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REPAIRED OF FIRE-DAMAGED CONCRETE-FILLED DOUBLE SKIN STEEL TUBULAR (CFDST) COLUMNS WITH FIBER REINFORCED POLYMER (FRP)

Shahiron Shahidan[1], Sharifah Salwa Mohd Zuki[1,2], Choong Kok Keong[2] and J. Jayaprakash[3]
[1]Department of Structural and Material Engineering, Faculty of Civil Engineering and Environmental Engineering, Universiti Tun Hussein Onn Malaysia, Batu Pahat, Johor, Malaysia
[2]School of Civil Engineering, Universiti Sains Malaysia, Malaysia
[3]Department of Civil Engineering, the University of Nottingham Malaysia Campus
E-Mail: shahiron@uthm.edu.my

ABSTRACT

Fire is one of the most severe natural disasters that a structure needed to face. Therefore, this study focused on post-fire repair of concrete-filled double skin steel tubular (CFDST) columns using fiber reinforced polymer (FRP). There are two types of repairing scheme that is discussed in this study, single and Hybrid FRP. Prior to being repaired, the CFDST columns were subjected to fire following ASTM E-119 standard fire curve until the temperature reached 600°C. Then, the temperature was kept constant for two different exposure times, i.e., 60 minutes, 90 minutes. Both single and Hybrid FRP enhanced the ultimate strength, secant stiffness and Ductility Index (DI) of fire-damaged CFDST columns.

Keywords: concrete-filled double skin steel tubular column, fiber reinforced polymer, post-fire repair, hybrid FRP.

INTRODUCTION

Concrete-filled double skin steel tubular (CFDST) columns were first introduced by (Shakir-Khalil 1991) in 1990's. It was first used in compression chambers in deep-sea pressure vessels. Afterward the used of CFDST columns became more popular. Most recently, it had been used as high-rise bridge piers in Japan (Zhao & Grzebieta 2002; Zhao *et al*. 2002) and in transmission tower in China (Han *et al.* 2014; Li *et al.* 2013). To date, the use of CFDST columns only can be found in outdoor construction even though it have great potential to be used as columns in high-rise building. The engineers are reluctant to use CFDST columns due to limited research on CFDST columns during and after fire. To author knowledge, there were only two experimental program on fire performance of CFDST columns (Lu, Han, *et al.* 2010; Lu, Zhao, *et al.* 2010) and none on post-fire behavior of this type of column. Post-fire behavior or residual strength of CFDST columns is important to calculate the extend of damaged done by fire. In addition, assessing the residual strength of fire damaged CFDST columns helped engineers to determine the most reasonable repair method to be used in order to salvage the building (Huo *et al.* 2009).

The most economical way of preserving fire-damaged structures is to repair the damaged members instead of demolishing and rebuilding. Nowadays, there are many available methods of repairing. From the most common method, i.e., steel jacketing to the most advance method that used composite materials as repairing materials such as fiber reinforced polymer (FRP) (Wu *et al.* 2006). However, the use of FRP as repairing materials is more favorable than steel jacketing. Among the benefits of FRP over steel jacketing are FRP is easier to install thus reducing time needed, reduce maintenance due to the nature of FRP that have resistance to corrosion, possess higher strength and lighter in weight when compared to steel and more durable (Wu *et al*. 2006).

Therefore, this study is focused on repaired of fire-damaged CFDST columns with FRP due to limited available research study on this type of columns.

EXPERIMENTAL PROGRAM

Preparation of specimens and material properties

Total of 21 specimens with outer steel tube and inner steel tube diameter of 152.4 mm and 101.6 mm, respectively were casted in upright position. The thickness is 4 mm and 2 mm for outer steel tube and inner steel tube, respectively. All specimens were 600 mm in height. Normal strength concrete with targeted strength of 30 N/mm2 was filled in the space between the two tubes. Six 100×100 mm concrete cubes were casted together with the CFDST columns in order to measure the strength of the concrete after 7 and 28 days. Three steel coupons were cut from the same outer steel tube and inner steel tube to determine the properties of the steel. Casted CFDST columns and concrete cubes were left to cure in the laboratory (air-cured). Concrete cubes were tested in accordance with BS EN 12390-3 (British Standard 2009) and tensile test for steel followed ASTM E8/E8M-11 (ASTM 2011 2011). Details of test result are shown in Table-1.

Naming convention for specimens listed in Table-1 is as follows; the first number indicates the diameter of outer steel tube. The second number C4 represent the thickness of outer steel tube and letter C indicates that the specimen is circular in shape. The third

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notation i.e., control, 60 and 90 represent the condition of the specimen. Where 'control' represent unheated and unrepaired specimens, 60 and 90 are specimens that were heated to 60 minutes and 90 minutes of fire. Lastly, the 'R' and 'RH' represent the repair condition of the specimens. R is for single layer of FRP whereas RH indicates that the specimens had been repaired with Hybrid layer of FRP.

Table-1. Details of tested specimen

Specimens	**Dia. of outer steel tube** [mm]	**Dia. of inner steel tube** [mm]	**Compressive strength of concrete after 28 days** [N/mm^2]	**Tensile stress of outer steel tube** [N/mm^2]	**Tensile stress of inner steel tube** [N/mm^2]	**Exposure time** [minutes]	**Tensile stress of FRP witness panel** [N/mm^2]
6-C4-Control	152.4	101.6	40	430	566	0	-
6-C4-60	152.4	101.6	42	430	566	60	-
6-C4-60-R	152.4	101.6	43	430	566	60	2924
6-C4-60-RH	152.4	101.6	40	430	566	60	4269
6-C4-90	152.4	101.6	38	430	566	90	-
6-C4-90-R	152.4	101.6	43	430	566	90	2924
6-C4-90-RH	152.4	101.6	35	430	566	90	4269

Heating regime

The specimens were heated in gas-fired furnace at Concrete Laboratory in Universiti Sains Malaysia (USM) one at a time as shown in

Figure-1(b). The specimens were heated in accordance of ASTM E-119 (ASTM 2010) standard fire curve until the temperature reached 600°C. After that the temperature was kept constant for either 60 minutes or 90 minutes. The exposure time of 60 and 90 minutes was chosen based on experimental program by other researchers on concrete-filled steel tubular (CFST) columns. The exposure time chosen for this research study provided fundamental guidelines for further research in this area. Once the designated exposure time was over, the furnace will automatically turn off. The lid of the furnace was open once the temperature of the furnace reached 100°C and the specimen was left to cool down inside the furnace until it was cooled enough to be touched. Only then, the specimen was lifted from the furnace and left to cool down to ambient temperature.

a) Unheated specimens

b) Specimens inside the furnace

c) Heated specimens

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d) Heated and repaired specimen

Figure-1. Condition of prepared specimens.

Temperature of the furnace, concrete and inner steel tube was measured during heating process. The furnace was equipped with 4 type-K thermocouple coated with ceramic attached to the furnace wall to measure the furnace temperature during heating process. In order to get the temperature reading of concrete, six specimens were equipped with thermocouple type-K. The thermocouple was inserted up until mid-height of the specimen into the concrete during casting process. As for the temperature measurement of inner steel tube, another thermocouple type-K was attached to the wall of inner steel tube prior to heating process. All thermocouple reading was recorded during entire heating process.

Repairing regime

12 heated specimens, six from 60 minutes and six from 90 minutes of fire exposure were repaired. From six specimens, three were repaired with single layer of Carbon FRP (CFRP) and another three were being repaired with Hybrid layer of FRP. Hybrid FRP is the combination of two different layer of FRP. The first layer is Glass FRP (GFRP) and the second layer is CFRP. As for single FRP scheme, the specimens were repaired with one layer of CFRP. The tensile stress of witness panels for CFRP and Hybrid FRP is shown in Table-1.

Prior to wrapping process, the specimens were cleaned with wire brush and the debris and dust was air blown. This is important in order to make sure better contact between FRP layer and steel surface. The FRP was wrapped in hoop direction using wet hand lay-up method with 100 mm overlap. For Hybrid FRP, the position of overlap between first and second layer was opposite to each other. This is to make sure that the specimens did not failed due to position of overlap and the strength of FRP can be fully utilized.

FRP was wrapped around the specimens using two parts MBrace Primer as the first layer and two parts MBrace Saturant as second layer. MBrace Primer was mixed together using 100 to 60 (Part A to Part B) ratios by weight. As for MBrace Saturant, it was mixed using 100 to 34 (Part A to Part B) by weight. Material properties of primer and saturant that was provided by manufacturer is shown in Table-2. Specimens were left to cure at least 7 days prior to testing.

Table-2. Materials properties of primer and saturant.

	MBrace primer	MBrace saturant
Compressive strength [N/mm^2]	73	50
Tensile strength [N/mm^2]	35	39
Flexural strength [N/mm^2]	55	62
Compressive modulus [N/mm^2]	2320	2400

Test procedure

All specimens were subjected to concentric axial compression load using Universal Testing Machine (UTM) with maximum capacity of 2000 kN until failure. Each specimen was equipped with four 10 mm electrical strain gauges located at specimen mid-height, two in vertical direction and another two in hoop direction. All strain gauges were placed perpendicular to each other and outside overlapping region for repaired specimens. Four Linear Variable Displacement Transducer (LVDTs) were placed at specimens' mid-height in order to measure any movement of tested specimens. During testing, axial load and axial displacement were measured by UTM. Test arrangement is shown in **Error! Reference source not found.**.

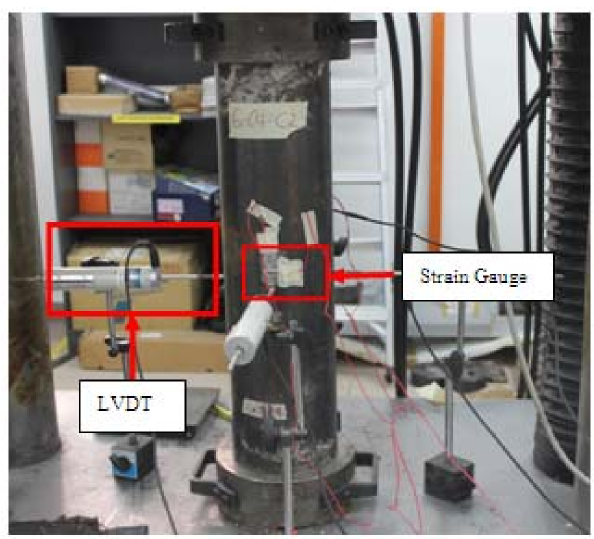

Figure-2. Test arrangement.

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During curing, small amount of shrinkage occurred at the top the specimens. The gap was filled with high strength epoxy. After the epoxy hardened, the surface was ground smooth using grinding wheel with diamond cutter. This is to prepare the surface for even distribution of stress during testing.

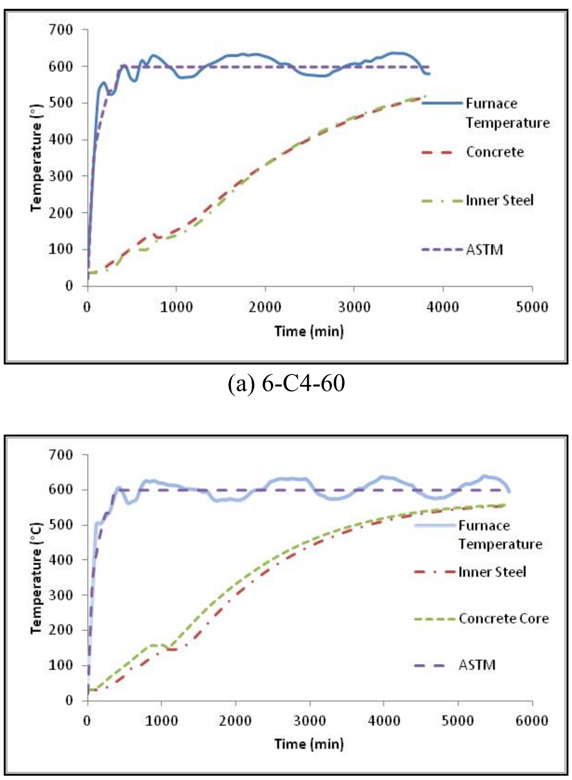
RESULT AND DISCUSSIONS

Temperature distribution

Temperature distribution of concrete and inner steel tube of specimens exposed to 60 and 90 minutes of fire is shown in

Figure-3. The furnace temperature is an average temperature from 4 thermocouples attached to the wall of the furnace. It can be seen that the curve is relatively unstable compared to ASTM E-119 standard fire curve. However, ASTM E-119 provided tolerances for furnace time-temperature curves and the curves plotted in

Figure-3 is acceptable.



(a) 6-C4-60

(b) 6-C4-90

Figure-3. Temperature distribution of (a) 6-C4-60 and (b) 6-C4-90.

From time-temperature curves of concrete shown in

Figure-3, there is relatively flat plateau when the temperature is around 100 to 200°C. The temperature seems to be almost constant with increased time. The heat during this time was used to transform water inside the concrete into vapor instead of rising the temperature of the concrete. This also being observed by other researchers (Lu, Zhao, *et al.* 2010; Lu, Han, *et al.* 2010).

Table-3. Maximum temperature of heated specimens.

Specimens	Concrete temp. [°C]	Inner steel temp. [°C]
6-C4-60	511	429
6-C4-90	557	536

The maximum temperature attained by concrete in inner steel tube was higher for specimens exposed to 90 minutes of fire (Table-3). It is expected since more heat was absorbed after being exposed for additional 30 minutes. However, it is interesting to note that even though the specimens were exposed to 90 minutes of fire, the maximum temperature of inner steel tube is lower than 538°C. According to ASTM E-119 critical temperature (temperature where steel columns started to lose its strength) of steel columns is 538°C. Therefore, CFDST columns exposed to 90 minutes of fire still able to carry most of its room temperature load since it's never reached the critical temperature stated in ASTM E-119.

As for concrete, 300°C was the onset of concrete to rapidly lose its strength. At this temperature, microcracks formed throughout the concrete due to dehydration of cement matrix. The formation of microcracks further weakened the concrete. From Table-3 it can be seen that the maximum temperature attained by concrete is way beyond 300°C. However, concrete in CFDST specimens exposed to fire acts more like protective coat to inner steel tube. Concrete already done an excellent job in protecting inner steel tube from reaching its critical temperature considering its thickness was only 25 mm.

Overall behavior

Physically, there is no changes can be seen from before (

Figure-1(a)) and after (

Figure-1(c)) the specimens being exposed to fire. Besides, the appearances of outer steel tube of specimens exposed to 60 and 90 minutes of fire were similar to each other. However, once the outer steel tube was removed, the color of concrete changed from grey to whitish grey indicating that the maximum temperature of concrete exceeding 300°C. Similar finding was reported by other researcher such as (Short *et al.* 2001). The changes in concrete color are shown in Figure-4. Furthermore, the concrete was still intact when the outer steel tube was removed for control and specimen with fire exposure of 60 minutes (Figure-4(a) (b)). However, concrete in specimen

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exposed to 90 minutes of fire was very brittle and crumble to touch (Figure-4(c)).

(a) Control specimen

(b) Specimen exposed to 60 minutes of fire

(c)Specimen exposed to 90 minutes of fire

Figure-4. Condition of concrete before and after fire exposure.

Control and heated unrepaired specimens failed by outwards local buckling of outer steel tube, crushing of concrete and local buckling of inner steel tube. Local buckling of outer steel tube, crushing of concrete and local buckling of inner steel tube is shown in Figure-6(a), Figure-4 and

Figure-5, respectively. However, local buckling of specimens exposed to 90 minutes of fire was more severe than control and specimens exposed to 60 minutes of fire. Position of local buckling of both steel tubes was found to be corresponded with crushing of concrete. Repaired fire-damaged specimens failed by explosive rupture of FRP followed by failure similar to control and unrepaired specimens either at mid-height or near to or bottom end of the specimens (Figure-6(b)). However, the explosive rupture of FRP was more violent in single layer than Hybrid FRP. Rupture of FRP occurred outside overlap region indicating that 100 mm of overlapping is sufficient to avoid premature failure due to poor installation of FRP. Prior to rupture of FRP, loud cracking sound can be heard from the tested specimen. This can be associated with FRP being fully activated.

Figure-5. Failure pattern of inner steel tube.

(a) (b)

Figure-6. Failure of tested specimens; (a) unrepaired and (b) repaired specimen.

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Table-4. Summary of tested specimens.

Specimens	Ultimate load [kN]	Disp. at ultimate load [mm]	Secant stiffness [kN/mm]	Ductility index [DI]
6-C4-Control	1402	6.98	201	1.49
6-C4-60	1292	7.22	179	1.56
6-C4-60-R	1428	11.69	122	1.17
6-C4-60-RH	1494	11.57	129	1.18
6-C4-90	1199	9.26	129	1.80
6-C4-90-R	1401	12.39	113	1.27
6-C4-90-RH	1386	10.90	127	1.18

Effect of FRP on ultimate load

This paper focused on repairing part of fire-damaged CFDST columns. Discussion of heated and unrepaired specimens can be found in (Zuki *et al*. 2015). Summary of all tested specimens are shown in Table-4.

Figure-7 and

Figure-8 shows ultimate load of fire-damaged and repaired fire-damaged specimens. It can be seen that wrapping fire-damaged CFDST specimens with FRP proved to greatly enhance the ultimate load especially for specimens exposed to 60 minutes of fire. The enhancement of ultimate load exceeded the ultimate load at room temperature. Adding another layer of FRP further enhanced the ultimate strength of specimens exposed to 60 minutes of fire. However, there is no enhancement in specimens exposed to 90 minutes of fire. Therefore, it can be concluded that wrapping more than one layer of FRP for severe fire-damaged specimens (specimen exposed to more than 60 minutes) yielded result similar to specimens repaired with single layer of FRP.

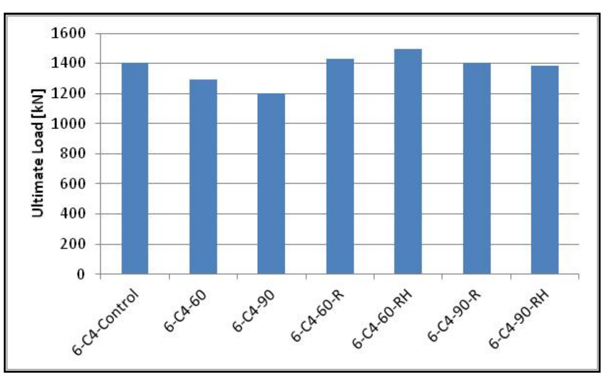

Figure-7. Ultimate load of tested specimens.

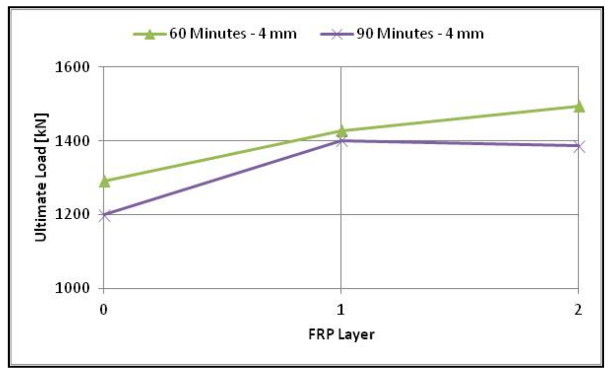

Figure-8. Effect of FRP on ultimate load.

Effect of FRP on secant stiffness

Secant stiffness is determined by dividing ultimate load of the tested specimens with it displacement at ultimate load.

Secant stiffness significantly reduced after being exposed to fire (

Figure-9). Using FRP as repaired materials failed to fully restore the secant stiffness up to its original value. However, adding another layer of FRP seems to increase the secant stiffness especially for specimens exposed to 90 minutes of fire as can be seen in

Figure-10. Secant stiffness of fire-damaged specimens can potentially be restored if more layer of FRP was added.

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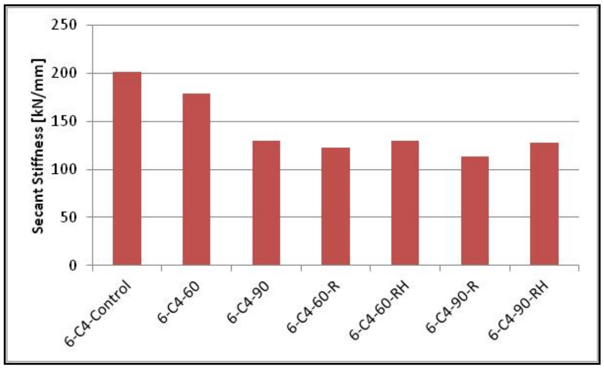

Figure-9. Secant stiffness of tested specimens.

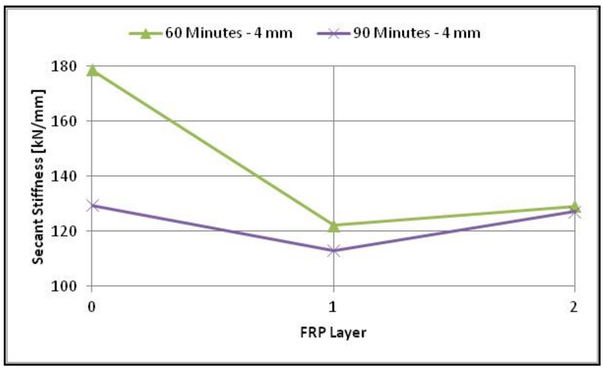

Figure-10. Effect of FRP on secant stiffness.

Effect of FRP on ductility index (DI)

Ductility Index (DI) calculated in this study was as suggested by (Liu *et al.* 2014). As shown in

Figure-11, DI of fire exposed specimens increased with increasing fire exposure time. In other words, the specimens became more ductile after fire. However, this phenomenon diminished after the specimens being repaired. Therefore, no enhancement in DI can be expected for repaired specimens due to previous increment of DI after fire. On the other hand, repairing the fire-damaged specimens with Hybrid FRP enhanced the DI of specimens exposed to 60 minutes of fire. Nevertheless, DI of specimens exposed to 90 minutes of fire remains almost similar to single FRP. This can clearly be seen in

Figure-12.

CONCLUSIONS

From above discussion, the following conclusion can be drawn:

1) The maximum temperature attained by inner steel tube was below critical temperature for steel column stated in ASMT E-119. Therefore, it is expected that the inner steel tube still able to carry most of its room temperature load during fire.
2) Control and heated unrepaired specimens failed by outward local buckling of outer steel tube, crushing of concrete and local buckling of inner steel tube. However, repaired fire-damaged specimens failed by rupture of FRP followed by similar failure pattern as control and heated unrepaired specimens.
3) Ultimate load of fire-damaged specimens were greatly enhanced after being repaired with single layer of FRP. The enhancement is more significant in specimens exposed to 60 minutes of fire. The enhancement in ultimate load exceeding its room temperature value.
4) Hybrid FRP further enhanced the ultimate load of fire-damaged specimens. However, for severe fire-damaged specimens, the effect of Hybrid FRP is similar to single FRP.
5) Secant stiffness of fire-damaged specimens increased with increased layer of FRP. It is expected that more layer of FRP can further increased the secant stiffness.
6) There is small increased in Ductility Index (DI) as FRP layer increased for specimens exposed to 60 minutes of fire. Unfortunately, DI for specimens exposed to 90 minutes of fire remains almost similar as FRP layer increased.

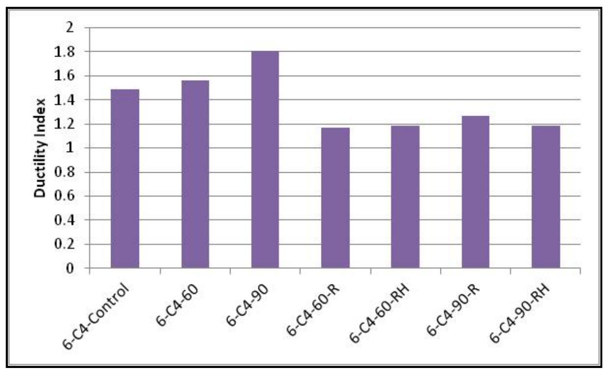

Figure-11. Ductility Index of tested specimens.

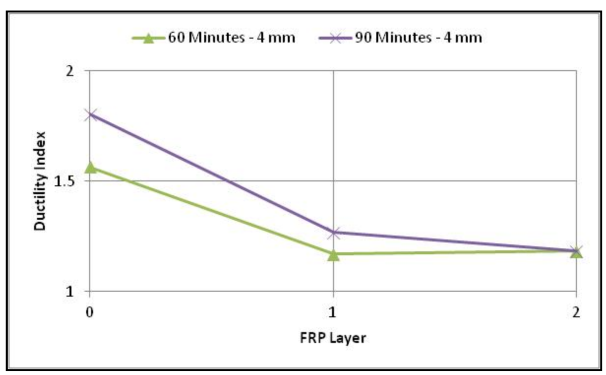

Figure-12. Effect of FRP on ductility index.

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