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Assessment of reinforced concrete strength degradation under molten salt spill event

Ahmed Elhattab[1], Matthew Clavelli[2], Nikolay Doumbalski[3], Andrew Lingenfelter[4], Brian Song[5],

[1] Senior Structural Engineer, Kairos Power, Alameda, California, USA, elhattab@kairospower.com
[2] Senior Engineer I, Kairos Power, Alameda, California, USA, clavelli@kairospower.com
[3] Principal Structural Engineer, Kairos Power, Alameda, California, USA, doumbalski@kairospower.com
[4] Lead engineer, Kairos Power, Alameda, California, USA, lingenfelter@kairospower.com
[5] Civil Structural Senior Manager, Kairos Power, Alameda, California, USA, song@kairospower.com

INTRODUCTION

Molten salt reactors (MSRs) are gaining considerable traction in modern small modular reactors (SMRs) due to their inherent safety features and operational advantages. These reactors use molten fluoride salts as a coolant and, in some designs, as the fuel medium, enabling operation at high temperatures and low pressures. This characteristic boosts safety by reducing the risk of high-pressure coolant loss accidents.

One of the key safety incidents that requires investigation is to study the consequences of losing one of the inventory management systems for the molten salt, which could result in dropping a large amount of molten salt onto the concrete slab of the reactor building. The damage to the structural members needs to be investigated, considering the unique loading conditions applied by the FLiBe.

Molten salts, particularly fluorides such as FLiBe, which is a combination of lithium and beryllium fluorides, showcase distinct characteristics and demonstrate excellent heat transfer capabilities and specific heat [1] [2]. FLiBe's specific heat is roughly 2414.17 J/(kg·K). If 1 ton of Molten Salt is spread over a 2 m x 2 m surface area, it will take about 30 hours for the temperature to decrease by 100°C. This suggests that the concrete component will endure high temperatures for a prolonged period. According to ACI 349, the permissible average surface concrete temperature during accidental cases is 177°C. FLiBe is typically utilized at high temperatures of around 700°C. Referring to the previous example, the Molten FLiBe will require roughly 4 to 6 days to reach the allowable concrete temperature limit. This paper will examine the impact of maintaining such high temperatures on concrete over an extended duration.

The approach adopted to assess the integrity of the reinforced concrete involves developing a non-linear thermal analysis coupled to a non-linear structural analysis to evaluate the degradation of concrete and steel over time due to high temperatures. This thermal analysis will consider the variations in concrete and steel thermal properties at different temperatures. In parallel, the structural analysis will focus on the degradation of the mechanical properties of both materials when subjected to elevated temperatures. The software employed for these analyses is the LS-Dyna FEA tool. The methods for modeling temperature-dependent thermal and mechanical properties are aligned with Eurocode 2 and 2,3, [3] [4] which provide guidelines for modeling reinforced concrete and steel structures exposed to fire. This Eurocode method will simulate the effects of a significant FLiBe spill onto the concrete surface. The Eurocode framework is suitable for temperatures up to 1200 °C. The Eurocode methodology applies in this context because the FLiBe operating temperatures range between 600 and 700 °C. The primary distinction between fire loading and FLiBe spill loading is that the latter begins at an elevated temperature that decreases over time, while fire loading initiates from ambient temperatures rising over time. Nonetheless, the proposed approach effectively addresses both scenarios regarding material behavior.

This paper discusses the proposed modeling approach, outlining the steps for conducting coupled nonlinear thermal/structural analysis. The modeling approach is validated through a full-scale fire loading test conducted by Anvari et al. [5]. This validation addresses both the thermal and structural responses of

the proposed finite element method (FEM). Finally, the modeling approach is employed to evaluate the damage to a 4ft thick concrete slab resulting from a FLiBe spill event, as illustrated in Figure 1.

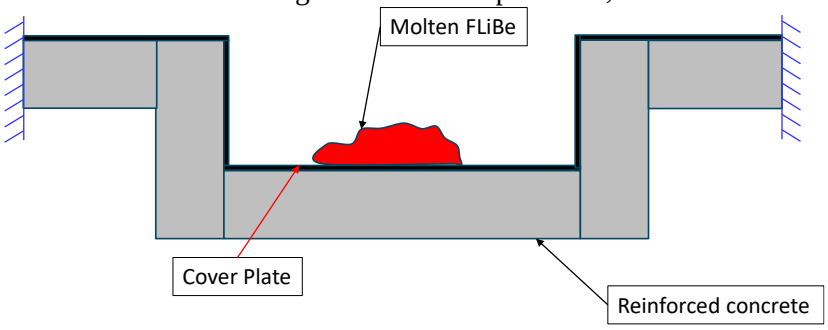


Figure 1. Problem configuration

EUROCODE METHOD FOR MODELING STRUCTURES UNDER ELEVATED TEMPERATURE.

The Eurocode method for modeling Structures at elevated temperatures focuses on establishing temperature-dependent properties in the constitutive models to reflect thermal behavior and strength degradation accurately. This approach has two components: A) Thermal properties and B) Mechanical or structural properties.

The thermal properties of the constitutive model encompass temperature-dependent characteristics for steel and concrete. Specific heat and thermal conductivity are defined as temperature-dependent properties. Figure 2 and Figure 3 illustrate the variations in specific heat and thermal conductivity for steel and concrete with respect to temperature. It is worth mentioning that concrete presents three specific heat curves that correlate with moisture content. These curves reveal a peak specific heat at 115 °C, which is linked to the evaporation of trapped water.

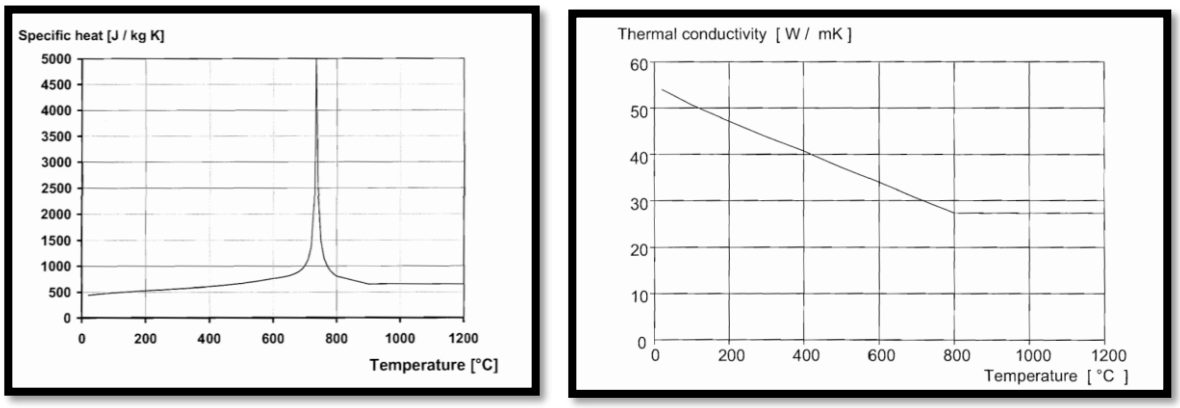


Figure 2. Thermal properties of the steel material

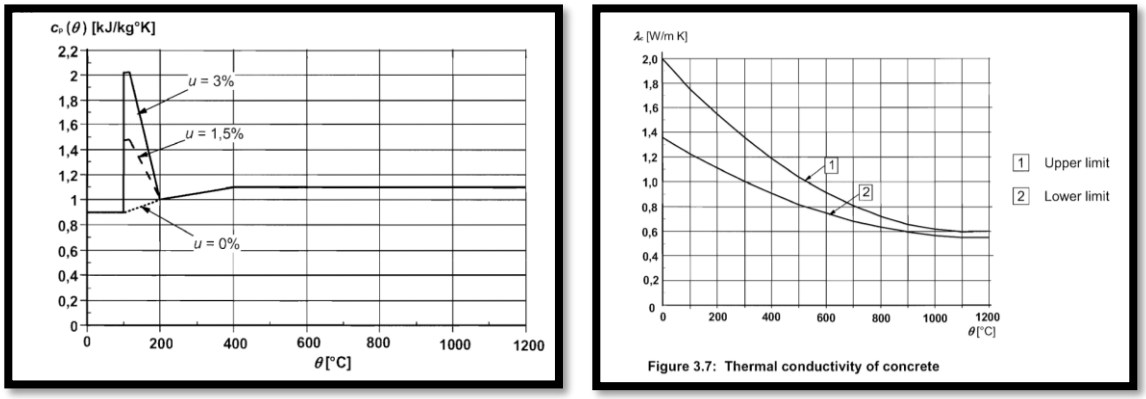
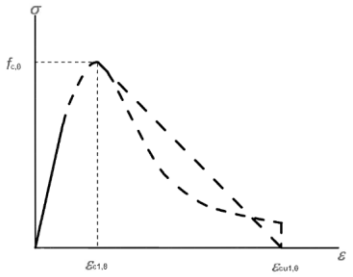


Figure 3. Thermal properties of the concrete material

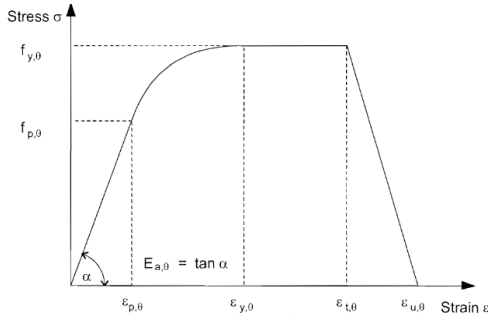
The Eurocode method outlines the mechanical properties of steel and concrete based on material temperature. Figure 4 presents the Eurocode equations and parameter values at various temperatures necessary for establishing the stress-strain relationship of concrete at these temperatures. Similarly, Figure 5 provides the equations and parameters for developing the stress-strain relations of steel. These two curves are the foundation for calculating material degradation at elevated temperatures. The range of temperatures considered spans from 20 °C to 1200°C. The equations specified by the Eurocode will be implemented in the Finite Element Model to define a temperature-dependent constitutive model. Figure 6 also presents the stress-strain relationship for 60 ksi steel and 6 ksi concrete, which is based on the methodologies outlined in Figure 4 and Figure 5. Like the stress-strain relationship, thermal elongation is a temperature-dependent property for both materials. Figure 7 presents the thermal elongation as a function of temperature for the two materials.



Range	Stress $\sigma(\theta)$
$\varepsilon \le \varepsilon_{c1,\theta}$	$\frac{3\varepsilon f_{c,\theta}}{\varepsilon_{c1,\theta}\left(2+\left(\frac{\varepsilon}{\varepsilon_{c1,\theta}}\right)^3\right)}$
$\varepsilon_{c1(\theta)} < \varepsilon \le \varepsilon_{cu1,\theta}$	For numerical purposes a descending branch should be adopted. Linear or non-linear models are permitted.

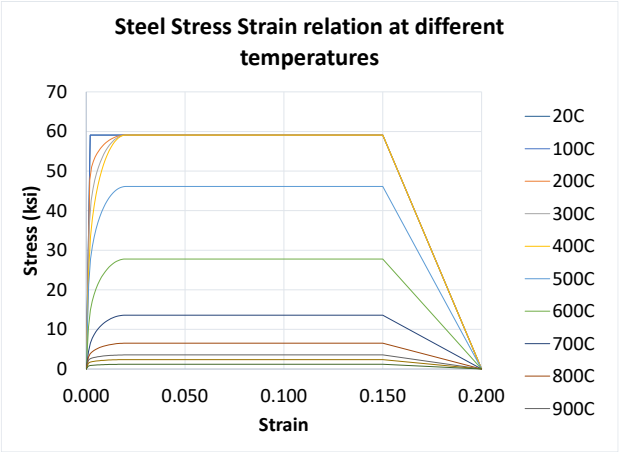
Concrete temp. θ	Siliceous aggregates			Calcareous aggregates		
	$f_{c,\theta}/f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$	$f_{c,\theta}/f_{ck}$	$\varepsilon_{c1,\theta}$	$\varepsilon_{cu1,\theta}$
[°C]	[-]	[-]	[-]	[-]	[-]	[-]
1	2	3	4	5	6	7
20	1,00	0,0025	0,0200	1,00	0,0025	0,0200
100	1,00	0,0040	0,0225	1,00	0,0040	0,0225
200	0,95	0,0055	0,0250	0,97	0,0055	0,0250
300	0,85	0,0070	0,0275	0,91	0,0070	0,0275
400	0,75	0,0100	0,0300	0,85	0,0100	0,0300
500	0,60	0,0150	0,0325	0,74	0,0150	0,0325
600	0,45	0,0250	0,0350	0,60	0,0250	0,0350
700	0,30	0,0250	0,0375	0,43	0,0250	0,0375
800	0,15	0,0250	0,0400	0,27	0,0250	0,0400
900	0,08	0,0250	0,0425	0,15	0,0250	0,0425
1000	0,04	0,0250	0,0450	0,06	0,0250	0,0450
1100	0,01	0,0250	0,0475	0,02	0,0250	0,0475
1200	0,00	-	-	0,00	-	-

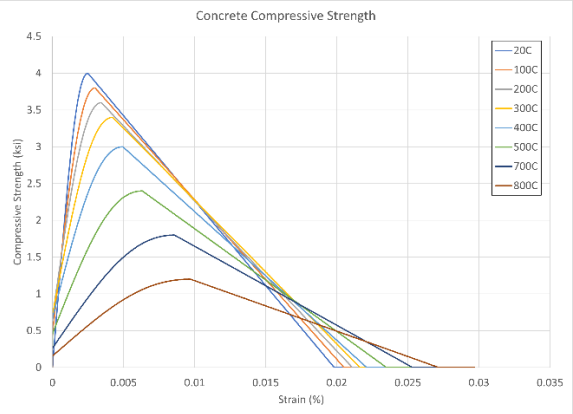
Figure 4. Concrete stress-strain relationship as a function of the temperature [4]



Strain range	Stress σ	Tangent modulus
$\varepsilon \le \varepsilon_{p,\theta}$	$\varepsilon E_{a,\theta}$	$E_{a,\theta}$
$\varepsilon_{p,\theta} < \varepsilon < \varepsilon_{y,\theta}$	$f_{p,\theta} - c + (b/a)\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0.5}$	$\frac{b(\varepsilon_{y,\theta} - \varepsilon)}{a\left[a^2 - (\varepsilon_{y,\theta} - \varepsilon)^2\right]^{0.5}}$
$\varepsilon_{y,\theta} \le \varepsilon \le \varepsilon_{t,\theta}$	$f_{y,\theta}$	0
$\varepsilon_{t,\theta} < \varepsilon < \varepsilon_{u,\theta}$	$f_{y,\theta}\left[1 - (\varepsilon - \varepsilon_{t,\theta})/(\varepsilon_{u,\theta} - \varepsilon_{t,\theta})\right]$	-
$\varepsilon = \varepsilon_{u,\theta}$	0,00	-
Parameters	$\varepsilon_{p,\theta} = f_{p,\theta}/E_{a,\theta}$ $\quad \varepsilon_{y,\theta} = 0{,}02$	$\varepsilon_{t,\theta} = 0{,}15$ $\quad \varepsilon_{u,\theta} = 0{,}20$
Functions	$a^2 = (\varepsilon_{y,\theta} - \varepsilon_{p,\theta})(\varepsilon_{y,\theta} - \varepsilon_{p,\theta} + c/E_{a,\theta})$ $b^2 = c(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} + c^2$ $c = \frac{(f_{y,\theta} - f_{p,\theta})^2}{(\varepsilon_{y,\theta} - \varepsilon_{p,\theta})E_{a,\theta} - 2(f_{y,\theta} - f_{p,\theta})}$	

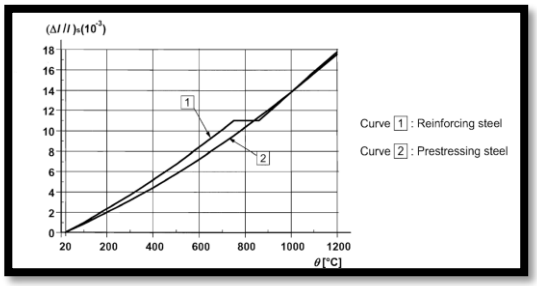
Figure 5. Steel stress-strain relationship as a function of temperature [4] [3]





(a) (b)

Figure 6. The stress-strain relationship for the steel and concrete material at different elevated temperatures was developed following the Eurocode method.



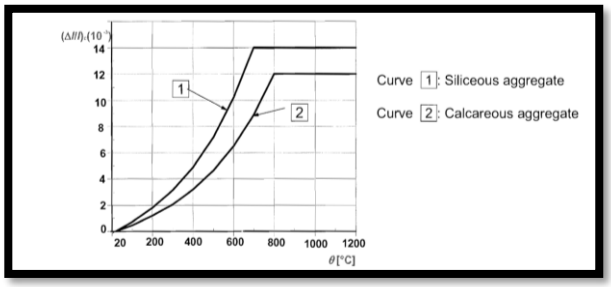


Figure 7. Thermal elongation for steel and concrete as a function of temperature.

FINITE ELEMENT MODELING APPROACH.

The analysis is conducted using the LS-Dyna explicit finite element modeling tool. It consists of two phases: a non-linear thermal analysis followed by a non-linear structural analysis. The two analyses are independent. The authors explored a coupled approach for a simple problem and found no substantial differences in results between the coupled and decoupled methods, provided the thermal boundary conditions remain relatively unchanged. The time needed for the decoupled analysis is about nine times less than that required for the coupled approach, prompting the authors to proceed with the decoupled method.

For the thermal analysis, the steel and concrete constitutive models are defined using the Thermal Isotropic Temperature Dependent Constitutive model (Mat-T10). The input properties for the material are the material density, thermal conductivity, and specific heat. The properties are defined as an input curve matching the values provided in Figure 2 and Figure 3. The FLiBe material is also modeled using the Thermal Isotropic Constitutive model, but the properties are kept constant with temperature change (Mat-T01). The FLiBe is modelled in the thermal analysis to provide the heat source, but was not modelled in the structural analysis (except its weight was included as a pressure load on the concrete). The following values were used for the FLiBe material:

Density : 2413kg/m^3
Specific Heat : 2386 J/(kg K)
Thermal conductivity : 1.1 W/(m K)

Different constitutive models are used for the structural analysis of steel and concrete. For concrete, a temperature-dependent constitutive model (MAT 172) is used to simulate the concrete material. The constitutive model properties are thermally sensitive, and the material data and equations governing the behavior (including thermal properties) are taken from Eurocode 2. The material can represent unreinforced concrete, reinforcing steel only, or a smeared combination of reinforced concrete with evenly distributed reinforcement. The rebars and the concrete material are defined using this constitutive model. The input to the material card is the concrete compressive strength and the steel yield strength. Using a single-element test, the authors tested the constitutive model and extracted the corresponding stress-strain relationship at different temperatures. The developed curve from the single-element test is the curve shown in Figure 6 for both concrete and steel rebars.

In the FLiBe spill analysis, stainless steel will be used as a liner. The steel plate material cannot be modeled with MAT 172; instead, it should be modeled using MAT 255 (piecewise linear plasticity thermal material model). This material requires user-defined temperature-dependent stress-strain curves. For this material, the curves are defined using the equations presented in Figure 5. The true stress-strain curves used for the steel liners (SS 304 material) are presented in Figure 8.

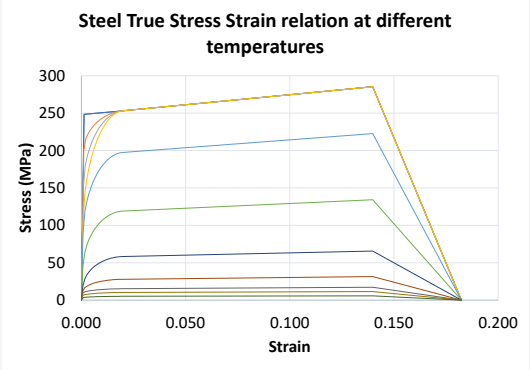


Figure 8. True stress-strain curve for the SS 304 material

VALIDATION AND VERIFICATION FOR THE MODELING APPROACH

Two validations are required for the proposed modeling approach: 1) validating the thermal response of the constitutive model, specifically the through-thickness temperature response over time, and 2) validating the structural/mechanical response of the constitutive model. To validate these two features, a full-scale test for a steel plate composite wall subject to high temperature loading (mimicking a fire loading) is used. Anvari et. al developed the test [5] to investigate the performance of Steel Plate composite walls under fire loading conditions. The authors did have five different specimens tested under different conditions. This paper utilizes specimen (CW-1) since it has through-thickness thermocouples to monitor the through-thickness temperature.

The experiment is presented in Figure 9. The specimen dimensions are 9” x3 ft x6 ft. The specimen was subjected to elevated heat on all faces using ceramic heaters. The specimen is axially loaded to mimic

an SC wall under fire. The specimen had thermocouples installed throughout its thickness to measure the heat transfer through the thickness of the wall. The specimen was heated using 14 different ceramic heaters. The surface temperature of the steel plate facing each heater was recorded during the experiment. These temperatures are used as input surface temperatures to the thermal model.

The results of the thermal analysis are presented in Figure 10. The through-thickness temperature from the experiment is compared to the analysis results and presented in Figure 11. As presented, there is a good match between the model and the experiment.

The output temperatures from the thermal analysis are then used as input to the structural analysis. The steel plate buckling at the end of the analysis time is compared to the one recorded during the experiment and is presented in Figure 12. As presented, there is a good agreement between the model and the experimental results. Similarly, the displacement of the and the FE model are compared as presented in Figure 13 and it shows good agreement.

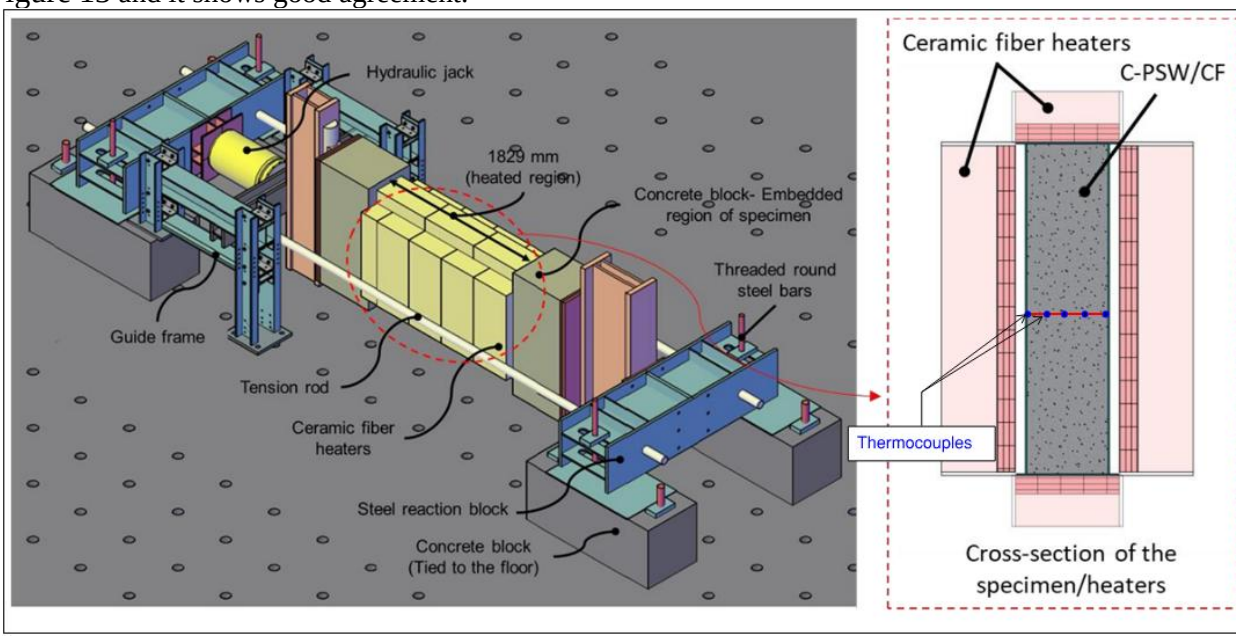


Figure 9. Steel Plate composite wall subjected to elevated temperature [5]

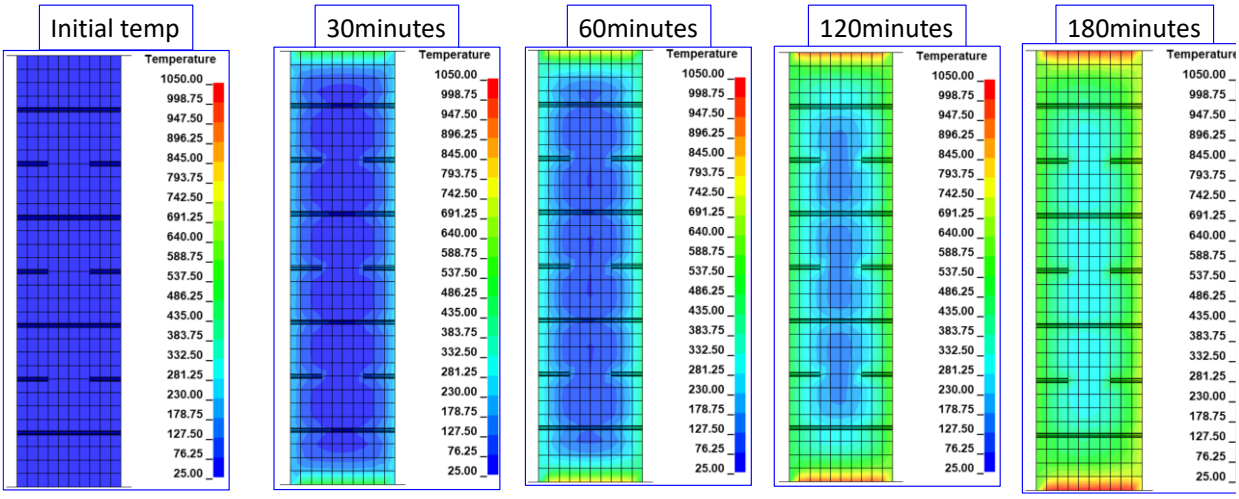


Figure 10. Through thickness temperature change over analysis time.

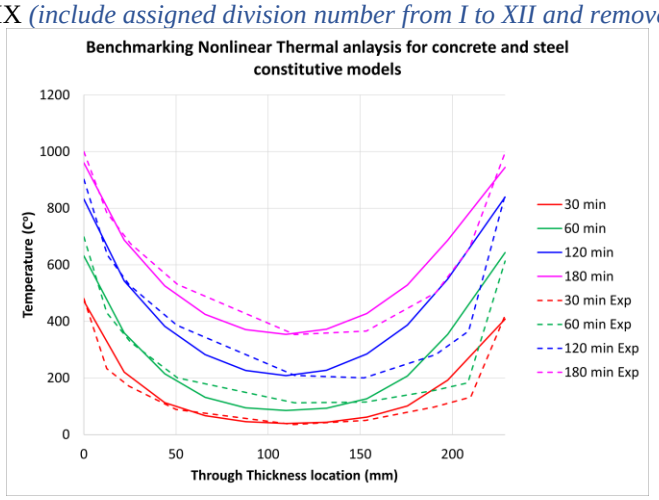


Figure 11. Comparing the through-thickness temperature from the FE model and the experiment thermocouples

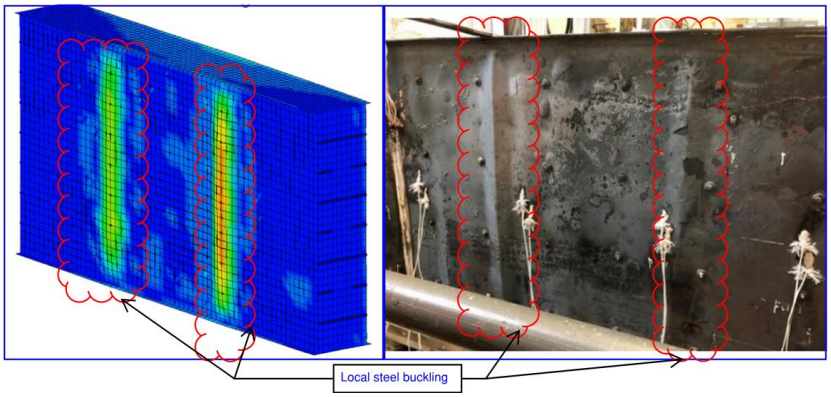


Figure 12. Local steel plate buckling (FE model versus experiment results

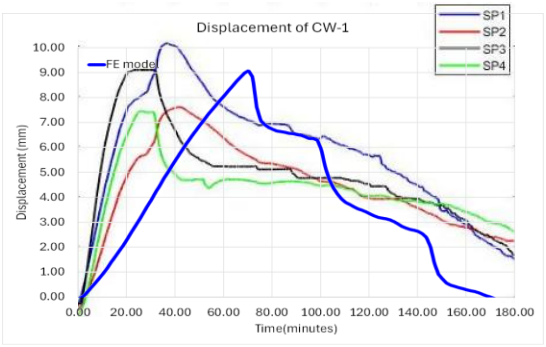


Figure 13. Displacement of the specimen (FE model-Blue and different displacement gauges from the experiment)

FE RESULTS FOR THE FLIBE SPILL EVENT

The problem configuration for the FLiBe spill problem is presented in Figure 14. The presented figure shows an initial design for an FLiBe pit where a large amount of FLiBe could be collected after a major leak event. The pit is 8ft deep and 18ft wide. The pit is lined with a ¼" Stainless Steel (SS-304 steel) liner. The model is representative of a 3ft strip. The amount of collected FLiBe in this pit is approximately 5 tons. The thermal and structural boundary conditions are as presented in the figure.

The Thermal analysis is first conducted by simulating the instantaneous FLiBe drop in the pit through a temperature increase in the FLiBe material. The FLiBe temperature is assumed to be 690 °C the temperature is increased gradually in 4 minutes using a smooth curve as presented in Figure 15. Afterwards, the temperature BC is terminated, and the FLiBe is left to cool down by convection and radiation boundary conditions presented in Figure 14. Heat transfer through radiation between the FLiBe and SS liners on the side is ignored in this analysis. The thermal response over time is presented in Figure 16. As presented, the FLiBe temperature decayed to approximately 100C° after approximately 3 days of the spill event. The temperature through the thickness over time is presented in Figure 17. From the figure, most heat is localized in the concrete's first 3 to 6". Accordingly, thicker covers are recommended to mitigate damage to the reinforcing rebars. The Structural analysis was then performed using the thermal response. The concrete damage is presented by showing the maximum principal strain in the concrete (Figure 18). The presented limits are associated with a 1.5mm crack width. The figure presents a significant tensile concrete cracking through the section. These cracks are due to the instantaneous elongation in the liners attached to the top surface of the pit. Figure 19 presents the buckling in the steel plate due to the thermal expansion. The flexural demand at the mid-section is extracted to assess the effect of the thermal expansion on the structure. The flexural demands increased from 5kip.ft/ft to 290kip.ft/ft after applying the FLiBe thermal loads. The section stabilizes again around 10kip.ft/ft after the section is cracked to release the thermal demands. The 4ft slab was observed to have significant damage. However, the section maintained its integrity and did not collapse.

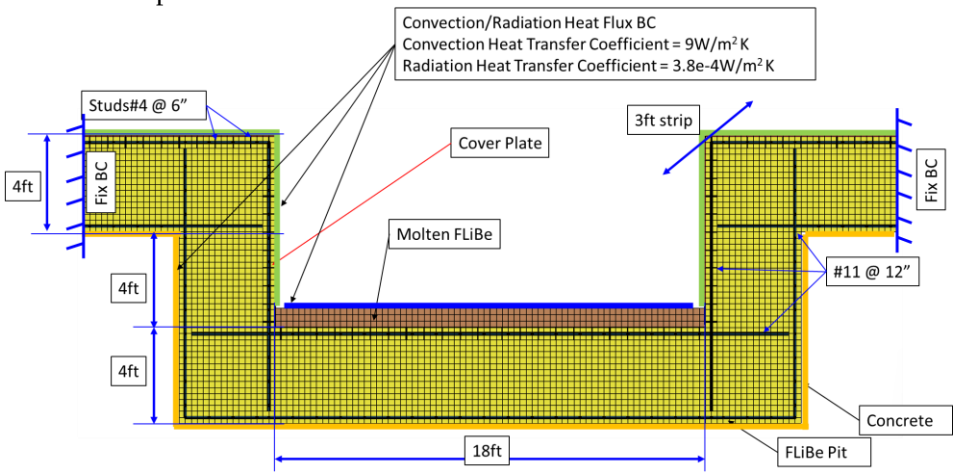


Figure 14. Configuration of the FLiBe spill event

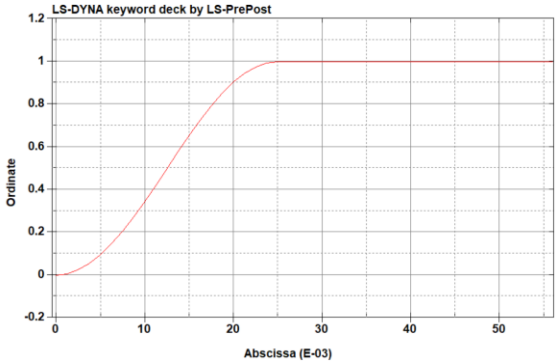


Figure 15. Ramping FLiBe temperature

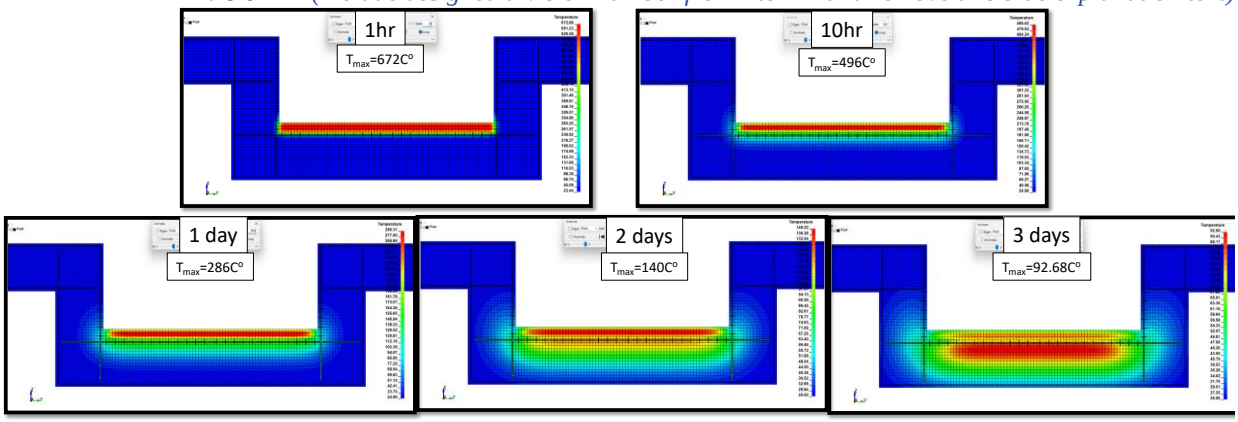


Figure 16. Temperature contour plots over time

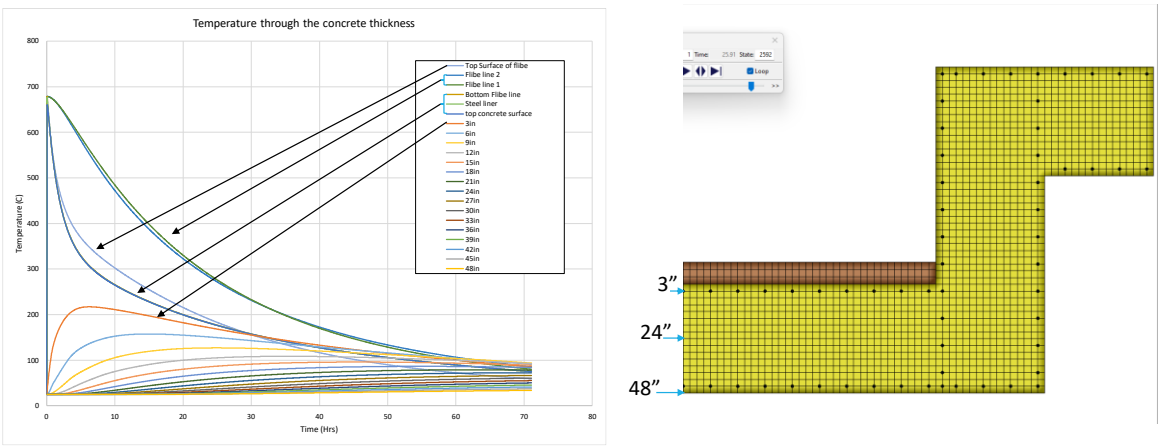


Figure 17. Temperature through the thickness of the concrete

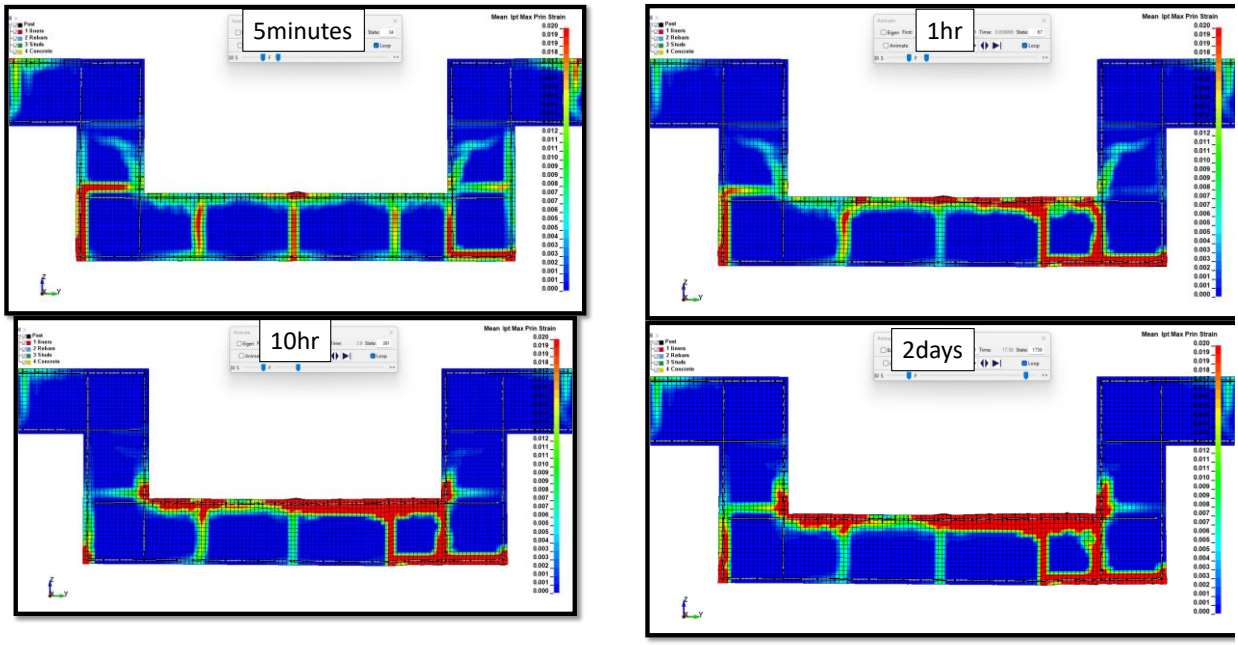


Figure 18. Concrete cracking is presented using Maximum Principal Strain contour plots

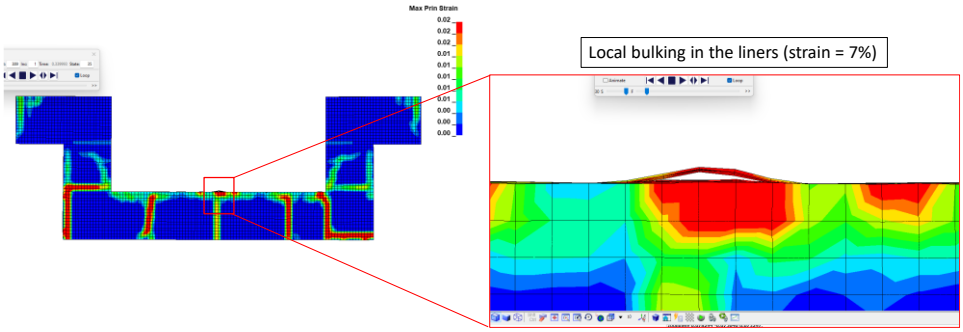


Figure 19. Steel plate buckling at the end of the analysis

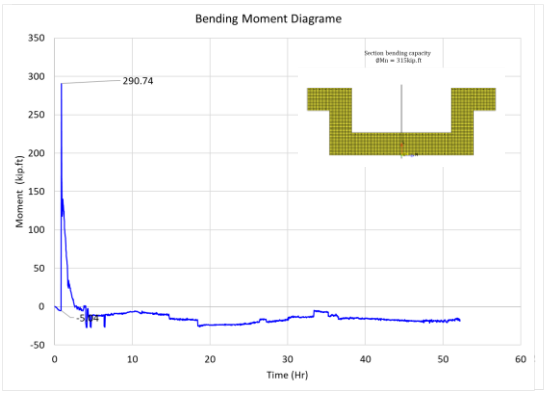


Figure 20. Temperature through the thickness of the concrete

CONCLUSION

Using an advanced modelling approach, this paper investigates the consequences of dropping a large quantity of molten FLiBe on a concrete slab. The proposed modelling approach has first been validated using full-scale experiment data. Both thermal and mechanical responses have been validated. The validated modelling method is then used to assess the consequence of dropping 5 tons of molten FLiBe onto a 4ft concrete slab. The results indicated significant cracking in the concrete, which is wide enough for the FLiBe material to leak. However, the used model included ¼" of SS liner. The analysis indicated that the liner did not tear out and just ended with some local buckling. Since the liner did not tear out, adequate sealing can be provided against the FLiBe leak. Additionally, the analysis indicated that the concrete section maintained resistance after the damage and did not collapse.

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