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Flexural Strengthening of Fire Damaged Reinforced Concrete Structural Member

Mahesh D. Gaikwad [1,*], Suvir Singh [2]

[1] Academy of Scientific and Innovative Research, Research Scholar, CSIR- Central Building Research Institute Roorkee, Roorkee 247 667, India
[2] Fire Research Laboratory, Chief Scientist, CSIR- Central Building Research Institute Roorkee, Roorkee 247 667, India

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Abstract

Fire causes extreme damage leading to huge loss of life and property. An extensive damage also occurs to structural members when exposed to fire. Therefore, after fire structural members need to be strengthened to restore their load carrying capacity to withstand the future service loads. The selection of strengthening technique depends on the residual capacity of fire exposed structures and required ultimate load carrying capacity. Further if strengthen structure is exposed to fire in future then strengthening techniques should also have good fire performance and remain intact with the base concrete. In this paper, efforts have been made to study and understand the effectiveness of fire damaged reinforced concrete jacked flexural member. Limited information is available on the flexural performance of fire damaged strengthen members with reinforced concrete jacket and concrete jacket. This work presents a simple and efficient technique to strengthen reinforced concrete beam in flexure using traditional materials. This paper discusses in detail the strengthening strategy program of two-hour fire exposed beam to restore its original load carrying capacity. The reinforced concrete beam is simulated in finite element-based software to compute the thermal profile, which used to calculate residual load capacity. The obtained result of thermal distribution and residual moment capacity are compared with proposed literature models and design code equations. The performance of fire damaged and strengthen beams are studied through moment-curvature relationship load-deflection behaviour. This paper provides a simplified analytical approach to determine the residual capacity of fire damaged beams along with that expected improvements in their capacity after strengthen by reinforced concrete and concrete jacket.

Keywords: Fire exposure, strengthening, flexure, residual strength, reinforced concrete jacketing, thermal profile, temperature, RC beam.

1. Introduction

Strengthening is essential to recover the load carrying capacity of damaged structure. It is required to fulfil the future upcoming loading demand and improve the performance of structure under static and dynamic loading. The major concern of strengthening is to avoid the complete demolishing and rebuilding of structure, adding to economic benefits. Strengthening of reinforced concrete structure can be done by various methods such as steel plate jacketing, fibre reinforced concrete jacketing, reinforced concrete jacketing and recently jacketing by wrapping fibre reinforced carbon strips to the concrete surface. But particularly reinforced concrete and concrete jacketing are more frequently adopted to repair and strengthen the existing reinforced concrete structural member. The presented study is carried out by using traditional strengthening technique which is having good fire resistance due to low conductivity of material. The procedure of jacketing involves attaching new concrete layers to the existing structural member and reinforcing the jacket with a properly designed amount of reinforcement [1]. The externally added concrete jacket is usually reinforced with longitudinal steel bars, stirrups, steel wire mesh or various kinds of fibrous materials. The structural performance of jacketed member is basically depending on enlargement of the transverse cross section and provided percent of reinforcement.

2. Jacketing of Flexural Members

Several studies were conducted to understand the influence of reinforced concrete jacketing on the flexural behaviour of structural members [2,3]. Along with that the effect of various jacket parameters on strengthened beams also studied [4]. The improvement of flexural strength is required to accommodate the certain design requirements whensubjected to sagging and hogging moments. Nowadays, jacketing of RC beams presents itself as a prominent method among the various strengthening techniques that are widely adopted by structural engineers worldwide. With this technique, a significant improvement in flexural strength and stiffness is observed. Further as a consequence of increase in the initial section as well as the addition of longitudinal reinforcement leads to reduce the displacements and deformations [5-6]. One experimentally tested strengthen flexural member is analytically modelled and validated with observed test data. The intent of validation is to ensure and workout the reliability of applied analytical methodology. Hussein et. al [6] conducted an experimental test to examine the effect of reinforced concrete jacketing on the flexural member by studying the

*Corresponding author. Tel: +918006479393; E-mail address: mahesh@cbri.res.in

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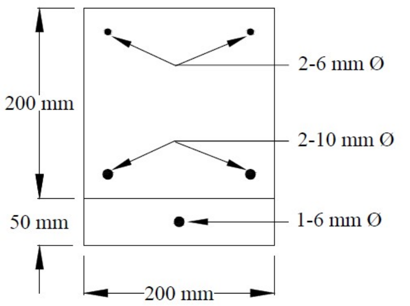
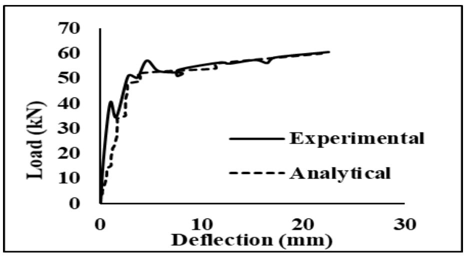
load-deflection behaviour, ultimate load and ductility. The reinforced concrete beam of length 1.5 m long subjected to two-point loads is strengthened without any damage inducing to it. The cross-sectional dimension is shown in Fig.1(a). The reinforcement configuration includes longitudinal rebar which is having two number of 6 mm diameter rebars at top and two number of 10 mm diameter rebar at bottom. The 50 mm concrete jacket was cast to the soffit of the beam and also provided one 6 mm rebar in concrete jacket. The material properties having concrete compressive strength is 25 MPa. The Tensile yield strength and ultimate strength of 6mm dimeter bar is 437 MPa and 631 MPa and for 10 mm bar is 360 MPa and 631 MPa respectively.

The RC jacketed section shown in Fig.1(a) is analytically modelled to generate the load deflection relationship. The analytically observed results plotted in Fig.1(b), showed analytical results having good consistency with experimentally obtained data. For cross-section analysis, a classic approach based on the use of discretization into layers at constant strain of the cross-section is made. The whole concrete section including newly jacketed portion is divided in strips of equal thickness. Alhadid and Youssef [7-8] proposed a calculation algorithm to determine the sectional relationships in jacketed beams. The equilibrium conditions are satisfied by calculating compression and tension forces of old section and newly added layer. The generated forces are accounted through applying material models on linear strain variation along the depth of section. The incompatibility between old section and newly added layer is not considered during analysis.



(a)



(b)

Fig.1 Comparison of observed results: a) Cross-sectional details of jacketed beam b) Load-deflection behaviour of jacketed beam [6]

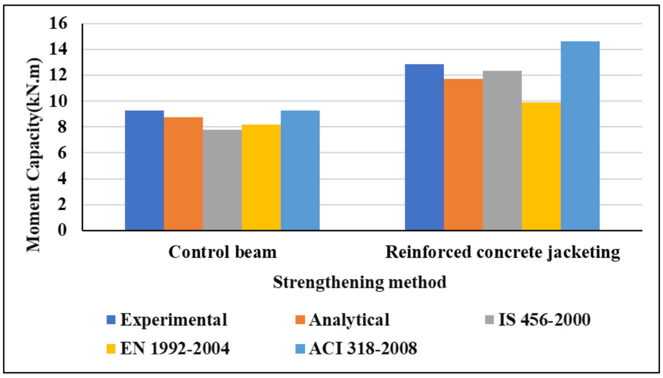


Fig.2 Comparison of moment capacity with different design code equations

The moment capacity obtained analytically for control beam as well as jacketed beam is compared with various design code equations such as ACI 318[**9**], EN 1992-2004[**10**] and IS 456-2000[**11**]. The moment capacity of experimentally tested specimen is calculated from load deflection curve as shown in Fig.1(b). From the data plotted in Fig.2 it is observed that the analytically obtained moment capacity of control and jacketed specimen are 8.77 kNm and 11.7 kNm respectively. In control specimen the analytically obtained results showed slight variation with experimental and design code equation data. In case of reinforced concrete jacketed specimen, the moment capacity obtained from experiment and IS 456-2000 are 12.82 kNm and 12.35 kNm respectively, which are more approximate with analytical moment capacity achieved as 11.70 kNm. Analytical results having very good consistency with experimentally as well as design code equations. On the basis of compared results the analytically methodology proved reliable to evaluate the load and moment capacity of jacketed beam.

3. Strengthening Strategy for Fire Damaged Beams

Flexural strengthening of reinforced concrete beam is investigated by proposed analytical program. The strength assessment strategy of fire damaged jacketed beam is formulated as shown in flowchart (Fig.5). The strengthening of fire exposed RC beam is carried out through concrete and reinforced concrete jacketing. The reinforced concrete beam exposed to 2-hour standard fire exposure is strengthened with concrete and reinforced concrete jacket. The two-hour fire exposure duration is adopted as the National Building Code (NBC) recommends that in most of the occupancies the structural element should have two-hour fire resistance rating[12]. The beam having 2.5 m length long with sectional dimensions as 150 mm width and 250 mm depth is subjected to two-point loads. The details of cross section and reinforcement configuration is shown in Fig.4(a). The compressive strength of concrete is considered as 50 MPa and tensile yield strength of steel is 500 MPa, for fire damaged section as well as jacketed section. The high strength concrete grade is adopted to target the reduction of structural member sizes in high rise structure. The 6 mm lateral ties with spacing 200 mm are used to confine the longitudinal reinforcement.

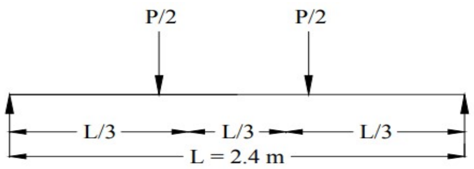


Fig.3 Loading details of jacketed RC beam

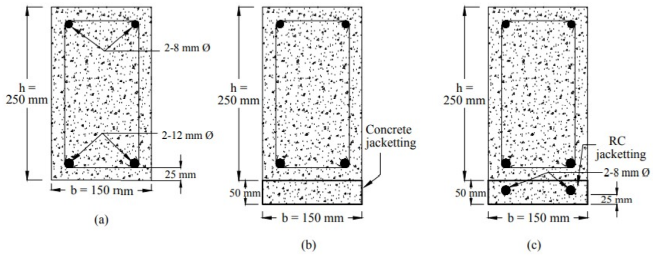


Fig.4 Sectional details of: a) Unstrengthen member b) Concrete jacketing at bottom soffit c) RC jacketing at bottom soffit

The first specimen is considered as the 'reference' case, while the second specimen strengthened by 50 mm thick concrete jacket at bottom soffit (Fig.4(b)). The third specimen is strengthened by RC jacket at bottom soffit with two number of 8 mm rebars (Fig.4(c)). The reinforcement is provided based on the load carrying capacity required to restore the section and also to study the effect of less amount of reinforcement on strength and stiffness. The additional reinforcements are tied with 6 mm dimeter rebar with same spacing as shown in fig.4(c). The residual strength assessment of fire damaged beam is discussed and then proceed to jacketing. The procedure presented in literature is followed to evaluate the residual load carrying capacity of fire damaged beam. The residual capacity of fire exposed beam is computed by estimating the temperature distribution across the cross section of beam and corresponding change in material properties [13-14]. The jacketing technique involves the proposed three main stages of analysis for predicting the maximum moment capacity as presented in Flowchart (Fig.5). It includes the thermal analysis for maximum duration of fire, post fire capacity for maximum experienced temperature and finally, jacketing of fire exposed member to restore the capacity. First step of computation of the residual capacity is to calculate the maximum temperature of fire exposed structural member [13]. The beam section simulated in finite element-based software to evaluate the temperature distribution. This thermal analysis program is discussed in detail.

3.1 Thermal analysis

A transient heat transfer analysis of beam section subjected to two-hour standard fire is carried out in finite element-based software ABAQUS. The required temperature dependent material properties are taken from EN 1992[10]. To account the effect of fire, the ISO 834[15] time-temperature curve (Fig.6) is applied on three faces.

The thermal profile of beam is assumed same as along the length. The reinforcing steel rebar is not specifically modelled in the thermal analysis because it does not significantly influence the temperature distribution in the beam cross section [16].

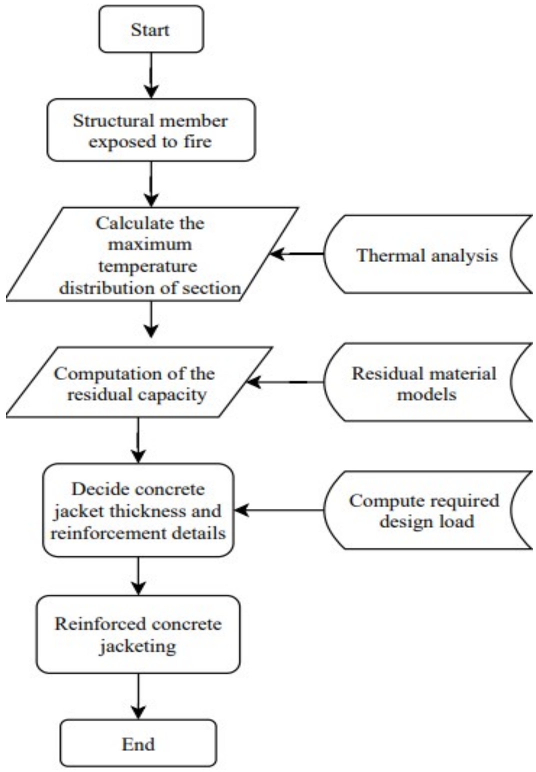


Fig.5 Flowchart of Jacketing of fire damaged beam

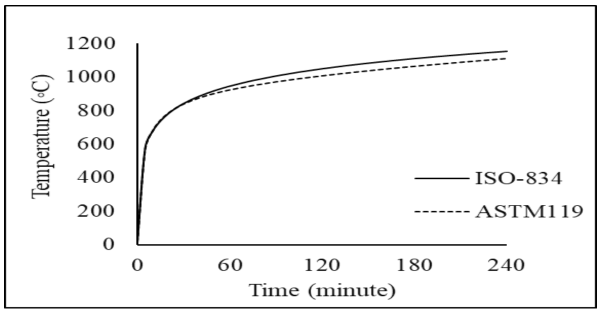


Fig.6. Standard time-temperature curve

The temperature of steel is extracted from location of rebar which represent the particular nodal temperature. The simulated section thermal profile is shown in Fig.7.The figure shows the various ranges of temperature contour for 1-hour and 2-hour fire duration.

There are some proposed empirical models, graphical methods and prescriptive approaches to estimate the temperature distribution (EN 1992-2004 [10], Wickstrom Method [17], Zhenhai Gau [18]). The variation of temperature on rebar location obtained from various

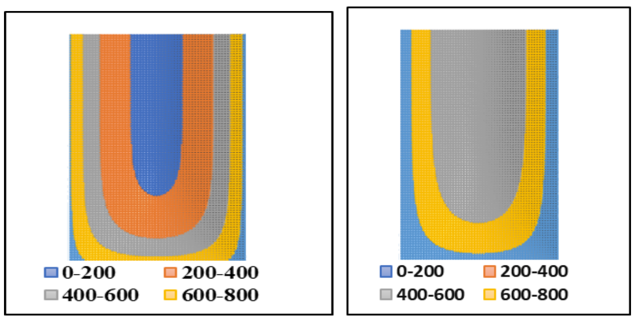
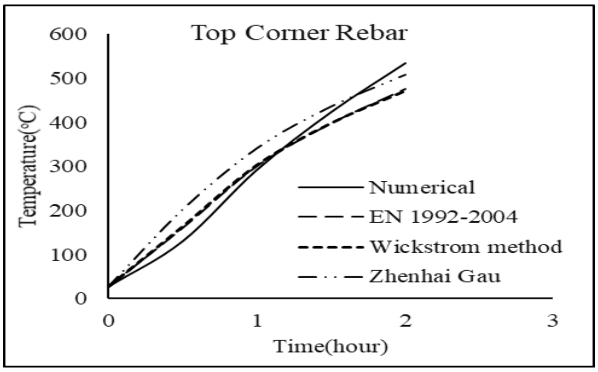
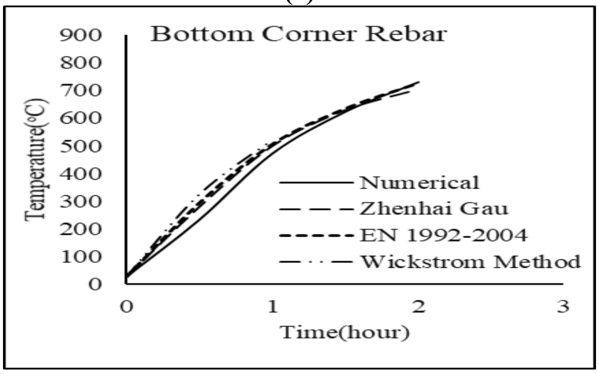


Fig.7 Thermal profile of fire exposed beam:(a) 60 min (b) 120 min



(a)



(b)

Fig.8. Comparison of time Vs temperature on rebar location: a) Top corner rebar b) Bottom corner rebar

approaches with the results obtained from simulation are plotted as shown in Fig.8(a&b). The obtained thermal variation is compared with available literature models.

From Fig.8(a&b), obtained plot shows good agreement between the compared result. From the plot it is clear that the consistency between temperature is maintained till two hours at bottom rebar location and slight variation at top rebar location.

3.2 Residual Capacity of Fire Exposed RC Beams

In case of fire exposure, a reinforced concrete structure might experience significant structural damage resulting from loss of concrete and rebar strength owing to high temperature. This may lead to very rapid degradation of strength and stiffness [19-20]. But after complete cooling down of fire exposed structure, it recovers the load carrying capacity to certain extent depending on the degree of damage. It is required to determine the residual strength after the fire exposure for carrying out strengthening and repair. The strengthening strategy should be defined on the basis of amount of strength required to be restore the original load capacity. The amount of strength to restore the original capacity is evaluated based on available load capacity which is residual load capacity after the cool down of structure. Once residual strength is computed then the strengthening strategy defined on the basis required required design load capacity. The methodology proposed in literature to calculate the residual capacity of beam is followed to execute flexural behaviour of fire damaged member [13-14]. Several studies are available in literature which are discussing the residual mechanical properties of

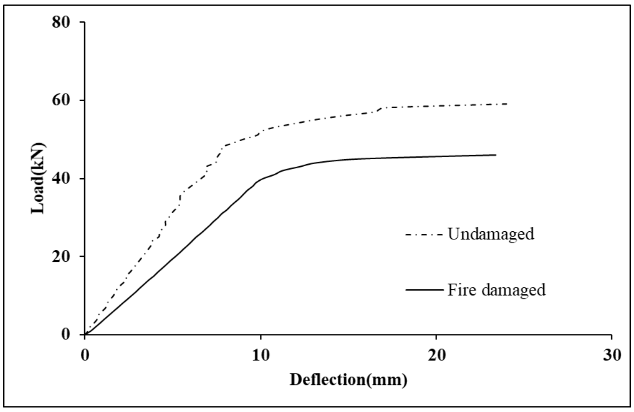


Fig.9 Load deflection relationship of fire exposed beam

concrete and reinforcing steel after being exposed to elevated temperatures and brought back to room temperature. The residual strength of the reinforcing steel should be computed considering experience maximum rebar temperature. The load deflection of undamaged and fire damaged RC beam are shown in Fig.8. The residual load deflection curve generated through layer approach proposed by El-Fitiany, S. F., and Maged A. Youssef [21]. The required residual properties of concrete and steel are calculated from Chang et al. [22] and Tao Z Chang et al. [23] respectively. From the Fig.9 it is observed that a significant damage occurred after the 2-hour fire exposure. The beam damaged due to fire is considerable loosed its strength and stiffness. The point where plastic deformation start that is yield load capacity shown considerable drop after the induced damage. The yield load capacity of undamaged beam is falls down from 49.07 kN to 39.20 kN after the fire damage. In addition to that deflection with respect yield load capacity is significantly increased from 8.30 mm to 10.23 mm. Therefore, the increased deflection is responsible for considerable drop of stiffness.

The evaluated residual moment capacity by analytical model [**Fig.10.**] is compared with design equation specified in codes (e.g. ACI 318 [**9**] Eurocode 1992-2004 [**10**]) to understand reliability of result predicted by analytical model, shown in **Fig.9.** The moment capacities are based on strength degradation of steel reinforcement at maximum reached temperature. The moment capacity is analytically calculated by applying internal equilibrium to generate moment curvature relationship. Where the plastic deformation starts in moment curvature relationship this moment capacity considered as yield moment which is used to compare with design code equations results. The moment curvature relationship generated through varying the neutral axis at constant strain level and calculating compressive and tension forces. The respective compressive and tension forces is calculated from residual material models of concrete and steel [22-23]. The analytically obtained moment capacity is shown in Fig.10. To account residual moment capacity from design code equation some proposed assumption is considered. According to **Kodur et al. [13]** concrete strength degradation should be ignored during residual strength calculation because of low thermal conductivity of concrete significantly reduces the temperature rise in that compression zone.

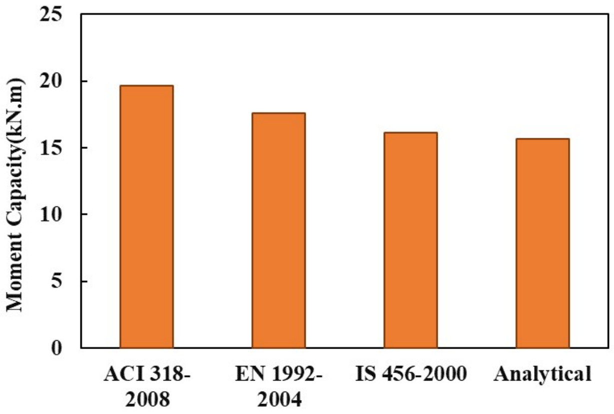


Fig.10 Comparison of residual moment capacity with different design code provision

Thus, the residual strength of concrete is assumed to be the same as that of the original concrete prior to fire exposure. The sectional dimensions in specified design equations (e.g. ACI 318 [**9**] Eurocode 1992-2004 [**10**]) are calculated by applying 500-isotherm method [**24**]. This assumption is reasonable given that the reduced cross-section of the beam to determination of flexural strength and to achieve the more approximate result. The 500°C affected area is subtracted from actual sectional dimension and newly formed dimensions are considered during calculation. The obtained results are compared with analytical model. On the basis of plotted graphs (**Fig.10**), the results obtained by analytical methodology are more consistent with design code equation. According to IS 456, EN 1992 and ACI 318 the residual moment capacity is 16.16 kNm,17.62 kNm and 19.66 kNm respectively. All values are very close to the moment calculated by analytically which is 15.68 kNm. From the obtained residual moment values makes sure that the analytical method is more appropriate to calculate the capacity of any fire exposed structural member and this methodology is proceed to calculated behavior of jacketed beam.

4. Flexural Behaviour of Jacketed Beams

Need to required determine the behaviour of fire damaged structural members strengthen by jacketing. However limited information is available on the same. Different methods can be employed to avail overall enhancement of its performance. The purpose of this study is to investigate the reduction in load carrying capacity of fire exposed flexural member and adopt suitable technique to restore its original strength. During this study flexural member is exposed to two hours of standard fire as per ISO 834. Subsequently the specimen is jacketed with 50 mm thick concrete at soffit. In another case additional reinforcement provided with concrete jacket. It was observed that providing sufficient amount of jacket thickness and present of reinforcement may definitely contribute in enhancing the performance. The investigation of jacketing is to be more effective to strengthen the flexural members [25]. The load capacity of jacketed beam is calculated through the followed methodology in previous section to find out residual load capacity. The iterative methodology is followed to account the generated forces in fire exposed and newly jacketed cross section. The stresses in compression block and steel is calculated through residual material models available in literature [22-23]. For the newly jacketed layer, the ambient temperature material models are used to calculate generated stresses in steel and concrete. The strain and stress distribution in fire damaged beam and jacketed portion is carried out by varying neutral axis at constant strain to achieve the internal equilibrium.

5. Result and Discussion

According to the explained methodology a parametric study is conducted and obtained results are discussed. In examined case of a concrete and reinforced concrete jacket it is found that the effect of jacket on the response envelope of a jacketed beam can be particularly significant on the deformation behaviour. The jacketed parameters with the percent of reinforcement and jacket thickness are varied to upgrade the flexural resistance of the beams. This helps to determine the capacity required to restore the original strength of fire exposed beam. It is noticed that the concrete jacket thickness affected on the yield and ultimate strength of the strengthened beam. This is caused due to the increase in cross sectional area. The load deflection behaviour of jacketed beam is shown in Fig.11. Addition of 50 mm concrete jacket increases the yield load capacity of fire damaged beam from 39.20 kN to 51.10 kN which is almost near to the load capacity of undamaged beam which is 49.07 kN. And after the addition of reinforcement yield capacity reaches to 83.85 kN which is almost double the load capacity of fire damaged beam. Addition of concrete jacket can successfully restore the original load carrying capacity and stiffness but in case of reinforced concrete jacket it helps to achieve the considerable higher load carrying capacity and stiffness.

From the Fig.11 it is observed that the jacketed member fully restores the flexural strength, to enhance their overall performance and improved their ductility. The reinforced concrete jacket is more effective due to additional generated forces in steel which significantly contributed in improving the load capacity.

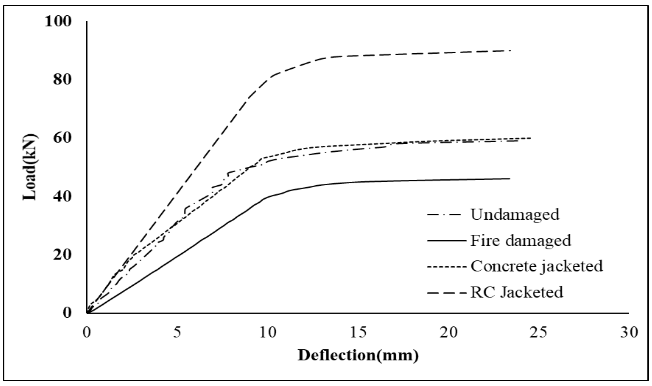


Fig.11 Load deflection behaviour of concrete jacketed beam

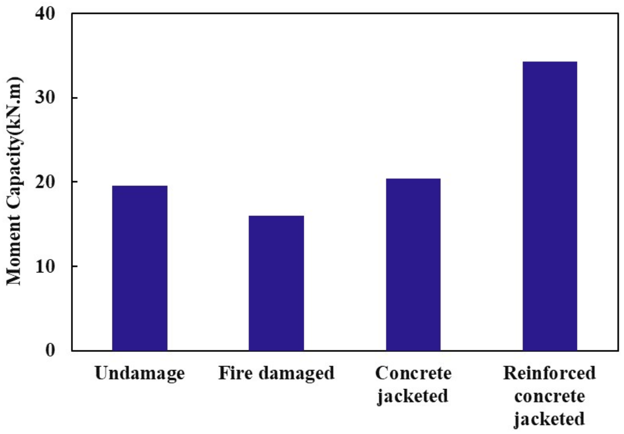


Fig.12 Maximum moment capacity of jacketed beam

A significant improvement in maximum moment capacity have been seen after the jacketing as shown in Fig.12. The moment capacity of jacketed beam is calculated according to methodology explained in previous section. After the fire exposure, the moment capacity of reinforced concrete section reduced from 19.62 kNm to 15.68 kNm. To restore this capacity a concrete and reinforced concrete jacketing is applied at bottom soffit. Concrete jacketed beam restored moment capacity from 15.68 kNm to 20.44 kNm and reaches to 33.54 kNm after addition of reinforcement. Therefore, jacketed beam is able to restore original moment capacity and achieved slightly higher initial stiffness compared to undamaged beam. On the basis of observed result it is inferred that the jacketing is an efficient technique to restore the fire damaged flexural member.

6.Conclusion

In this study a comparison of the flexural behaviour of reinforced concrete beams, strengthened with concrete and reinforced concrete jacket cast to their soffit after fire exposure is made. The proposed approach provides an estimate of the flexural capacity of fire damaged jacketed beams. The aim of strengthening strategy program is to rapid evaluation of residual strength of fire damaged member as well as restoration of original section capacity through jacketing. The analytically calculated residual capacity is consistent with current codes of practice such as ACI 318 (ACI, 2008) and Eurocode 2 (ECS, 2004). Increasing the percent of reinforcement is observed to affect the moment capacity significantly. With addition of 50 mm concrete jacket to the fire damaged beam help to enhance the maximum moment capacity by 24.84% along with addition of 0.223% steel an increase the 54.73% moment capacity. The load-deflection response of the strengthened beams indicates that the addition of 0.223% steel reinforcement jacket may significantly enhanced stiffness and load carrying capacity. The proposed strengthening strategy has proven efficient for fire exposed beam to restore original load carrying capacity.

Disclosures

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