring to the architectural plan of the floor in Figure 3, it can be seen that the space immediately below the bay being studied is an open plan office with two smaller rooms adjacent to a corridor. For the purpose of this analysis, it is assumed that the compartment is an open 9m x 15m office.

5.1 Design Fires

The design fire considered for this paper is a Eurocode Parametric Fire (CEN, 1991-2002). The time-temperature curve for the fire is defined in equation [3] below, and is derived from curve-fitting an extensive series of time-temperature curves that Pettersson et al. derived based on heat transfer analyses of compartment fires (Petterson, Magnuson, & Thor, 1976).

[3] $\theta = 20 + 1325(1 - 0.324e^{-0.2t^*} - 0.204e^{-1.7t^*} - 0.427e^{-19t^*}$

where

θ	=	Compartment gas temperature
*t**	=	Nonphysical time parameter, equal to $t\Gamma$
Γ	=	Dimensionless quantity.

The reader is referred to the Eurocode (CEN, 1991-2002) or Wang et al. (2013) for a more thorough derivation of the Eurocode Parametric fire. Other design fires to be considered are beyond the scope of this paper, however; the authors are currently considering these.

In the analysis, one variable in the derivation of the design fire was modified to create a series of design fires. In this way, it is easier to describe the range of fires considered and translate this to specific events for the stakeholder. Specifically, the fuel load was modified to create five different design fires. Physically, this fuel load would relate to the actual combustible material in the compartment at the time of the fire, and how much of this fuel was combusted during the fire. The design fuel load was calculated as 642 MJ/m^2 using the Eurocode formulation based on occupancy. This was take as the peak fuel load, denoted q=100% herein, with other possible design fires using a fraction of this design fuel load ranging from q=80% to q=20%. The resulting time-temperature curves considered as the design fires is shown in Figure 4 below.

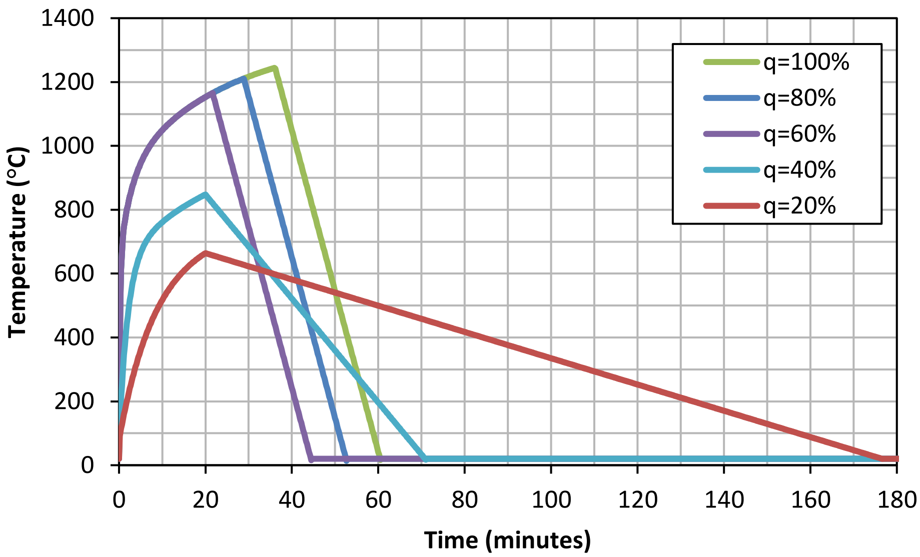

Figure 4: Design fires considered

5.2 Analysis Results

The first step in any structural fire analysis is to determine the temperatures within the structure, including both the spacial distribution of temperatures and their variation with time. The floor plate was analyzed for the design fires using the finite element software program SAFIR, a special purpose software for the thermal and structural analyses associated with fire (Franssen, 2012). SAFIR first performs a thermal analysis on all the structural elements in 2D, and the user subsequently imports the results of these thermal analyses into a 3D model for a structural analysis. SAFIR is currently a preferred analysis software for fire design in Canada due to its low cost, previous validations, and abundant documentation. The results of the thermal analysis are shown in Figure 5. Because of the nature of a parametric design fire and the assumption that the entire compartment can be represented by this temperature history, this temperature profile is the same along the length of the floor beams. Temperatures in Figure 5 are measured at the bottom flange which experiences the highest fire temperatures.

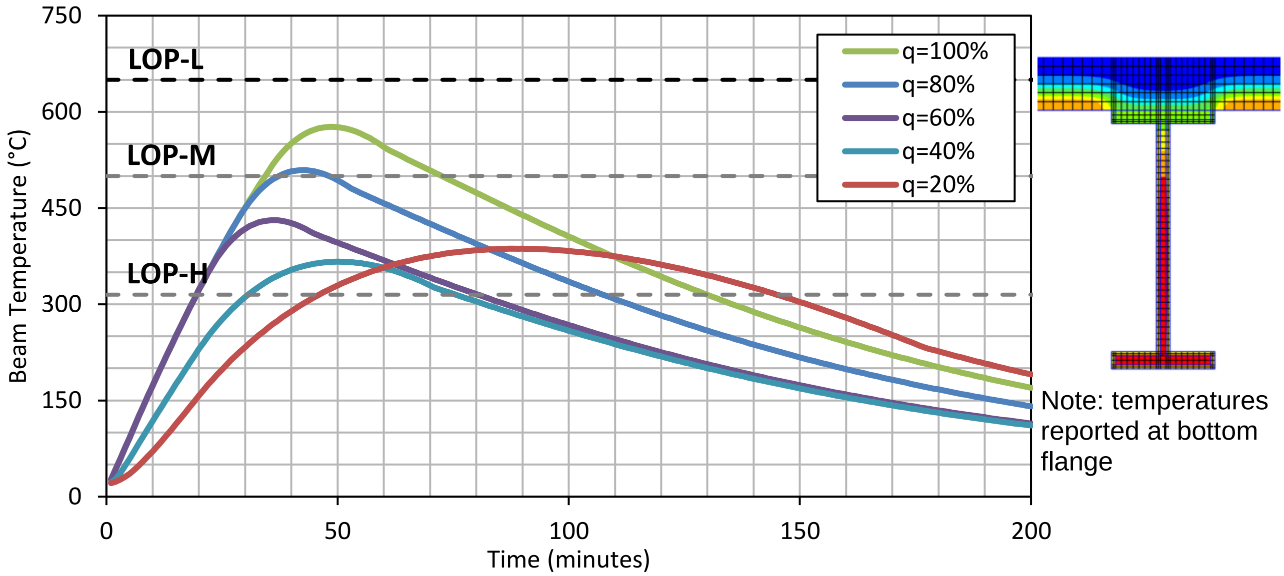

Figure 5: Temperature in the steel beams during design fires (W610x140)

Following the thermal analysis on all beam and slab elements affected by fire, the structural analysis was performed on the floor plate. The analysis was a nonlinear dynamic analysis, capturing all material degradation effects due to elevated temperature as well as thermal expansion and geometric nonlinearity. The nonlinear analysis was required since material properties change with temperature over time, while a dynamic analysis was performed to allow load redistribution if any instabilities occurred due to lateral torsional buckling of the beams. The composite floor exhibited very large deflections representative of tensile membrane action and remained stable throughout the duration of the fire. The deflected shape of the floor is shown in Figure 6, with the deflection-time histories of the five design fires shown in Figure 7. Lastly, the connection forces are shown in Figure 8.

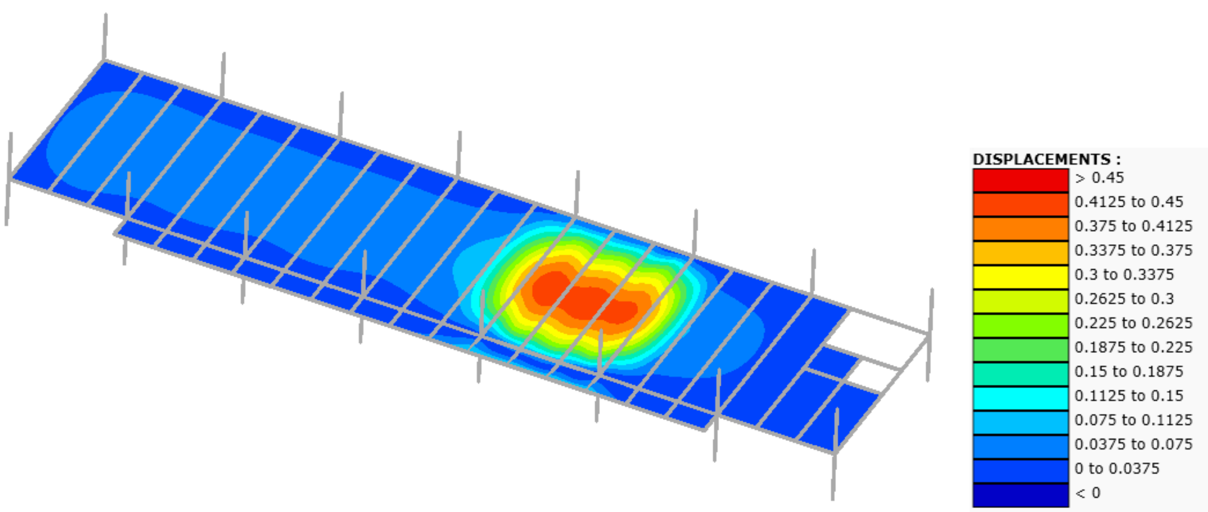

Figure 6: Deflected shape of floor

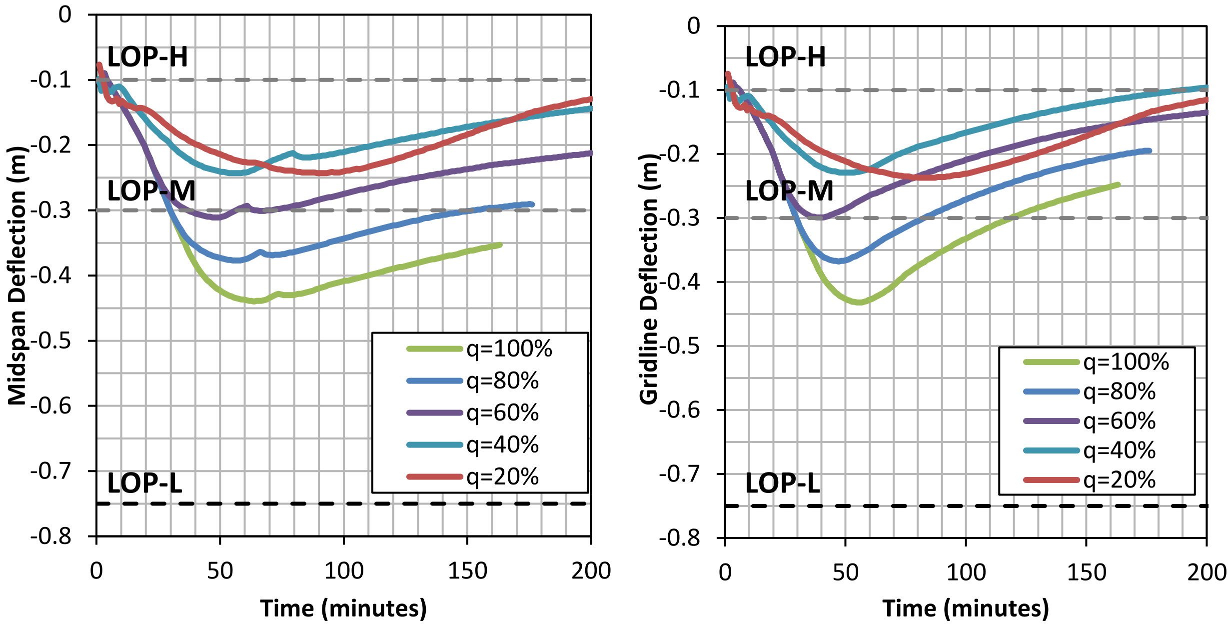

Figure 7: Deflection results at centre of floor (above left) and along gridline under partitions (above right)

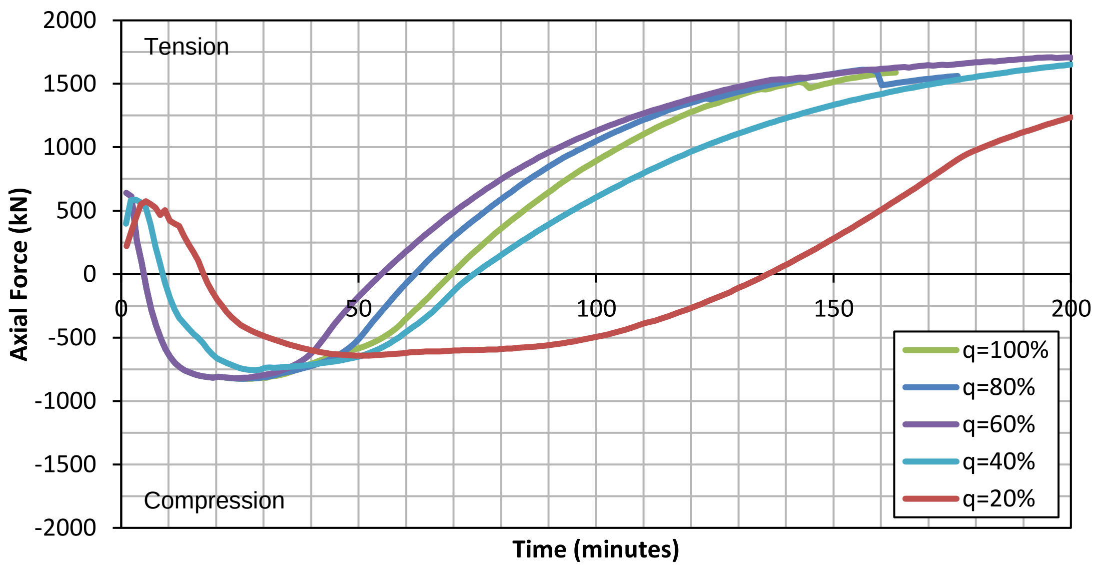

Figure 8: Connection forces in steel beam above fire compartment

6 Discussion

Looking at the analysis results, it can be seen that the results often conflict with one another. One metric may suggest the structure has a medium level of performance for a certain size of fire, while another metric shows a low level of performance for that same fire.

In terms of steel temperatures, all sizes of design exceed the 315ºC limit that the literature suggests local failures may begin. Further analysis indicated that the fuel load must be below 10% of q_{max} for the temperatures to remain at this level. Up to a fire size of just below 80%, the structure still has a medium level of performance. This suggests there is a set of small to medium size fires that the stakeholders can plan for ahead of time that will require localized repair to the compartment of origin, but adjacent compartments can be occupied immediately. For larger fires, above 80% of q_{max}, the structure requires extensive repair and the tenants of that space should anticipate relocating their operation temporarily.

The deflection results tell a similar story. Up to a fire size of 60% of q_{max}, the structure has a medium level of performance. These small to medium fires will require the partitions to be repaired, but the adjacent spaces should be occupied within the next business day. Beyond 60% of q_{max}, these larger fires cause significant deflections to the floor which require extensive repair. These deflections will inherently impact the tenant space above the compartment that the fire originated in and the adjacent compartments as shown in Figure 6.

Connection forces also illustrate an important consideration at the fire limit state. Figure 8 shows very large tensile forces developing in the connections as the structure cools. The differential cooling of the steel beams and the concrete slab, combined with the large plastic deformation remaining in the slab, introduce tensile forces well beyond anything considered during design. As discussed in Section 4.3, the connection must either be designed as ductile elements to relieve thermal strains or must be designed to resist the axial forces, with the DCR being proposed as a possible indicator of performance.

With the performance of the building quantified for a range of fire events, the impact on operational resilience can be understood. Referring to Figure 1, it was shown that resilience is a direction function of the operations ability to recover functionality following an event. Figure 9 illustrates two possible ways that quantifying performance to fire can improve resilience.

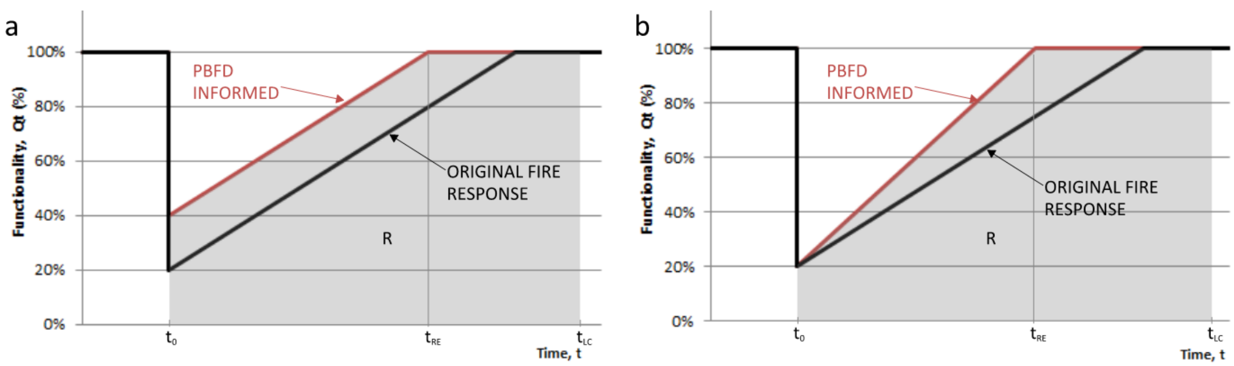

Figure 9: Impact on resilience of performance improvements (a) and recovery improvements (b)

During the design stage or through retrofit of the structure, the performance of the structure to a fire event can be improved. This can be through increased fire protection, intelligent re-arrangement of structural members, reduction or mitigation of the fire risk itself, etc. By doing so, the initial functionality loss due to a fire event is minimized. This is shown in Figure 9a it can be seen that the resilience is improved without any change to the rate of recovery. Additionally, without any retrofit or improvement to the structure, the stakeholders can use the information provided by the PBFD and improve their recovery plans for fire events. This does not change the initial loss in functionality due to the fire but it does increase the rate of recovery. Resilience is again improved as shown in Figure 9b. It should be noted that the slope of the recovery function has been assumed linear for illustration, but the actual slope will be a function of the specific operation and their ability to mobilize and recover following an event.

7 Conclusion

This paper has demonstrated a methodology for assessing a structure for a fire event and assigning levels of performance to it based on the structural response. This was done for a range of parametric design fires, from small fires to the largest possible if the room had its expected fuel load. The practical implications of this analysis are that structures can be designed to perform during a fire in such a way that the operation within that structure can better respond to and recover from the fire. This enables operational resilience within the structure. The results of this structural fire analysis can directly feed into a business continuity plan for that operation and a series of recovery plans can be created for different sizes of fire events within the space.

This analysis has strictly looked at the effect of the fire on the structure. The effect of active suppression systems, sprinklers, has not been accounted for and is beyond the scope of the current discussion. These would greatly reduce the structural temperatures during a fire but would cause extensive water damage. As well, the damage state of structures following fires requires further study, as does the expected scopes of repair for different damage states following a fire. The performance criteria presented in Table 2 are the authors suggestions based on the available literature. This analysis has been a proof of concept that structural engineers can more intelligently consider the fire limit state in design and consider a range of performance levels that will impact their client in different ways.

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