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Behavior of Earthquake Damage Steel Structures in Fire

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***Abstract*—Fire following an earthquake (FFE) has become a major threat for building, particularly in seismic areas. Many FFE events have caused a high level of damage and casualties. On the other hand, current design codes do not support a specific loading case for FFE. Moreover, the modern design philosophy for seismic design allows a certain level of damage that may affect the structure's vulnerability during a post-earthquake fire. Many previous studies have investigated the structural behavior of the building under FFE. This study is intended to showcase, theoretically and practically, the numerical analysis methods used in previous studies. The main objective is to improve the understanding of the performance of steel structures under a post-earthquake fire. A brief historical review of the numerical analysis methods is presented. It is observed that there are four stages of analyses adopted in the previous studies, which are structure system, earthquake analysis, fire analysis, and evaluation. However, different concepts and methods have been used in every stage. This study discusses the advantages and disadvantages of the design concept and the numerical analysis method. It was found that the key aspect of fire following an earthquake analysis is interpreting earthquake damage as an initial condition for the subsequent fire action. The 3D model is required since the composite slabs have a significant role in the frame's survival through tensile membrane action. Furthermore, several parametric fires must be considered to simulate a fire event after an earthquake.**

***Keywords*—Fire following the earthquake; thermal properties; fire engineering; mechanical properties; steel moment frame.**

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I. INTRODUCTION

The behavior of steel structures subjected to fire following an earthquake (FFE) has recently come to wide attention. Many recorded experiences indicate that FFE events cause more severe damage than earthquakes [1]. For instance, the post-earthquake fire resulted in 80% of the total damages in the 1906 San Francisco Earthquake. In the 1923 Tokyo earthquake, FFE destroyed more than 70% of the total buildings, and the death toll was over 140,000.

It is worth noting that the risk of FFE is not uniform since many earthquake events have not been followed by a fire, for instance, 2007 Yogyakarta (Indonesia) and 2011 Christchurch (New Zealand). However, the level of urbanization and industrialization results in dense gas, fuel, and electrical network that may increase the risk of FFE. In this case, FFE may create a chain of catastrophic events [1]. Recently, in the 2018 Lombok earthquake [2] and the 2019 California earthquake [3], several FFE occurred due to damage to gas mains. These events have shown that the potential of a post-earthquake fire disaster is still high.

These historical records indicate that the municipal structures are not designed for multi-hazard events such as FFE. Although fire protection systems such as a sprinkler may extinguish a fire in a normal situation, the systems may be damaged and not work properly due to an earthquake. On the other hand, the earthquake may devastate the pipework, which may cause a loss of water supplies. Therefore, more attention should be given to the fire protection system in seismic zones than non-seismic zones [4]–[6].

Despite its potential as a cause of major devastation, it is widely held that the FFE scenario is not considered a specific loading case in the recent design approach. In the philosophy of seismic design [7], [8], it is permissible to have a certain degree of damage to the structural members, making the building more susceptible when subjected to FFE. Due to the significant damages in the previous events, it is now essential to evaluate the behavior of steel structures under multi-hazard events such as FFE.

Steel is the main material in many industries. In the past decades, the steel industries have increased significantly. The largest customer of steel production is the housing and construction sector. The steel structure is relatively light due to a high ratio between strength and density. In addition, steel structure behaves ductile, which gives an advantage in the structure design for seismic. In contrast, since steel is sensitive material, there is a significant reduction of material properties such as strength and modulus of elasticity in elevated temperature, as shown in Table I.

TABLE I
REDUCTION FACTOR FOR STEEL AT ELEVATED TEMPERATURE [9]

Temperature	Reduction factor for yield	Reduction factor for elastic modulus
20 ºC	1	1
100 ºC	1	1
200 ºC	1	0.9
300 ºC	1	0.8
400 ºC	1	0.7
500 ºC	0.78	0.6
600 ºC	0.47	0.31
700 ºC	0.23	0.13
800 ºC	0.11	0.09
900 ºC	0.06	0.0675
1000 ºC	0.04	0.045
1100 ºC	0.02	0.0225
1200 ºC	0	0

Currently, limited studies are conducted on the behavior of steel moment frames under FFE. This study provides a brief historical review of the numerical analysis methods in previous studies. The key aim of this short review is to showcase, theoretically and practically, the numerical analysis methods used in the studies. Several aspects are highlighted, including numerical analysis and gaps among the studies that require further investigation.

II. MATERIALS AND METHOD

A. Methodology

Sequential analysis is required to evaluate the behavior of earthquake damage steel in fire [10], [11]. Fig. 1 illustrates the step of the sequential analysis that considers both earthquake and fire. It can be seen that the gravity loads are initially applied in the structure, and the next step is the application of earthquake load. Finally, the fire load is applied to the structure.

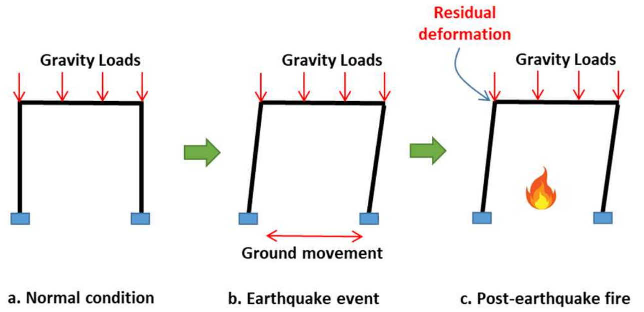

Fig. 1 Step of the sequential analysis

The main aspect of FFE analysis of structure is to determine the structural stage after an earthquake to represent the initial condition for a post-earthquake fire. It is theoretically difficult to provide information on earthquake damage because of the randomness and uncertainties of the structural properties and earthquake vibration [11]–[13]. Although the capability of computational modeling for an earthquake has been significantly improved, it remains challenging to provide accurate predictions of structural performance.

For simplicity, Della Corte et al. [14] introduced two forms of damage that can be used as the initial step for fire analysis, as follows:

- Geometrical damage is a change of structural geometry, such as inter-story drift, that may cause excessive P-delta effects.
- Mechanical damage is a reduction of mechanical properties that may initiate deformation in the plastic range.

There are three common analysis procedures to evaluate the structure's behavior: linear static, nonlinear dynamic and nonlinear static analysis. The linear static analysis is the most common and simplest method with the application of a single factor to accommodate dynamic effect and nonlinearity of material and geometric [15], [16]. Thus, this method cannot accurately predict the nonlinearity and dynamic effect.

On the other hand, nonlinear dynamic analysis is theoretically the most precise yet complex approach because it considers all types of nonlinearities. Besides, it requires time history ground motion data to simulate the dynamic effect. Pushover analysis has been introduced as a new technique to solve the abovementioned problem. This method considers both material and geometrical nonlinearity but does not need time history records to calculate the dynamic response. However, this is an approximation technique that may not accurately simulate the dynamic response of the structure.

In the pushover analysis, a nonlinear static analysis is performed to produce a pushover curve or capacity curve. A specific lateral load pattern is subjected to the structure [17]. Then, the load magnitude is incrementally increased until the target displacement is reached or the structure collapse, as illustrated in Fig. 2.

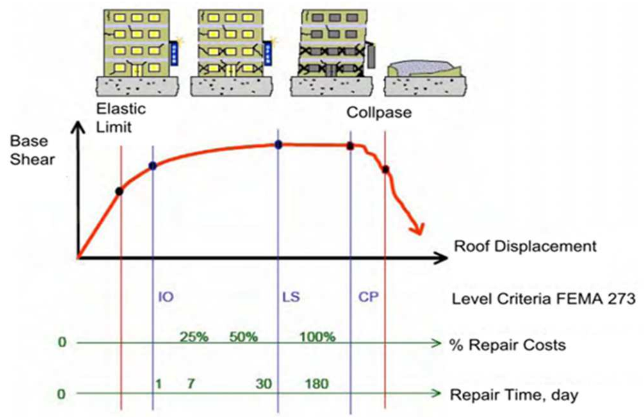

Fig. 2 Capacity curve of the building

The relationship between base shear and lateral displacement, called the pushover curve, expresses structures' global response against lateral loads. The target displacement, roof displacement during an earthquake, represents structural performance levels. It recently became a popular tool used as a new technique to evaluate the performance of buildings subjected to an earthquake [18]–[21]. It should be noted that the pushover analysis is an approximation approach, and the result may not represent the actual behavior of the structure. This is due to the fact that the load duration and dynamic effect are not considered in the pushover analysis.

According to ASCE 41-17 [7], there are four acceptable performance levels, i.e., Operational (O), Intermediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP). The performance level depends on the lateral displacement of the structure or inter-story drift ratio (IDR) as shown in Fig. 2. When the IDR is less than 0.7%, 0.7 - 2.5%, and 2.5 – 5%, the performance level of steel structure is IO, LS, and CP, respectively. Each category represents the damage level, as shown in Table II.

TABLE II
LEVEL OF BUILDING PERFORMANCES [7]

Level	Description
Operational (O)	Very little damage, temporary drift, the structure retains original strength and stiffness, and all systems are normal
Immediate Occupancy (IO)	Little damage, temporary drift, the structure retains original strength and stiffness; the elevator can be restarted, Fire protection still works
Life Safety (LS)	Fair damage, some permanent drift, some residual strength and stiffness left, damage to partition, the building may be beyond economical repair
Collapse Prevention (CP)	Severe damage, large displacement, little residual stiffness, and strength but loading bearing column and wall function, the building is close to collapse

On the other hand, nonlinear dynamic analysis is considered a tool to assess the behavior of structures due to the nonlinear response of structures under earthquake [19], [22]. Nonlinear dynamic analysis is conceptually the most reliable method to determine the nonlinear response of structures regardless of complexity [19], [23], [24]. The analysis produces accurate predictions for evaluating the inelastic behavior of the structure. Nonlinear dynamic analysis is conducted by integrating the formulation of the motion of the system step by step [22]. The variables are then updated incrementally, corresponding to time steps. The step-by-step procedure is the most effective method in nonlinear dynamic analysis.

To simulate an earthquake event, several ground motion records are selected according to ASCE 41-17 [7]. The earthquake is adjusted to the target spectral acceleration associated with the period based on the codes. As stated in FEMA-P695 [25], the ground motion records need to be normalized to peak ground velocity, and every record is scaled to match the median spectral acceleration of the maximum considered earthquake response spectrum at the period of the structure.

For fire analysis, current design methods permit the development of finite-element models that are utilized to evaluate building performance in a fire condition. The models are subjected to temperature-time curves (design fire) and then observed the behavior of the structure.

Traditional design fires use the standard fire curve published in 1978, which assumes that the temperature conditions are uniform and subjected to the whole compartment floor. The standard fire curves used in most building codes are adopted from either the ASTM E119 test [26] or the ISO 834 test [27], as shown in Fig. 3.

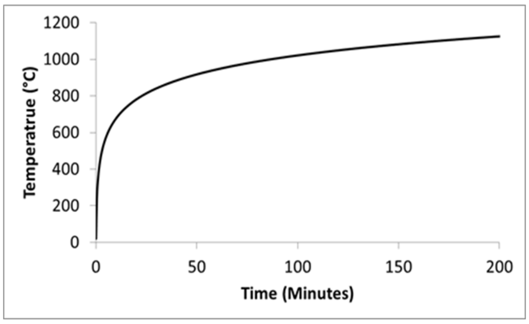

Fig. 3 ISO 834 Standard fire curve [27]

The standard fire has a slow growth rate, no temperature reduction, and is not influenced by characteristics of the building, such as fuel load, ventilation, and geometry. The standard fires do not accurately represent real fires that may burn locally but spread through the enclosure with time, creating lower temperatures and longer duration [11]. The difference in temperature, location, and time between standard fire and real fire may result in a different structure response [28], [29]. Fire modeling methods have been improved rapidly to overcome the limitation of standard fires. The methods allow for a large number of possible fires so that the designer can consider one of the most severe fires for design. The most sophisticated method recently used to predict the fire scenario is parametric and traveling fire methods.

Wickström [30] developed the background concept of the parametric fire method, which is relatively simple to use. He proposed that the compartment fire based on heat balance depended on the opening factor, thermal inertia, fire load, and gamma factor. These theoretical assumptions were validated with the experimental data developed by Magnusson and Thelandersson [31]. Then, the parametric fire was adopted and described fully in Eurocode [32].

There are two phases in the parametric fire method, the heating stage and the cooling stage, as shown in Fig. 4. In the heating phase, the gas temperature is as follows.

$$\theta_g = 20 + 1325(1 - 0.324e^{-0.2t*} - 0.204e^{-1.7t*} - 0.472e^{-19t*}) \quad (1)$$

$t*$ is obtained by the time t multiplied by a dimensionless parameter Γ defined by:

$$\Gamma = \frac{(O/b)^2}{(0.04/1160)^2} \quad (2)$$

where O is an opening factor defined by:

$$O = \frac{A_v\sqrt{h_{eq}}}{A_t} \quad (3)$$

b is the thermal absorptivity of surrounding surfaces of the compartment obtained by:

$$b = \sqrt{\rho c \lambda} \quad (4)$$

where A_v is the total area of vertical openings on all walls, heq is the weighted average of window heights on the wall, A_t is the total area of the enclosure, ρ is the density, c is the specific heat, and λ is the thermal conductivity of the boundary of the enclosure.

On the other hand, the cooling phase is generated by:

$$t^*_{max} \leq 0.5h \rightarrow \theta_g = \theta_{max} - 625(t^* - t^*_{max}) \quad (5)$$

$$0.5h < t^*_{max} < 2h \rightarrow \theta_g = \theta_{max} - 250(3 - t^*_{max})(t^* - t^*_{max}) \quad (6)$$

$$t^*_{max} \leq 2h \rightarrow \theta_g = \theta_{max} - 250(t^* - t^*_{max}) \quad (7)$$

where maximum gas temperature occurs in the heating phase at t_{*max} obtained by:

$$t^*_{max} = \left(0.2 \times 10^{-3}\, q_{t,d}/O\right)\Gamma \quad (8)$$

$$q_{t,d} = q_{f,d}A_f/A_t \quad (9)$$

where $q_{t,d}$ is the fire load density, A_f is floor area and At is the total area of enclosure.

The Parametric Fire depends on several factors, such as ventilation, fire load, and protection. Thus, the fire load density, $q_{f,d}$ can be defined by:

$$q_{f,d} = \delta_{q1}\delta_{q2}\delta_n m q_{f,k} \quad (10)$$

where δ_{q1} is the risk of fire activation, δ_{q2} is the type of occupancy, δ_n is the active fire-fighting measure, m is the combustion factor and $q_{f,k}$ is the characteristic fire load density.

However, there are limitations in the application of parametric fire. The parametric curve equation is limited for the floor area is up to 500 m^2 with a maximum of 4 m height. The thermal inertia is also restricted between 1000 and 2000 J/m^{2}s$^{1/2}$K, so the highly insulating materials could not be considered.

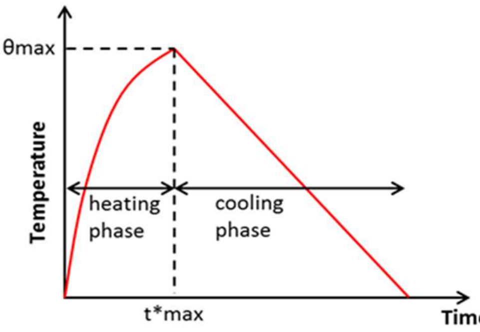

Fig. 4 Eurocode parametric fire curve [32]

The factor in the parametric fire curve may change due to an earthquake. When an earthquake occurs, there is a possibility of window breakage or damage to the building envelope and partitions after an earthquake that can increase the opening factor of the compartment. Furthermore, fire-fighting measures may be different between a fire in normal conditions and a fire after an earthquake. Fig. 5 compares the parametric fire curve before and after an earthquake developed by Suwondo et al. [11]. It can be seen that the changes in factors due to an earthquake increase both the maximum temperature and burning time.

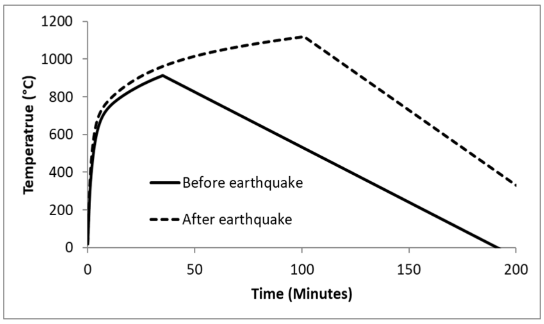

Fig. 5 Comparison of the parametric fire curve before and after an earthquake [11]

To overcome the various limitation expressed above, a newly designed method called traveling fire has been introduced. The traveling fire method is developed based on an observation from an actual building fire [33], [34]. Since the fire travels across the slab, the traveling fire method distributes the effect of fire dynamics into two temperature fields: near and far. The near-field is a high temperature in the burning zone of the fire, and the latter is the cooler temperature for the rest of the compartment. Fig. 6 shows the concept of traveling fire with two temperature fields. After consuming all fuel in the location, the near-field starts traveling to the far-field temperature location.

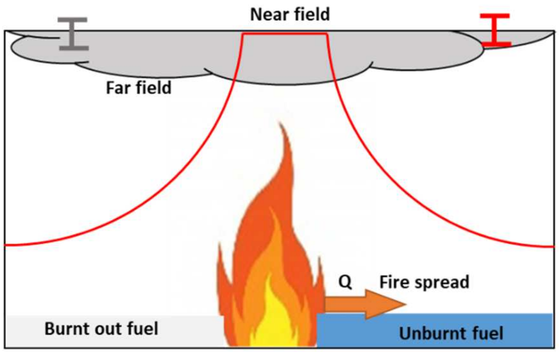

Fig. 6 The concept of travelling fire [35]

Unlike the conventional method, the concept of the travelling fire method is that the effect of all travelling fire scenarios on the structure is considered instead of the worst-case basis. It can be seen in Fig. 7 that fire burns over a certain percentage of the floor area, ranging from high temperature with short duration to low temperature with a long duration.

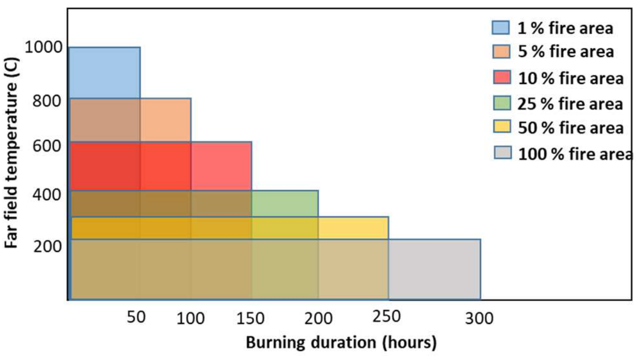

Fig. 7 Travelling fire curve [33]

B. Previous Studies

Several studies have been done on the behavior of steel frames under FFE. Della Corte et al. [14] investigated fire resistance of moment frames subjected to FFE. Two types of frames were considered. The first frame was designed to satisfy only the ultimate limit state, called ULS. The second fame was designed to satisfy the ultimate limit state and seismic serviceability design called SLS. Nonlinear dynamic analysis using several ground motion records was performed to simulate earthquake load. The thermal-mechanical analysis was then applied using the ISO 834 curve to simulate fire events. The results revealed a significant reduction of fire resistance for the ULS frame. Contrary, the reduction of fire resistance for the SLS frame is relatively smaller. It is also worth noting that the residual drift due to earthquake load significantly affects the structure's fire resistance. This study shows that the design philosophy plays an important role in the performance of steel structures under FFE.

Faggiano et al. [36] adopted nonlinear pushover and thermal-mechanical analysis to study the behavior of 2D steel moment frames subjected to earthquake and fire. The study revealed that fire resistance and collapse mechanism are similar when the frame's performance does not exceed the operational performance limit during an earthquake. A small reduction in fire resistance was detected at the frames with LS and near collapse level. Faggiano and Mazzolani [37] assessed the robustness of steel frames under FFE under the performance-based design. The study presented the identification of the seismic damage state and determination of the residual bearing capacity of the seismic damaged structures under fire conditions. Yasin et al. [38] reviewed FFE hazards and the behavior of steel building structures under FFE. An analytical study of 2D unprotected steel frames under the effects of lateral seismic loads and following fire has been presented. The study showed that the lateral deformation caused by the seismic ground motion affects the FFE of the steel frames.

Similarly, Zaharia et al. [39] evaluated fire resistance rating for earthquake damage steel frames. The earthquake response was introduced by imposing residual deformation using pushover analysis. Two different frames (designed for moderate and severe seismic regions) were prepared. The structure is pushed by lateral load until the target displacement. Once the target displacement is obtained, the lateral load is released. In this stage, the residual stress and residual displacement exist since the structures respond in the plastic range. Then, a fire analysis was performed. This study applied two different fire curves, the Standard ISO 834 and the Eurocode parametric fire curves. The results showed that the frame designed with higher seismic produced a higher fire resistance rating in the case of FFE. This is due to the fact that the structures designed for higher seismic have an important reserve of resistance under a fire situation.

Behnam and Ronagh [40] presented the evaluation of 2D steel frames subjected to FFE. Pushover analysis was applied to represent the earthquake load. Then, thermal analysis was performed using the ISO834 and the natural fire model. As a benchmark, fire analysis was also performed for the frame without damage. The results showed that the earthquake-damaged frames have lower fire resistance than the frame without damage.

Furthermore, Memari et al. [41] presented finite element simulation to study the performance of moment resisting frames with reduced beam section (RBS) connection subjected to FFE. This study selected low-, medium- and high-rise steel frames. The Eurocode parametric fire curve was adopted in the fire analysis. The material was assumed as elastic-perfectly plastic. This study considers thermal, deformational, and temperature-dependent mechanical properties. The frames were subjected to ground motion records to simulate the earthquake using dynamic analysis. The frames were considered fireproofed, and the fire was applied in the RBS connection considering that fireproofing was damaged during an earthquake. Although the Parametric fire curve was used in the fire analysis, changes in factors due to an earthquake that may affect the curve were not taken into account. The evaluation is performed on the global and local responses. The global response is evaluated based on the inter-storey drift ratio (IDR). On the other hand, the local response is investigated by highlighting the axial force-moment interaction of the structural elements. The results show that in terms of inter-storey drift ratio, post-earthquake fires result in smaller IDR than the earthquake itself. This indicates that the global performance of the building is not affected by the post-earthquake fire and the potential for systems collapse does not appear to be imminent because of these applied post-earthquake fires. The scenarios which result in asymmetric heating of the frame may give rise to excessive P-delta effects, leading to the possibility of collapse. Moreover, a localized event, such as the failure of a connection, which is not investigated in this study, may trigger a disproportionate collapse.

III. Result and Discussion

It was observed that there are four stages in the studies of steel behavior subjected to FFE. Firstly, seismically designed moment resisting frames (MRF), which can be either unprotected or fireproofed, were prepared. The considered frames were then subjected to an earthquake. The earthquake performance of the structures can be evaluated using pushover analysis or nonlinear dynamic analysis.

In the next step, the fire load is subjected to the structure. The temperature-time curves have been developed to assess the performance of the structure during a fire. The ISO 834 standard fire, parametric fire, and traveling fire were used in the previous studies. Finally, the evaluation can be conducted to determine whether the structure is adequate. It can be presented by the fire resistance rating or global and local response. Fire resistance rating is defined as a period in which the integrity of a member is maintained to resist the applied load during a fire. Resvani and Ronagh [42] adopted two measures to determine structural failure. Firstly, the failure is defined as buckling columns which can be identified while the vertical displacement of the top of the column suddenly decreases as temperature increases. Secondly, the failure is defined by the midspan deflection of the beam span. The deflection limit is L/200, in which L is the beam span.

On the other hand, according to ASCE 41-17 [7], the global response of the structure can be assessed in terms of Inter-storey Drift Ratio (IDR) limit. To evaluate local response, the axial force-bending moment interaction can be applied to each structural element.

Fig. 8 summarizes the methodologies that have been used in the previous studies. It shows that the majority of the studies performed pushover analysis to represent the earthquake load. The application of pushover analysis has become popular worldwide because it is a relatively simple approach to predict the nonlinear response of the structure. However, no dynamic effects were considered in this method. On the other hand, nonlinear dynamic analysis can capture all possibilities including dynamic effects.

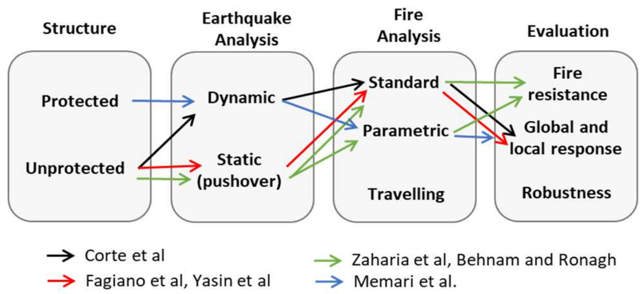

Fig. 8 Previous studies methodology[14], [36], [38], [39], [43], [44]

When performing thermal analysis, the majority of studies have adopted the standard fire (ISO834) and the parametric fire (Eurocode) instead of the traveling fire. The standard fires do not accurately represent real fire, and parametric fire has several limitations, as mentioned above. A recent study [45]–[47] found that traveling fires have a more severe impact on the behavior of the structure than parametric fire.

As previously discussed, each analysis has advantages and disadvantages. The author proposed the most appropriate method based on the previous study above, as shown in Fig. 9, to analyze earthquake damage steel in fire. This method has been developed by considering the advantages and disadvantages of each numerical analysis method.

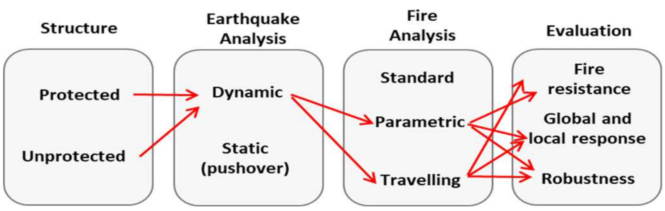

Fig. 9 The proposed methodology

In terms of the generic building, the evaluation can be performed for both protected and unprotected steel structures. Most of the steel structures were assumed unprotected, which is very rare in practice. Fire insulation is commonly applied to steel structures. However, the role of fire insulation can be compromised if it becomes detached from the steel member. Recent studies [48], [49] have shown that fire insulation can delaminate under static and dynamic loading. Fire insulation delamination in steel structures has a significant effect on the reduction of failure time. Therefore, it is essential to quantify the protection delamination in assessing a building subjected to fire following an earthquake.

Moreover, the previous studies mainly focused on the 2D Frame in which the presence of the concrete slabs was neglected. This is in line with the findings from Quiel and Garlock [50], who argued that the concrete slab could be disregarded in the 2D stress analysis of frames. By contrast, Suwondo [51] recommended that designers should consider the integrity of the concrete slab using various traveling fires. Furthermore, the actual behavior will be different from the bare steel frame since floor slabs are joined to beams as composite actions [28], [52]–[56]. The composite slab may cause the failure of the beam bottom flange in the beam to column connections which occurred in the Kobe earthquake. Therefore, the full 3D model, including the presence of concrete slab should be considered in the analysis of post-earthquake fire.

To simulate an earthquake event, performing a nonlinear dynamic analysis is suggested. Although nonlinear dynamic analysis is complex and requires ground motion data, it can be performed with the aid of advanced computational tools. A series of artificial ground motions can be used in the analysis as suggested by applicable codes such as ASCE 41-17 [7]. Thus, realistic damage due to an earthquake can be obtained as an initial condition for post-earthquake fire analysis. Nonlinear pushover analysis may be used for simple building when the dynamic effect may not significantly affect the structural behavior since it is relatively easy and fast.

Two fire curves, parametric and travelling fire, can be adopted for fire analysis. Both fire curves consider heating and cooling phases during a fire event which are not considered in the standard fire curve. Suwondo [46] found that the cooling phase may be critical to the performance of the building during a fire. It is understood that the traveling fire scenario is more realistic compared to that of parametric fire. However, fire spread is unlikely to occur, particularly for small compartments, so the traveling fire curve in the analysis may not be effective. Therefore, parametric and traveling fire analysis applications depend on the fire compartment size.

At the end of the analysis, the evaluation is carried out based on the objective of the analysis. Fire resistance is commonly used for evaluation. Fire resistance can be defined as a period in which a heated member element is maintained to sustain the applied load. It should be noted that the fire resistance of the element does not represent the failure of the building, and this is because the load previously sustained by the heated element can be transferred to the adjacent element.

On the other hand, robustness analysis is required to investigate the progressive collapse of the whole building. It is worth noting that the structural robustness evaluation, including steel connection, has not so far been considered in the analysis. Initial connection failure may lead to the collapse of a large part of the structure or even the whole structure. Therefore, the performance of steel connections has become a vital subject in the FFE.

IV. Conclusion

The historical records have demonstrated that FFE is a major threat to steel structures. The study aims to showcase, theoretically and practically, the numerical analysis methods used in the previous studies. A series of analyses are required to investigate the behavior of the structure under FFE. The main aspect is to determine earthquake damage which can be schematized as the combination of geometrical damage and mechanical damage.

For seismic analysis, nonlinear dynamic analysis is a better method than pushover analysis in terms of accuracy.

Nonlinear dynamic analysis can capture all possibilities, including types of nonlinearity and dynamic effects, using advanced computational tools.

For fire analysis, it is suggested to adopt the parametric and traveling fire curves in which both the heating and cooling phases are considered. It is worth noting that several assumptions must be considered to determine the parametric fire. An opening factor and fire-fighting measures may be different between normal conditions and after an earthquake.

This study has presented a review of relevant research on FFE analysis of steel structures. It was found that some aspects have not been covered yet in the previous studies. Most of the studies considered 2D plane frame behavior, which is not appropriate, particularly at elevated temperatures. This is because the slab contribution significantly affects the frame's behavior during a fire.

In addition, it is understood that fire insulation plays an important role in structural behavior during a fire. Previous studies have focused on unprotected steel frame, and there is yet a lack of detailed research on the influence of fire insulation delamination that may occur due to an earthquake. Therefore, further investigation is required to improve understanding the behavior of earthquake-damaged steel frame structures subjected to post-earthquake fire.

This study has achieved its objective of improving the understanding of the behavior of steel frames subject to FFE. However, it is believed that this study has certain limitations. Further investigation based on statistical data is needed to determine parameters in developing the Parametric Fire curve after an earthquake. This is because of uncertainty in several factors such as fire load, fire protection system, opening factor etc.

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REFERENCES

[1] C. Scawthorn, J. Eidinger, and A. Schiff, "Fire following earthquake," *Tech. Counc. Lifeline Earthq. Eng. Monogr. No. 26. Am. Soc. Civ. Eng.*, 2005.

[2] BBC News, "Lombok earthquake," 2018. https://www.bbc.com/indonesia/indonesia-45237703

[3] BBC News, "California earthquake: Larger 7.1 magnitude quake hits," 2019. https://www.bbc.co.uk/news/world-us-canada-48891511

[4] N. Elhami Khorasani and M. E. M. Garlock, "Overview of fire following earthquake: historical events and community responses," *Int. J. Disaster Resil. Built Environ.*, vol. 8, no. 02, pp. 158–174, Jan. 2017, doi: 10.1108/IJDRBE-02-2015-0005.

[5] M. Memari, H. Mahmoud, and B. Ellingwood, "Stability of Steel Columns Subjected to Earthquake and Fire Loads," *J. Struct. Eng.*, vol. 144, no. 1, 2018, doi: 10.1061/(ASCE)ST.1943-541X.0001909.

[6] T. Jelinek, V. Zania, and L. Giuliani, "Post-earthquake fire resistance of steel buildings," *J. Constr. Steel Res.*, vol. 138, pp. 774–782, 2017, doi: 10.1016/j.jcsr.2017.08.021.

[7] ASCE, *ASCE 41-17: Seismic Evaluation and Retrofit of Existing Buildings*. 2017.

[8] BSN, "SNI 1726-2019: Tata cara perencanaan ketahanan gempa untuk struktur bangunan gedung dan non gedung," *Badan Stand. Nas. Indones.*, 2019.

[9] CEN, "Eurocode 3 : Design of steel structures Part 1-2: General rules — Structural fire design," *Br. Stand. Institute, London, UK*, 2005.

[10] M. Memari and H. Mahmoud, "Framework for a performance-based analysis of fires following earthquakes," *Eng. Struct.*, vol. 171, no. April, pp. 794–805, 2018, doi: 10.1016/j.engstruct.2018.05.099.

[11] R. Suwondo, M. Gillie, L. Cunningham, and C. Bailey, "Effect of earthquake damage on the behaviour of composite steel frames in fire," *Adv. Struct. Eng.*, 2018, doi: 10.1177/1369433218761138.

[12] K. A. R. V. D. Kahandawa, N. D. Domingo, K. S. Park, and S. R. Uma, "Earthquake damage estimation systems: Literature review," *Procedia Eng.*, vol. 212, pp. 622–628, 2018, doi: 10.1016/j.proeng.2018.01.080.

[13] A. Shishegaran, M. Moradi, M. A. Naghsh, B. Karami, and A. Shishegaran, "Prediction of the load-carrying capacity of reinforced concrete connections under post-earthquake fire," *J. Zhejiang Univ. A*, vol. 22, no. 6, pp. 441–466, 2021, doi: 10.1631/jzus.a2000268.

[14] G. Della Corte, R. Landolfo, and F. M. Mazzolani, "Post-earthquake fire resistance of moment resisting steel frames," *Fire Saf. J.*, vol. 38, no. 7, pp. 593–612, 2003, doi: 10.1016/S0379-7112(03)00047-X.

[15] A. Y. Rahmani, N. Bourahla, R. Bento, and M. Badaoui, "Adaptive upper-bound pushover analysis for high-rise moment steel frames," *Structures*, vol. 20, no. January, pp. 912–923, 2019, doi: 10.1016/j.istruc.2019.07.006.

[16] R. Zare Bidoki and M. Shayanfar, "An energy-based pushover-analysis with torque-effects in assessment of the structures with asymmetric plan," *Soil Dyn. Earthq. Eng.*, vol. 108, no. February, pp. 58–68, 2018, doi: 10.1016/j.soildyn.2018.02.005.

[17] L. S. Hogan, I. Giongo, K. Q. Walsh, J. M. Ingham, and D. Dizhur, "Full-scale Experimental Pushover Testing of an Existing URM Building," *Structures*, vol. 15, no. April, pp. 66–81, 2018, doi: 10.1016/j.istruc.2018.04.007.

[18] R. Suwondo, D. Mangindaan, L. Cunningham, and S. Alama, "Non-linear analysis of seismic performance of low-rise concrete buildings in Indonesia," *IOP Conf. Ser. Earth Environ. Sci.*, vol. 794, no. 1, 2021, doi: 10.1088/1755-1315/794/1/012024.

[19] M. Bhandari, S. D. Bharti, M. K. Shrimali, and T. K. Datta, "Assessment of proposed lateral load patterns in pushover analysis for base-isolated frames," *Eng. Struct.*, vol. 175, no. August, pp. 531–548, 2018, doi: 10.1016/j.engstruct.2018.08.080.

[20] R. Suwondo and S. Alama, "Seismic evaluation of reinforced concrete moment resisting frames using pushover analysis," *IOP Conf. Ser. Earth Environ. Sci.*, vol. 426, no. 1, 2020, doi: 10.1088/1755-1315/426/1/012048.

[21] D. Yahmi, T. Branci, A. Bouchaïr, and E. Fournely, "Evaluation of behaviour factors of steel moment-resisting frames using standard pushover method," *Procedia Eng.*, vol. 199, pp. 397–403, 2017, doi: 10.1016/j.proeng.2017.09.130.

[22] S. Li, Z. Zuo, C. Zhai, and L. Xie, "Comparison of static pushover and dynamic analyses using RC building shaking table experiment," *Eng. Struct.*, vol. 136, pp. 430–440, 2017, doi: 10.1016/j.engstruct.2017.01.033.

[23] A. Salihovic and N. Ademovic, "Nonlinear analysis of reinforced concrete frame under lateral load," *Coupled Syst. Mech.*, vol. 6, no. 4, pp. 523–537, 2017, doi: 10.12989/csm.2017.6.4.523.

[24] C. G. Chiorean and S. M. Buru, "Practical nonlinear inelastic analysis method of composite steel- concrete beams with partial composite action," *Eng. Struct.*, vol. 134, pp. 74–106, 2017, doi: 10.1016/j.engstruct.2016.12.017.

[25] FEMA-P695, *Quantification of building seismic performance factors*. 2009.

[26] ASTM, "ASTM E-119 Standard Test Methods for Fire Tests of Building Construction and Material," 2020.

[27] BS EN, "ISO 834-2:2019 Fire-resistance tests — Elements of building construction — Part 2: Requirements and recommendations for measuring furnace exposure on test samples," 2019.

[28] E. Rackauskaite, P. Kotsovinos, A. Jeffers, and G. Rein, "Structural analysis of multi-storey steel frames exposed to travelling fires and traditional design fires," *Eng. Struct.*, vol. 150, pp. 271–287, 2017, doi: 10.1016/j.engstruct.2017.06.055.

[29] E. Rackauskaite, P. Kotsovinos, and G. Rein, "Model parameter sensitivity and benchmarking of the explicit dynamic solver of LS-DYNA for structural analysis in case of fire," *Fire Saf. J.*, vol. 90, no. August 2016, pp. 123–138, 2017, doi: 10.1016/j.firesaf.2017.03.002.

[30] U. Wickström, "Temperature Calculation of Insulated Steel Columns Exposed to Natural Fire," *Fire Saf. J.*, vol. 4, no. 4, pp. 219–225, Jan. 1981, doi: 10.1016/0379-7112(81)90024-2.

[31] S. Magnusson and S. Thelandersson, "Temperature-time curves for the complete process of fire development – a theoretical study of wood fuels in enclosed spaces," *Acta Polytech. Scand. Stock.*, vol. 65, 1970.

[32] CEN, "Eurocode 1: Actions on structures. Part 1–2: General actions. Actions on structures exposed to fire," *Br. Stand. Institute, London, UK*, 2002.
[33] X. Dai, S. Welch, and A. Usmani, "A critical review of 'travelling fire' scenarios for performance-based structural engineering," *Fire Saf. J.*, vol. 91, pp. 568–578, 2017, doi: https://doi.org/10.1016/j.firesaf.2017.04.001.
[34] N. Alam *et al.*, "Large scale travelling fire tests with open ventilation conditions and their effect on the surrounding steel structure– The second fire test," *J. Constr. Steel Res.*, vol. 188, p. 107032, 2022, doi: https://doi.org/10.1016/j.jcsr.2021.107032.
[35] J. Stern-Gottfried and G. Rein, "Travelling fires for structural design-Part II: Design methodology," *Fire Saf. J.*, vol. 54, pp. 96–112, 2012, doi: 10.1016/j.firesaf.2012.06.011.
[36] B. Faggiano, M. Esposto, and F. M. Mazzolani, "Risk Assessment of Steel Structures Under Fire," *14th World Conf. Earthq. Eng. 2008, Beijing, China*, 2008.
[37] B. Faggiano and F. M. Mazzolani, "Fire after earthquake robustness evaluation and design: Application to steel structures," *Steel Constr.*, vol. 4, no. 3, pp. 183–187, 2011, doi: 10.1002/stco.201110025.
[38] H. Yassin, F. Iqbal, a Bagchi, and V. K. R. Kodur, "Assessment of Post-Earthquake Fire Performance of Steel-Frame Buildings," *14th World Conf. Earthq. Eng.*, 2008.
[39] R. Zaharia and D. Pintea, "Fire after earthquake analysis of steel moment resisting frames," *Int. J. Steel Struct.*, vol. 9, no. 4, pp. 275–284, 2009, doi: 10.1007/BF03249501.
[40] B. Behnam, P. J. Lim, and H. R. Ronagh, "Plastic Hinge Relocation in Reinforced Concrete Frames as a Method of Improving Post-earthquake Fire Resistance," *Structures*, vol. 2, pp. 21–31, 2015, doi: 10.1016/j.istruc.2014.12.003.
[41] M. Memari and H. Mahmoud, "Performance of steel moment resisting frames with RBS connections under fire loading," *Eng. Struct.*, vol. 75, pp. 126–138, 2014, doi: 10.1016/j.engstruct.2014.05.040.
[42] F. H. Rezvani and H. R. Ronagh, "Structural response of a MRF exposed to travelling fire," *Proc. Inst. Civ. Eng. - Struct. Build.*, vol. 168, no. 9, pp. 619–635, 2015, doi: 10.1680/jstbu.14.00046.
[43] B. Behnam and H. R. Ronagh, "Post-Earthquake Fire performance-based behavior of unprotected moment resisting 2D steel frames," *KSCE J. Civ. Eng.*, vol. 19, no. 1, pp. 274–284, 2015, doi: 10.1007/s12205-012-0527-7.
[44] M. Memari, H. Mahmoud, and B. Ellingwood, "Post-earthquake fire performance of moment resisting frames with reduced beam section connections," *J. Constr. Steel Res.*, vol. 103, pp. 215–229, 2014, doi: 10.1016/j.jcsr.2014.09.008.
[45] J. Martinez and A. E. Jeffers, "Structural response of steel-concrete composite floor systems under traveling fires," *J. Constr. Steel Res.*, vol. 186, p. 106926, 2021, doi: https://doi.org/10.1016/j.jcsr.2021.106926.
[46] R. Suwondo, L. Cunningham, M. Gillie, and C. Bailey, "Progressive collapse analysis of composite steel frames subject to fire following earthquake," *Fire Saf. J.*, vol. 103, no. December 2018, pp. 49–58, 2019, doi: 10.1016/j.firesaf.2018.12.007.
[47] J. Jiang, Y. Lu, X. Dai, G.-Q. Li, W. Chen, and J. Ye, "Disproportionate collapse of steel-framed gravity buildings under travelling fires," *Eng. Struct.*, vol. 245, p. 112799, 2021, doi: https://doi.org/10.1016/j.engstruct.2021.112799.
[48] V. Kodur and A. Arablouei, "Effective properties of spray-applied fire-resistive material for resistance to cracking and delamination from steel structures," *Constr. Build. Mater.*, vol. 84, pp. 367–376, 2015, doi: 10.1016/j.conbuildmat.2015.03.022.
[49] N. L. Braxtan and S. P. Pessiki, "Postearthquake Fire Performance of Sprayed Fire-Resistive Material on Steel Moment Frames," *J. Struct. Eng.*, vol. 137, no. 9, pp. 946–953, 2011, doi: 10.1061/(ASCE)ST.1943-541X.0000441.
[50] S. Quiel and M. Garlock, "Modeling High-Rise Steel Framed Buildings under Fire," in *Structures Congress 2008*, American Society of Civil Engineers, 2008, pp. 1–10.
[51] R. Suwondo, L. Cunningham, M. Gillie, and C. Bailey, "Analysis of the robustness of a steel frame structure with composite floors subject to multiple fire scenarios," *Adv. Struct. Eng.*, no. 9, 2021, doi: 10.1177/1369433221992494.
[52] B. Jiang, G.-Q. Li, L. Li, and B. a. Izzuddin, "Experimental Studies on Progressive Collapse Resistance of Steel Moment Frames under Localized Furnace Loading," *J. Struct. Eng.*, vol. 144, no. 2, pp. 1–10, 2017, doi: 10.1061/(ASCE)ST.1943-541X.0001947.
[53] J. Jiang and G. Li, "Progressive collapse analysis of 3D steel frames with concrete slabs exposed to localized fire," *Eng. Struct.*, vol. 149, pp. 21–34, 2017, doi: 10.1016/j.engstruct.2016.07.041.
[54] J. Jiang and G.-Q. Li, "Disproportionate collapse of 3D steel-framed structures exposed to various compartment fires," *J. Constr. Steel Res.*, vol. 138, pp. 594–607, 2017, doi: https://doi.org/10.1016/j.jcsr.2017.08.007.
[55] R. Suwondo, L. Cunningham, M. Gillie, M. Suangga, and I. Hidayat, "Model Parameter Sensitivity for Structural Analysis of Composite Slab Structures in Fire," *Int. J. Technol.*, vol. 12, no. April, pp. 339–348, 2021, doi: 10.14716/ijtech.v12i2.3919.
[56] J. Jiang and G.-Q. Li, "Parameters affecting tensile membrane action of reinforced concrete floors subjected to elevated temperatures," *Fire Saf. J.*, vol. 96, no. October 2017, pp. 59–73, 2018, doi: 10.1016/j.firesaf.2017.12.006.