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The Behaviour of Stiffened Steel Plated Decks Subjected to Unconfined Pool Fires

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***Abstract*—A recent fire experiment carried out by the Health and Safety Laboratory (HSL), Buxton, UK as part of the joint industry project to study the behaviour of spreading hydrocarbon pool fires on steel substrate has revealed that structural integrity of steel-plated deck may not be achieved within the fire resistance period for unprotected steel members. Based on this outcome, it was required that further investigations were conducted. This paper examines the ultimate capacities of steel-plated decks subjected to running hydrocarbon pool fires. I observed from the numerical calculations that the type of fire as well as the interplay temperature between two successive fire regions for the travelling fire scenarios can significantly affect the way the deck will fail. A preliminary sensitivity study was conducted to examine how changes in some design variables would affect the response parameters. The findings suggest that plate and stiffener web slenderness ratios can notably affect both the axial capacity and the vertical displacement of the deck. It was also discovered that although lateral pressure can markedly affect the deformation capacity of the deck, it does have little or no effect on its axial capacity under fire conditions. I also suggest that the repeated buckling of the plate caused by the travelling fire induces alternating tension and compression in the deck which is likely to have implication on the integrity of the welded connections.**

***Index Terms*—elevated temperature, running pool fire, sensitivity study, ultimate capacity**

I. Introduction

The behaviour of running hydrocarbon pool fires has not been fully studied to gain an appropriate knowledge about the hazards they pose to structures and personnel. Structural and fire safety design of offshore structures is a broad and complex subject requiring the ability to manage the various components involved. The nature of hazards to which both personnel and structures are exposed in the offshore environments makes it more demanding. There is a need to examine the behaviour of offshore fires as well as offshore installations when exposed to high temperature loadings. However, previous research projects have demonstrated that travelling fires can be more severe than the case when the fires are assumed uniformly applied to the surfaces of structural assemblies [1]-[2].

A recent fire experiment carried out by the Health and Safety Laboratory, Buxton [3] to study the behaviour of pool fires spread on the deck surface of a simulated offshore platform has revealed it is possible structural integrity may not be achieved within the specified fire resistance period. Although BS 5950-8 [4] recommends that unprotected steel members should be able to withstand fire up to 30 minutes, before possible interventions can be offered some of the welded connections of the steel deck in the HSL fire test failed within a fire resistance period of 15 minutes. It is indicated, based on the unexpected result of the HSL fire experiments, that the simulated deck requires further investigation. Unfortunately, the only measured data were temperature profiles and heat fluxes. Displacement and stress data were not measured because the primary objective was to examine pool fire behaviour on the steel deck surface and it was not anticipated that the deck would fail unexpectedly.

The performance of plate elements under compressive and lateral loads has been investigated in detail for many years, and several parametric studies have been conducted to examine the effect of various design parameters on the collapse strength of plates [5]-[11]. However, a few studies have been identified on the collapsed strength of plates elements subjected to fire conditions [12]-[13]. Most of the studies carried out to examine the collapse strength of plate elements have generally considered individual structural members acting separately. In real structures, the interaction between structural elements continually takes place permitting the redistribution of loads from failed localised regions to other parts of the structures. In welded plate structures, the interaction between the plate elements as well as between the plates and the supporting members can be very vital to the survival of the structures.

Donegan [14] emphasized that the principal mechanism causing collapse in a fire is the release of potential energy when the strength and stability of the structure is reduced by the effects of the fire. The loss of structural strength and stability is a direct consequence of loss of material strength, material stiffness and thermal strains. A structural element under the influence of thermal loadings experiences thermal strains irrespective of the end conditions. For unrestrained end conditions, the effects of thermal elongation will produce strains without any change in the internal stresses.

It may be surmised that a structural deck has been considered as secondary steelwork which may be the reason why less emphasis is directed towards its evaluation in fire conditions. However, Donegan [14] asserted that when real fires are burning, the performance of secondary steelworks together with the effective control of combustion products plays a significant role in the safe evacuation of the platform. On the strength of this, platform decks need to be critically examined to be able to characterise their behaviour

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in fire conditions. Donegan [14] however, recommended that to avoid possible collapse of structural steelwork under localised heating, restraint to thermal expansion should be limited wherever possible without loss of strength and stability.

This study considers the performance of steel plated deck subjected to lateral loads under running pool fires that can result during accidental fires in offshore platforms. A series of calculations of load time and deflection time curves is carried out for plated decks of different plate slenderness, stiffener web slenderness, and weld stiffness. The slenderness definition for the stiffened plate is shown in Table 1. To allow for the variation of plate slenderness and stiffener web slenderness the width of plate and depth of stiffener were kept constant while varying the thickness of each element. The stiffener/plate cross sectional area ratio was approximately varied between 0.21 and 0.056 for the stiffened plates A to D. The relationship between this variable and plate slenderness ratio is approximately linear, which can be represented by (1).

$$\lambda = 35.03\left(\frac{A_s}{A_p}\right) - 0.6398 \qquad (1)$$

where A_s = cross-sectional area of stiffener web, A_p = cross-sectional area of plate and λ = plate reduced slenderness ratio.

The heating of the plate elements will induce the tendency for the plates to expand thus inducing compressive forces in the plate due to the constrained boundaries. The buckling of the plates caused by the induced compressive forces together with the reduced strength at elevated temperatures may further reduce the collapse strength of the plates. Previous research by Quiel and Garlock [15] has suggested that the local buckling capacity of fire-exposed steel members can be affected by the degradation in strength and stiffness associated with an increase in steel temperature. The compressive capacity of plate members will decrease at the onset of buckling. With further decrease in material properties at elevated temperatures, the collapse strength of the buckled plates will be significantly reduced.

Table 1 Matrix of slenderness limits for the plate and stiffener web

Stiffened Plate	$t_p(mm)$	$t_w(mm)$	b/t_p	λ	d/t_w	A_s/A_p
A	10	5.7	150	6.71	30.2	0.21
B	20	6.4	75	3.35	26.9	0.11
C	25	8.8	60	2.68	19.6	0.099
D	50	11.4	30	1.34	15.1	0.056

where *b*= width of plate; *d* = depth of stiffener; t_w = stiffener web thickness; t_p = plate thickness and λ = reduced slenderness ratio of plate.

II. MATERIAL BEHAVIOUR

The performance of material in relation to heat transfer and stress analysis is central to a proper understanding of the behaviour of structures in fire conditions. When a steel structure is exposed to fire, the temperature of the member will increase from ambient to a high level, thereby leaving the material to degrade in strength and stiffness. Eurocode 3 Part 1-2 [16] describes various models that can be used to determine the reduction for the mechanical properties of steels at elevated temperatures. The reduction factors for the mechanical properties of high strength steel at elevated temperatures are illustrated in Fig. 1. The stress-strain-temperature curve used to model the high strength steel and the welded connection is shown in Fig. 2. The yield strength of the steel and the weld used in this study is 420 MPa having an elastic modulus and Poisson ratio of 210 GPa and 0.3 respectively. The EC 3 stress-strain model for carbon steel at elevated temperatures, which explicitly excludes the effect of strain hardening, is illustrated in Fig. 3.

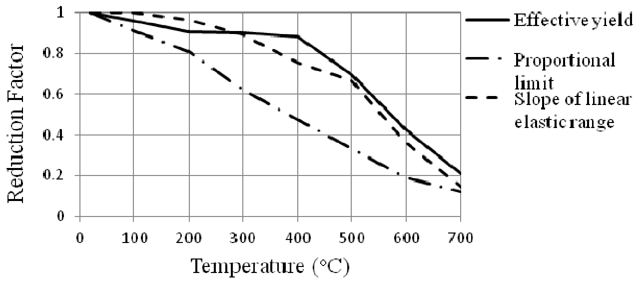

Fig. 1. The reduction factors for yield strength, proportional limit and linear elastic range for high strength steel S420M at elevated temperatures.

A spreadsheet is generally developed to compute these stresses and strains. Although stiffness computation is somewhat implicit, it should be noted that it is a function of the linear elastic range within the stress strain characterization. The conventional reduction factor and the stress-strain curves at elevated temperatures described in EC 3 are respectively shown in Figs. 4 and 5.

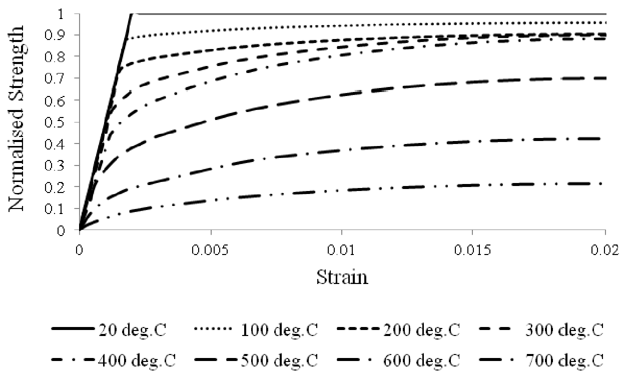

Fig. 2. Stress-strain curves for HS S420M steel at elevated temperatures.

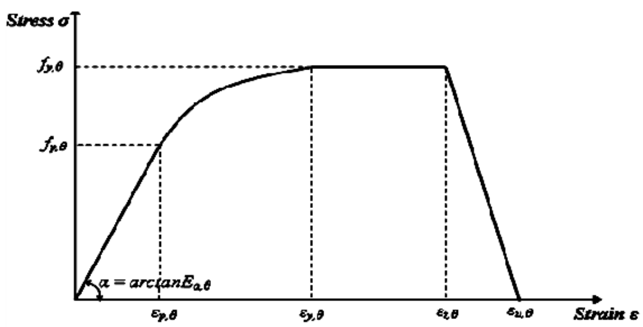

Fig. 3. Stress-strain model for carbon steel at elevated temperatures.

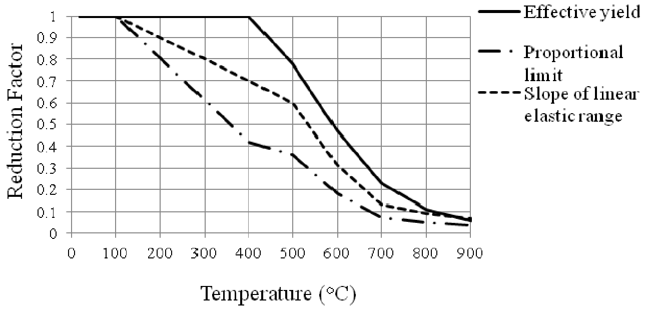

Fig. 4. The reduction factors for yield strength, proportional limit and linear elastic range for carbon steel at elevated temperatures.

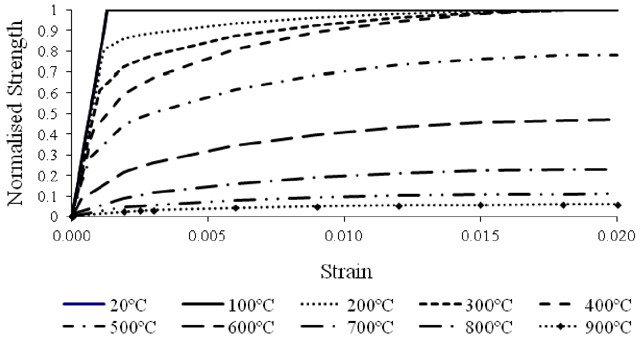

Fig. 5. Stress-strain curves for carbon steel at elevated temperatures.

III. FIRE MODELS

To examine the behaviour of localized fires, both the radiative and geometric properties of the fire flame are considered. Bukowski [17] remarked that a model may be accurate and yet the results from calculations can be seriously wrong if the input data do not correctly represent the conditions under investigation. Therefore, localized fire models are developed to represent the fire loading conditions under which the deck at the Health and Safety Laboratory (HSL), Buxton, UK [3] was tested. The flame height correlation of Heskestad [18] is adopted in the present study to calculate the fire loads. The heat release rate of the burning fuel can be characterized by the correlations given in (2) and (3). The power-law relation developed by Heskestad [19[-[20], in which the fire is assumed to attain a heat release rate of 1055KW in 150 seconds, is used to develop the localised fire model. This yields a convective fire growth coefficient of 0.0469KWs^{-2}. The power-law correlation is a t-squared fire model given in (4).

$$\dot{Q} = \dot{m}''\Delta h_c A \quad (2)$$

$$\dot{m}'' = \dot{m}''_\infty\left[1 - \exp(-k\beta D)\right] \quad (3)$$

where:

$\dot{m}''$ = fuel mass burning rate [kg/m^{2}s], $\dot{m}''_\infty$ = fuel maximum mass loss rate, $k\beta$ is a property of the fuel defined as the mean beam length corrector-flame attenuation coefficient product, D = the fuel fire diameter [m], A = pool (or spill) area [m^2] and Δh_c is lower heat of combustion [KJkg^{-1}].

$$\dot{Q} = \alpha_c\left(t - t_o\right)^2 \quad (4)$$

where:

$\dot{Q}$ is the heat release rate [KW], α_c is the convective fire growth coefficient [KWs^{-2}], t is time [s] from ignition and t_o is the virtual time at origin.

The methodology for developing the localised fire load implemented in this study is fully described in a previous paper [21]. Moreover, the measured temperature data from the HSL fire experiment [3] shown in Fig. 6 was implemented to simulate the actual fire loading. The calculated temperature profile for the localized fire derived from the power-law is shown in Fig. 7. The strategy is that the fire is assumed to start at the core of the deck and spread in the outward direction as illustrated in Fig. 8. The fire in the subsequent region will only start once that in the previous region has reached its maximum. Each fire load is assumed to occupy approximately 25% of the total plate area. This fire size has been demonstrated by Law *et al.* [2] to be the critical fire size in a spreading fire situation.

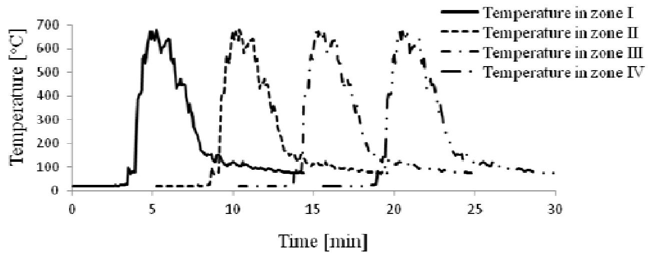

Fig. 6. The HSL measured fire temperature applied as localised travelling fires [1].

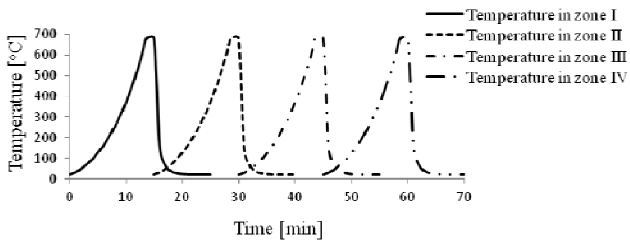

Fig. 7. Localized travelling fires based on the t-squared fire model.

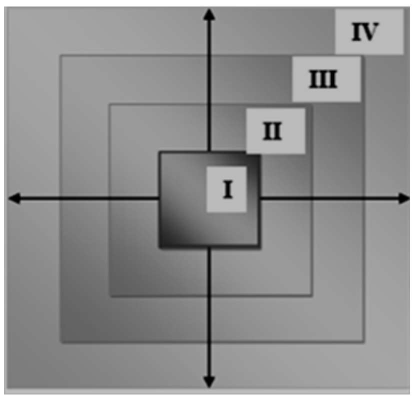

Fig. 8. Fire path (outward ring) used in the study

IV. MODELLING TECHNIQUE AND STRUCTURAL BEHAVIOUR

The numerical calculations were carried out using a general-purpose commercial code-ABAQUS [22], which has the capability of predicting the thermal and mechanical responses of structures based on the finite element method.

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The large displacement effects of the non-linear finite elements are considered using an updated Lagrangian formulation.

The modelling technique includes structural response while incorporating the HSL fire data and the localised fire load developed using the power-law methodology. The heat loads were directly applied incrementally to the surface nodes of the plate elements using the temperature profiles. However, in sequentially coupled stress analyses, the temperature fields are usually applied to the mechanical models as forcing functions to calculate the stresses and strains by means of incremental plasticity theory.

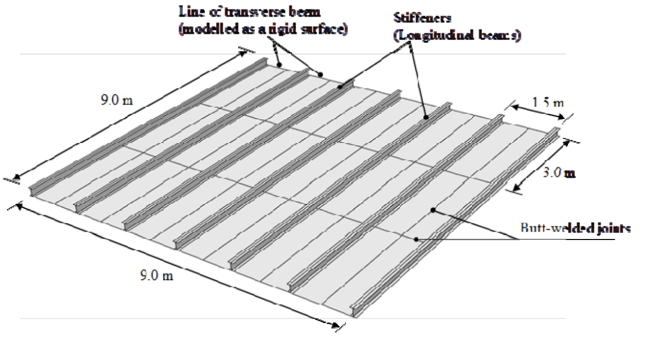

Fig. 9. Representative model of the steel plated deck.

The multi-linear stress-strain curves at various temperatures describe the non-linear behaviour of the material. The plasticity is calculated by the von Mises yield criteria and the associated flow rule. The deck models developed are based on the assumption that the behaviour of the plate can be represented by linear elastic shell elements with finite membrane strains. The deck which measures 9.0 m x 9.0 m x 0.01 m has two main transverse beams located at the ends and reinforced with several longitudinal beams spaced 1.5 m centers as shown in Fig. 9. The main beam spanning 9.0 m, consists of a 914 x 304 x 201 UB section while the transverse beam is made up of a 203 x 133 x 25 UB spanning 9.0 m long. These beams are not directly exposed to the fire; however it is possible for the beams to receive a fraction of the heat load by conduction.

The transverse beams were assumed as a rigid surface to which the longitudinal beams were attached. The longitudinal beams (stiffeners) and the plate elements were modelled using shell elements. The behaviour of the butt-welded connections in the deck assembly was assumed to be represented by a series of spring elements whose force-displacement characteristics is defined according to the model illustrated in Fig. 10. The model for the spring elements was characterized by the elevated temperature reduction factors for the stress strain model of Fig. 2. The material model often adopted in the analysis of steel structures is assumed to follow the von Mises yield criterion and the associated flow rule [23].

The general purpose non-linear finite element analysis code ABAQUS [22] adopts the Newton-Raphson approach to incrementally solve the resulting differential equations. In this condition, the mechanical strength and behaviour of a structural member need to be assessed by employing (5) and (6) [24]-[25].

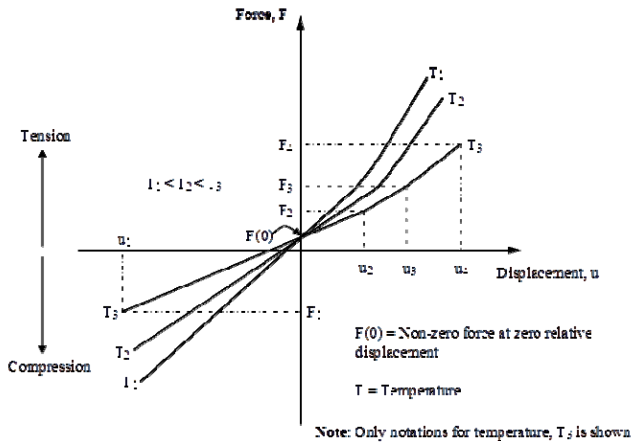

Fig. 10. A model spring stiffness characterisation at elevated temperatures.

$$\sigma_{ij,j} + f_i = 0 \tag{5}$$

$$\{\Delta\sigma\} = \{D^{ep}\}[B]\{\Delta U_e\} - \{C^{th}\}[M]\{\Delta T_e\} \tag{6}$$

where σ_{ij} is a stress tensor [Nm^{-2}], f_i is a body force [Nm^{-2}], $\Delta\sigma$ is the incremental stress [Nm^{-2}], D^{ep} is the elasto-plastic stiffness matrix, B is strain-displacement matrix, U_e is nodal displacement vector, C^{th} is thermal stiffness matrix, M is temperature shape function and ΔT_e is nodal incremental temperature matrix.

V. Results and Analysis

In principle, one of the measures for evaluating failure under serviceability limit state is the maximum out-of-plane displacement a structural member can reach under sustained loadings. As the deck deflects in the direction orthogonal to its plane when stressed, the carrying capacity weakens as a result of the plastification that may have taken place in some sections within the plate. Hence, as the mid-span vertical displacement of the deck increases its compressive axial capacity reduces correspondingly.

It is apparent from Table 1 that as the thickness increases the rigidity of the plate increases more in proportion than that of the stiffener thereby shifting the weaker zone towards the stiffeners. Thermal loading induces axial stresses in the restrained structural member while at the same time degrading the material properties at high temperatures. Consequently, as the plate and stiffener web slenderness ratios decrease the axial forces in the member will increase which may then result in increased vertical displacement. The calculated axial forces are shown in Figs. 11 to 14.

It is apparent from the comparison of Figs. 11 to 14 that the mechanical loadings will have no significant effect on the axial capacity of the deck. However, the mid-span vertical displacement can be very dependent on the mechanical loading as demonstrated in Figs. 15 to 18. Considering the vertical displacements of the deck shown in Figs. 15 to 18 it is evident that the type of fire plays a part in the way the deck will fail. For the travelling fire scenarios the interplay temperature between two fire regions need to be considered. In the HSL fire model the interplay temperature is approximately 120°C whereas that of the

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localized fire model is approximately 60^{o}C. This signifies that most of the vertical displacement of the deck under the HSL and localised travelling fires will be recovered in each stage as the fire moves from one zone to another as demonstrated in Figs. 16 and 18 for the thermo-mechanically loaded cases.

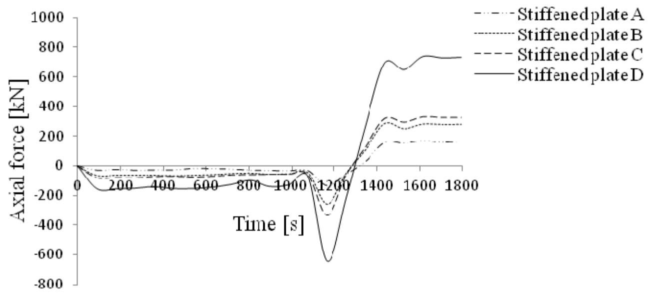

Fig.11. Axial force with HSL travelling fire under thermal loading.

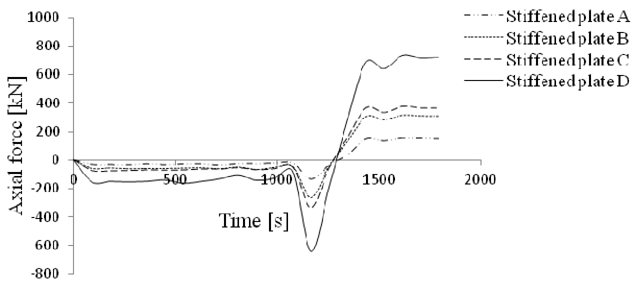

Fig. 12. Axial force with HSL travelling fire under thermo-mechanical loading.

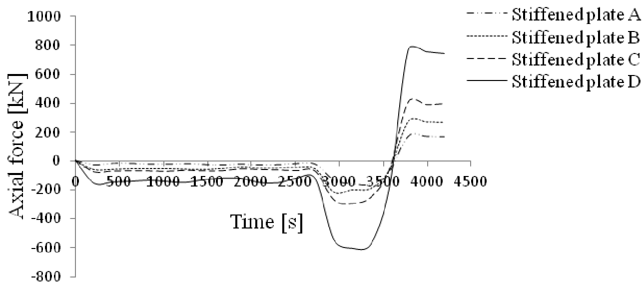

Fig. 13. Axial force with localized travelling fire under thermal loading.

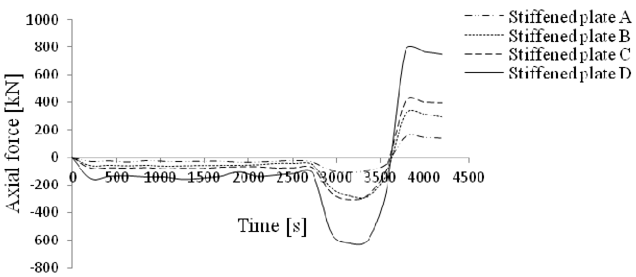

Fig. 14. Axial force with localized travelling fire under thermo-mechanical loading.

The deck appears to go into recovery as the temperature begins to cool down. As the fire moves to the next zone the induced compressive forces produced by the rising temperature allow the plate to displace further. The unstable vertical displacement of the deck plate under the travelling fire will continue to vary until the fire in the final zone begins to cool down. This phenomenon may adversely affect the integrity of the welded connections due to the repeated tension and compression induced in the plate. However, it is noted that the amount of recovery of the vertical displacement depends on the interplay temperature between any two connecting fire regions.

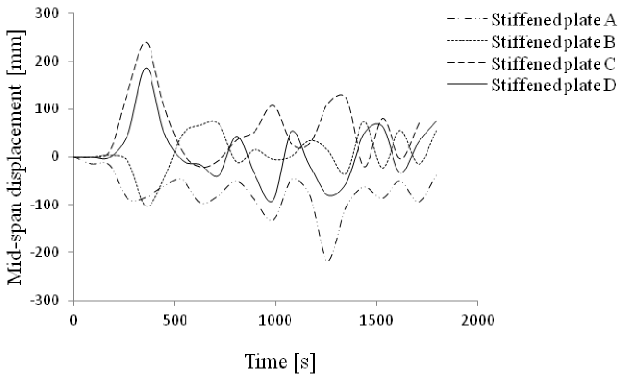

Fig. 15. Midspan vertical displacement with HSL travelling fire under thermal loading.

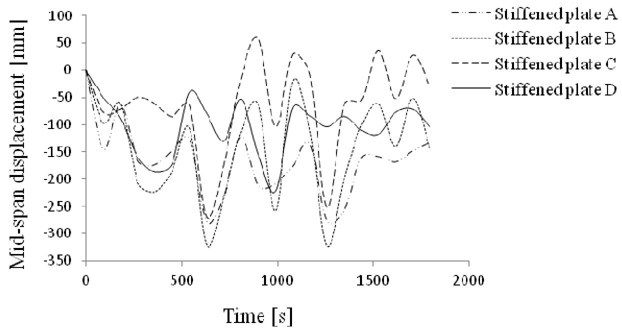

Fig. 16. Midspan vertical displacement with HSL travelling fire under thermo-mechanical loading.

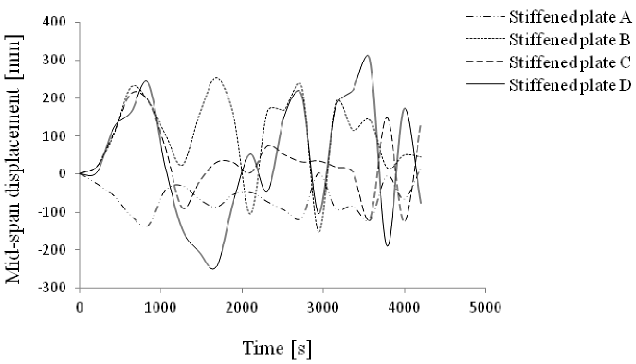

Fig. 17. Midspan vertical displacement with localized travelling fire under thermal loading.

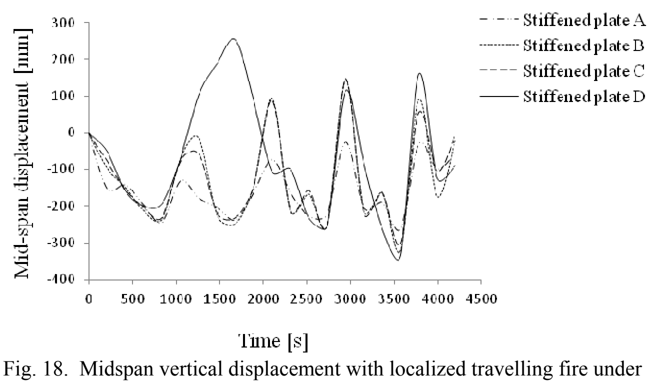

Fig. 18. Midspan vertical displacement with localized travelling fire under thermo-mechanical loading.

The effect of weld stiffness on the ultimate capacity of the deck was studied. Weld stiffnesses based on the EC 3

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design model and that of the high strength steel were utilized. This preliminary investigation shows that a slight variation in the weld stiffness will alter the deformation capacity of the plated deck and the effect on the axial capacity is rather insignificant as depicted in Figs. 19 and 20.

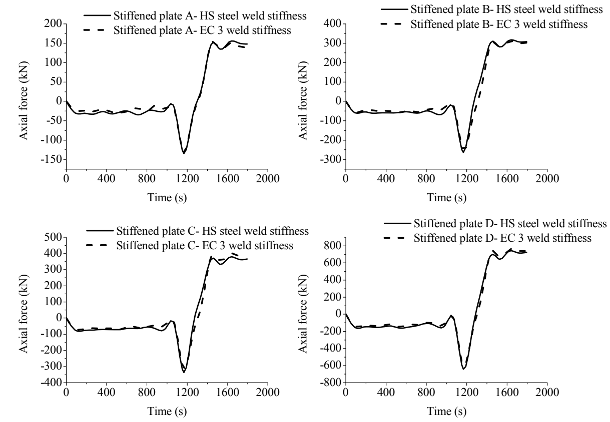

Fig. 19. Effect of weld stiffness on the axial capacity under the HSL travelling fire.

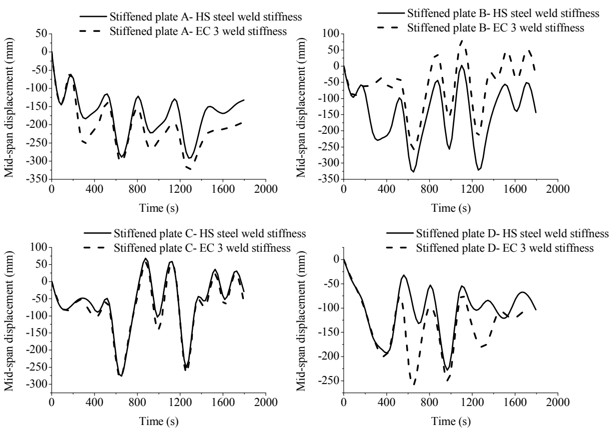

Fig. 20. Effect of weld stiffness on the midspan vertical displacement under the HSL travelling fire.

VI. Conclusion

The HSL fire experiment has shown that it is possible unprotected structural steel decks may not achieve the required fire resistance period under running pool fires. Unfortunately, no stress or displacement data were measured in the test. During the test structural failure was observed around some regions of the butt-welded connections. Nevertheless, based on this preliminary investigation, I suggest the following: A variation in plate and stiffener web thicknesses is likely to induce unexpected vertical displacements of the deck when subjected to thermal loadings. A small difference in the weld stiffness will alter the deformation capacity of the plated deck without significantly affecting its axial capacity. However, for stiffened plate C the effect of weld stiffness is insignificant indicating an optimum design for the plated deck. It should be noted that final conclusions cannot be drawn at this stage due to missing experimental data.

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