Using Recycled Coarse Aggregate in Reinforced Concrete Beams Strengthened for Shear by GFRP bars Using NSM Technique

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Abstract. The NSM technique began to apply as a modern technique to treat defects in structural elements and to increase the shear and flexural strength of structural elements. For this technique to be effective, a series of practical experiments were conducted to characterize the behavior of the element strengthened by the NSM technique for flexure and shear. Shear strengthening with GFRP rods is the focus of this paper for concrete beams that contain 30% coarse aggregate replacement ratio of thermstone (volumetric ratio) obtained from the rubble of demolished buildings. A total of 7 beams were loaded under four-point load test, the parameters examined were the angle of inclination and the distance between the GFRP bars, the presence and absence of stirrups and the thermstone aggregate replacement ratio. The characterization of the tested beams includes failure mode, load-deflection curves, load-strain curves of stirrups, rebars and GFRP rods and the surface concrete strain in the shear zone of beam. The results showed that the use of GFRP rods to strengthen concrete beams was effective, especially in the presence of stirrups, where the gain in shear strength was 29.6% and 22.2% when the distance between the GFRP bars was (200 and 300) mm, respectively with the presence of stirrups. While the gain in shear was just (3.7% and 11.1%) in the absence of stirrups, when the distance between the GFRP bars was (200 and 300) mm, respectively.

1. Introduction

The tremendous development of the world in the field of building and construction requires to find other materials for use in concrete rather than traditional materials. On the other hand, there was a huge increase in the amount of waste daily produced throughout the world, due to the expired lifetime of the buildings and the wars are becoming an environmental problem [1], the concept of sustainable development, which includes energy conservation, environmental protection, and maintenance of natural not renewable resources [2]. The growing consumption of natural aggregate calls to think about finding alternative sources of aggregate. The Federal Highway Administration (FHWA, 2004) estimates that two billion tons of new aggregate are produced each year in the United States, and it is expected to increase to more than 2.5 billion tons by the year 2020 [3][4]. Since aggregate represents about (60%-80%) of the volume of concrete, on the other hand, approximately 3 billion tons of waste is generated in the European Union each year [2]. In line with the increase in demand for natural aggregate and increasing construction waste production, it is necessary to search for sustainable solutions in the
recycling of the waste materials and use it for production of new concrete as recycled concrete aggregate. On the other hand, reinforced concrete beams can become deficient during their service life and need to be strengthened and repaired. Strengthening the structural element in bending may result in shear failure rather than giving the desired load carrying capacity. "Strengthening", includes modifications in a building to increase its load capacity, hardness, ductility and stability [5]. Islam et. al., [6], used the Near Surface Mounted (NSM) technique to investigate the influence of Carbon Fiber Reinforced Polymer (CFRP) on shear strength. The study included only four concrete beams, including one control beam and three model beams. The beams were strengthened in shear by using (#3) CFRP bars placed vertically at a distance of 152 mm on either side. The shear strength has been found to increase from 17% to 25%, as a result of using CFRP bars with the NSM technique. The shear span to effective depth ratio plays an important role in the effectiveness of CFRP bars used for strengthening in shear. Sharaky, et. al., [7], numerically and experimentally studied the behavior of reinforced concrete beams strengthened by near-surface mounted (NSM) (GFRP) rods with and without anchor end. The results showed that the load-carrying capacity for the reinforced concrete beams strengthened by bottom (NSM) bars is relatively high when compared to beams strengthened only with side (NSM) bars. Thamrin et al.,[8], studied nine models of reinforced concrete beams without stirrups, (800mm span with 125×250 mm cross-section). The beams were tested under a four-point loading system and were strengthened with NSM steel bars with 45° and 90° to the axis of the beam, it was found that the steel bars increase the shear capacity of the strengthened beams (three longitudinal reinforcement ratios were used 1%, 1.4%, and 2.4%). The strengthened beams with reinforcement ratios of 1% and 1.4% not failed in shear but reached the required flexural capacity. Tang and Lo [9], considered eight beams that were tested under a four-point loading system, (1200mm×180mm×250mm) dimensions. The type and inclination of NSM bars were examined in addition to the type of adhesives. The results showed that the strengthening with 45° inclined GFRP bars is more effective for shear strengthening. The failure mode became a concrete compression failure rather than shear failure with remaining the system of NSM intact with beams without debonding. Also, it was found that the presence of stirrups is more effective on strengthening the shear capacity with the NSM technique. Rahman et. al., [10], studied the behavior of reinforced concrete beams strengthened with (NSM) technique with steel bars to get quick and economic strengthening solution. Seven beams (125×250×2000mm) in dimensions were tested to obtain mode of failure, failure load, strain behavior and deflection response. The highest improvement of load capacity reached was 46.8%, and the noticed failure modes were very similar. Mostofinejad, et. al. [11], used four beams (2000×300×200mm in dimensions) strengthened in shear with near-surface mounted (NSM) laminates technique. They studied different concrete compressive strength values and the presence of steel stirrups in the beams. The experimental results showed that the shear capacity of the beams was increased up to 69% and 41% for beams with and without stirrups, respectively due to the NSM technique. Also, the FRP (Fiber-Reinforced Polymer) shear contribution decreases in presence of the stirrups.

2. Test specimens
In this study, 7 beams were tested with a length of 2400mm, 160mm width and 300mm height. Two of them remained without strengthening. The first one was cast with normal concrete and the second was cast with 30% thermstone coarse aggregate replacement ratio to compare the beam failure mode and the decrease in the load carrying capacity due to partial replacement of natural aggregate with thermstone. The remaining five beams were cast with 30% coarse aggregate replacement ratio of thermstone (volumetric ratio), two of them which do not contain stirrups. The beams were designed to have a high flexural capacity to ensure the failure occurs in the shear zone. Three 16mm bars (had a 580MPa yield strength) were used as reinforcement for flexural, and 6mm stirrups bar (250MPa yield strength) @200mm, with two 8mm (420MPa yield strength) top bars. The loading system was four-point bending load, the details of beams and loads are shown in figure 1. All beams had the same longitudinal reinforcement but differed in the presence or absence of stirrups. Five styles of Near-Surface Mounted (NSM) strengthening configuration were used for shear strength by (GFRP) bars, the diameter of the used (GFRP) bars was 6mm. The models also differ in the style of strengthening configuration, there were five different patterns. The first pattern was inclined strengthening without stirrups, the second
type was inclined strengthening with stirrups, the third one was vertical strengthening with a distance of 300 mm without stirrups, the fourth type was vertical strengthening with a distance of 200 mm with stirrups and the fifth type was vertical strengthening with a spacing of 300 mm with stirrups. One of the beams remained without strengthening, as shown in table 1 and figure 1. The beam specimens were loaded under a four-point loading system as simply supported beams as shown in figure 1. The load was increased until failure of beams. The deflection at mid-span was collected by using LVDT, while the strains were recorded by data acquisition.

Table 1. Details of tested beams

| Beam No. | Lightweight coarse aggregate volumetric replacement ratio | Steel stirrups details | Spacing of Strengthening GFRP |
|----------|----------------------------------------------------------|------------------------|-------------------------------|
| B1       | 30% thermstone without stirrups                          | Ø6mm@ 300mm GFRP       |                                |
|          |                                                          | Ø6mm@ 300mm GFRP       | at 45° to the beam longitudinal axis |
| B2       | 30% thermstone Ø6@200mm                                  | Ø6mm@ 300mm GFRP       | at 45° to the beam longitudinal axis |
| B3       | 30% thermstone without stirrups                          | Ø6mm@ 300mm vertical GFRP |
| B4       | 30% thermstone Ø6@200mm                                  | Ø6mm@ 200mm vertical GFRP |
| B5       | 30% thermstone Ø6@200mm                                  | Ø6mm@ 300mm vertical GFRP |
| B6       | 30% thermstone Ø6@200mm                                  | ---                    |                                |
| B7       | Normal concrete 0% thermstone                            | Ø6@200mm               |                                |

Figure 1. Details of used beams in the test.
3. Materials
Ordinary Portland cement available in the local market has been used for the concrete mixtures approved for this research. The natural coarse aggregate of 4.75-19 mm in size was used. Where the samples were taken according to ASTM specifications, sieve analysis was done for samples and the obtained gradients were within the limits of the specifications. For fine aggregate, samples were also taken according to the ASTM specifications, and the results of the sieve analysis of the samples used were within the limits of the ASTM specifications. The lightweight aggregate was obtained from the remains of the broken thermstone, where it was re-crushed to a size similar to the size of the natural coarse aggregate. The sieve analysis was done and the sizes that passed through sieve were 19 mm and those remained on sieve 4.75 were used.

4. Concrete mix design
The approved mixture proportions were 1: 1.8: 2 with a water-cement ratio of 0.38, where the compressive strength was 35MPa at 28 days. Also, part of the coarse aggregates was replaced with lightweight aggregates of thermstone with (30%), volumetric ratio. The compressive strength at 7 and 28 days was tested using standard cylinders measuring (150 * 300 mm) in addition to test the hardened concrete for flexural and splitting tensile strengths at 28 days, as shown in tables 2, 3 and 4.

| Table 2. Details of concrete mixtures |
| Mix No. | W/C | L.W. rep. % | Kg/m³ |
|        |     |            | Water | Cement | F.A. | C.A. |
| 1      | 0.38 | 0          | 179.5 | 472.4  | 803  | 944.8 |
| 2      | 0.38 | 30         | 179.5 | 472.4  | 803  | 661.4 |

L.W.=Lightweight, F.A.=Fine aggregate, C.A.=Coarse aggregate, %.=Volumetric ratio, rep.= Replacement

| Table 3. Compressive strength of concrete at (7 & 28) days. |
| Mix No. | Replacement ratio. (%) | f’c after 7 days (MPa) | f’c after 28 days (MPa) | Drop in f’c (%) |
| 1       | 0 (normal concrete)     | 30.84                  | 35.26                  | ---            |
| 2       | 30                      | 22.86                  | 31.50                  | 10.66          |

| Table 4. Splitting tensile strength and modulus of rupture of the mixtures |
| Mix No. | Replacement ratio (%) | splitting tensile strength (MPa) | Modulus of rupture (MPa) |
| 1       | 0 (normal concrete)   | 3.45                    | 3.95                   |
| 2       | 30                    | 3.23                    | 3.55                   |

It is normal for the density of concrete to decrease when replacing the natural aggregates with lightweight aggregates such as thermstone. Certainly, the percentage of decrease in density depends on the type of the lightweight aggregate replaced and also the percentage of aggregate ratio. Table 5 shows the concrete density for different proportions of replacement with lightweight aggregates of thermstone.

| Table 5. Density of concrete with and without thermstone aggregate replacement ratio. |
| Mix No. | Replacement ratio (%) | Density of concrete (Kg/m³) | Drop in density (%) |
| 1       | 0 (normal concrete)   | 2407.9                   | 0                    |
| 2       | 30                    | 2233.2                   | 7.25                 |
5. Strengthening of beam specimens

To strengthen the beams using the NSM technique, (6 mm) diameter GFRP bars were used, with (400 MPa) break strength. Five patterns of configuration have been used to strengthening the beams, while the reference beams have not been strengthened for sake of comparison, as shown in figure 1 and table 1. The NSM technique that has been used in this research was done as follows: Firstly, grooves of (12*12mm) were made on both sides of the beam according to the specific locations, a thin layer applied on the grooves of primer base (which consists of a mixture of two materials produced by the DCP company), then it was left for 24 hours to dry. Then the GFRP bars were glued to the surface of the beams with an adhesive designated (Sikadur-31) was used as an adhesive to attach the GFRP bars to the concrete surface as a second layer (it comes in the form of a 2-component thixotropic epoxy adhesive). The grooves that the GFRP rods installed into them were filled. Figure 2 shows pictures of the installation stages of the GFRP rods on the beams.

![Stages of installing GFRP bars](image)

6. Installation of strain gauges

Two types of strain gauges (Tokyo Measuring Instruments Lab.) were used. The first type was mounted to the steel with a length of (5 mm) and the other type was applied on concrete surface with a length of (30 mm) with their adhesives obtained from the same corporation. Strain gauges were fixed at six locations of the beam, as shown in figure 3. Two of them were mounted to the stirrups at the left and right sides of the beam, and one was mounted to the middle of main longitudinal reinforcing steel bar, two were mounted on the concrete surface at the shear zone. They were inclined at an angle of 45° perpendicular to the direction of the shear cracks. The last strain gauge was fixed on one of the GFRP bars. The data were collected by the (data acquisition) that was connected with the computer and the strains were recorded during the test for all stages of loading.
7. Results and discussion
For the results to be arranged and easy to compare, they have been divided into many categories, load carrying capacity, load deflection curves, load strain response and mode of failure.

7.1. Load carrying capacity
When observing table 6, it is noticed that the maximum gain in load capacity was occurred in beam B4, where the ultimate load is increased by 29.6% compared to unstrengthened beam B6. Beams B2 and B5 had a load carrying gain of 22.2% when compared to beam B6, where these beams had stirrups and had the same number of strengthening bars but with different inclination angle. It was found that the angle of inclination of the strengthening bars did not have a significant effect on the load carrying of the beam due to that the part of the internal strengthening bar was located in the flexural zone and was not completely within the shear zone. Also, beams B1 and B3 had the lowest gain in the load carrying capacity which was 11.1% and 3.7% respectively (due to the absence of stirrups).

7.2. Load-deflection curves
All the deflections were measured up to the failure load, the location of the (LVDT) was at the midspan of the beams. Figure 4 shows the load versus mid-span deflection of the tested beams. When comparing the response of beams B6 and B7 (without external strengthening), it is noticed that beam B6 had a deflection higher than the deflection of normal weight concrete beam B7, with an increase in the deflection values, but with same curve pattern. When comparing the results of beams B1 and B2 (which were strengthened with 45° inclined GFRP rods) with beam B6, it is noticed that beam B1 had a smaller deflection than the other two beams, as for beam B2, the deflection curve was almost identical to that of beam B6. But it has the highest deflection value among the other two beams. While for beams B3 to B5, the difference in deflection is almost negligible during applying the load, beam B4 had less deflection at early stages of loading, and had the maximum deflection at failure load than other beams due to the small distance between the strengthening GFRP bars with the presence of steel stirrups. It can be seen in this figure, that beams B2 and B4 showed the highest value of maximum deflection (25.98 and 25.1mm) respectively, as shown in table 6. Except for beam (B1), all the beams had a deflection equal to or greater than that of the reference beam (B6) deflection at the maximum load level.

Table 6. Test results of maximum load and deflection

| Beam No. | Max. load (kN) | Deflection corresponding to max. load (mm) | Gained in max. load compared to B6 (%) | Mode of failure         |
|----------|----------------|------------------------------------------|--------------------------------------|-------------------------|
| B1       | 150            | 17.94                                    | 11.1                                 | Shear                   |
| B2       | 165            | 25.98                                    | 22.2                                 | Shear                   |
| B3       | 140            | 21.02                                    | 3.7                                  | Shear                   |
| B4       | 175            | 25.13                                    | 29.6                                 | Concrete crushing       |
| B5       | 165            | 22.42                                    | 22.2                                 | Shear                   |
| B6       | 135            | 20.26                                    | -                                    | Shear with concrete crushing |
| B7       | 185            | 20.13                                    | -                                    | Shear                   |
7.3. Load-strain response

As previously mentioned, the strain gauges have been placed at six locations on the beams as shown in figure 3. Table 7 shows the value of strain that has been recorded at failure load of each beam. Figures (5-8) show the load strain response at variable locations of the beams for different strengthening configurations. By noting figure 5 which illustrates the strains occurring at the main reinforcing rods for all beams, it can be noted that the strains have almost the same pattern as that of the reference path beam which had a linear response until reaching the failure load. This trend can be attributed to the main reinforcement in the beams which was designed to resist high moments and loads. Therefore, the strains in the main reinforcing rods remained within the elastic range.

If the internal moment is calculated according to the actual recorded strain values obtained from the test:

\[ \sigma = E \epsilon \]  \hspace{1cm} (1)

\[ E = 200,000 \text{ MPa (for steel)} \]

\[ F (\text{force}) = \sigma A_s \]

\[ \text{Hence, } F = E \epsilon A_s \]  \hspace{1cm} (2)

\[ M (\text{moment}) = F (d - \frac{a}{2}) \]  \hspace{1cm} (3)

\[ M = E \epsilon A_s (d - \frac{a}{2}) \]  \hspace{1cm} (5)

\[ a = \frac{A_s f_y}{0.85 f'c b} \]  \hspace{1cm} substitution in eq. (5)

\[ M = E \epsilon A_s (d - \frac{A_s f_y}{2 \times 0.85 \times f'c b}) \]  \hspace{1cm} (6)

For the beam (B6):

\[ f'c = 31.5 \text{ MPa, } b = 160\text{mm, } A_s = 603\text{mm}^2, f_y = 580\text{MPa, } \epsilon = \text{collected from the test for example, at load 70kN, } \epsilon = 1071.88 \]

\[ M = 200,000 \times 1071.88 \times 603 \times \left( \frac{261 - \frac{603 \times 580}{2 \times 0.85 \times 31.5 \times 160}}{2} \right) = 28.46 \text{kN.m} \]

The calculated moment at this stage of loading \( = \frac{P}{2} \times 0.8 = 70/2 \times 0.8 = 28 \text{ kN.m} \)

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**Figure 4.** Load deflection relationships of tested beams.
It is noticed that there is a convergence between the moment values calculated from the moment diagram and the values calculated through the strain that occurred in the main reinforcement during the test.

Figure 5. Load strain relationship at mid-span of longitudinal bottom rebars.

Table 7. Strain values at the failure load for each location of the beams (um/m)

| Beam No. | Left side on stirrups | Main reinforcement on stirrups | Right side on stirrups | Left side on concrete | Right side on concrete | GFRP Rods |
|---------|------------------------|-------------------------------|------------------------|-----------------------|------------------------|----------|
| B1      | 1411                   | 2130.8                        | 1492                   | -42.799               | -70.52                 | 675.86   |
| B2      | 1680                   | 2356                          | 1034                   | 190                   | 193                    | 1350     |
| B3      | 1017                   | 2112                          | 1232                   | -1546                 | 3276                   | -123     |
| B4      | 1532                   | 2881                          | 819                    | 81                    | -109                   | 2125     |
| B5      | 666.87                 | 589.38                        | Out of range           | 114.59                | -57.09                 | Out of range |
| B6      | 705.37                 | 2046.44                       | 1236.79                | 28.06                 | 54.62                  | -----    |
| B7      | 2062.98                | 3322.01                       | 1597.84                | 21.92                 | 2072.00                | -----    |

Figure 6. Load strain relationship at left and right stirrups.
As for the strain on the stirrups, figure 6 a & b. It can be noted that the recorded strain at stirrups was recognized at a load level of about (40-50 kN) approximately, that indicates the beginning of the transferring of loads from the cracked concrete to the stirrups at this stage. These values are close to the theoretical values of concrete strength that is calculated using equation (7).

\[ V_c = \lambda \frac{\sqrt{f_{c'}}}{6} \times b_w \times d \]  

(7)

Where \( \lambda \) is the modification factor to reflect the reduced mechanical properties of lightweight concrete relative to normal weight concrete [12]. The value of \( \lambda \) equals to 1 for normal concrete, and equal to 0.75 for concrete contained lightweight coarse aggregate. In this case when the replacement ratio of coarse aggregate with lightweight thermstone equal to 30%, the value of \( \lambda \) is equal to 0.925, by interpolation.

As for the strain on the GFRP strengthening rods. It begins to be significant at load level of 100 kN for beams that have stirrups, and at a loaded of 60 kN for beams without stirrups, the transfer of loads to the GFRP rods begins early, figure 7. These values of the loads are obtained when adding the values of concrete shear strength with the resistance of the stirrups calculated using equation (8)

\[ V_s = \frac{Av_f f_y d}{s} \]  

(8)

Ramezanpour et. al.,[13], used equation (9) to estimate the capacity of GFRP rods to resist shear force.

\[ V_f = \alpha \frac{A_f f_{fu} d}{s} \left( \cos\theta_f + \sin\theta_f \right) \]  

(9)

- \((Af)\), is the cross-sectional area of GFRP bars, \((28\text{mm}^2 \times \text{number of bars})\)
- \((f_{fu})\), is the ruptured tensile strength of GFRP bars, \((402\text{MPa from laboratory test})\)
- \((d)\), is the effective depth of beam specimen, \(201\text{mm}\)
- \((s)\), the spacing of the GFRP bars to the longitudinal axes of the beam, \((200\text{mm or 300mm})\)
- \((\theta_f)\), the angle of the GFRP bars to the longitudinal axes of the beam, \((45^\circ \text{ or } 90^\circ)\)
- \((\alpha)\), is a constant factor that indicates the ratio of the actual shear contribution of GFRP bars to the theoretical value.

The value of \((\alpha)\) was calculated through several models, and its value ranged from 0.44 to 0.8.[13]. In our calculations \((\alpha)\) was tacked (0.62), as average of the values (0.44 and 0.8). Table 8 shows the values of strength of concrete and stirrups depending on equations (7 & 8), and the estimated shear strength capacity of GFRP depend on equation (9).

By observing table 8, it is clear that the actual shear strength values that carried by GFRP bars are close to that values estimated by equation (9) for all beam specimens, except for beam B4, in which the estimated load carrying by GFRP rods was greater than the actual experimental value, this was the cause of failure occurred in the concrete compression zone and did not happened in the shear zone. The shear strain at the left side of shear zone of the beams is shown in figure 8. It is noticeable that the strain of the reference beam (B6), which has no external strengthening, begins to increase when the load reach 40 kN, as the strain before this load, the strain value was negligible. As for the other beams, which contain external strengthening with different patterns, it is noticed that the strain curve for these beams
is not uniform as it was in beam B6, where the strain value on the concrete surface is affected by the presence of the GFRP strengthening bars.

### Table 8. Shear strength of concrete and stirrups of strengthened beams

| Beam No. | f’c at 28 day (MPa) | (Vc) calculated as equation (7) (kN) | (Vs) calculated as equation (8) (kN) | (Vc+Vs) calculated as equations (7&8) (kN) | Experimental shear force at failure (kN) | The actual load-carrying by GFRP bars (kN) | Estimated load-carrying by GFRP bars using equation (9) (kN) |
|----------|---------------------|--------------------------------------|--------------------------------------|-------------------------------------------|--------------------------------------|------------------------------------------|------------------------------------------|
| B1       | 31.5                | 36.075                               | -----                                | 36.075                                    | 75                                   | 38.925                                   | 38.6                                     |
| B2       | 31.5                | 36.075                               | 18.27                                | 54.345                                    | 82.5                                 | 28.155                                   | 38.6                                     |
| B3       | 31.5                | 36.075                               | -----                                | 36.075                                    | 70                                   | 33.925                                   | 36.42                                    |
| B4       | 31.5                | 36.075                               | 18.27                                | 54.345                                    | 87.5                                 | 33.155                                   | 72.85                                    |
| B5       | 31.5                | 36.075                               | -----                                | 36.075                                    | 82.5                                 | 28.155                                   | 36.42                                    |

**Figure 8.** Load strain relationship of left side concrete beam (shear zone).

#### 7.4 Mode of failure

Figure 9 illustrates the failure mode of the tested beams. Failure of all beams was shear failure mode except in beam B4 which failed by concrete crushing. This is due to the small spacing between the strengthening GFRP bars compared to the distance used on other beams. The shape of the crack pattern of beams (B1, B2, and B5) was almost similar, as the crack starts from the top of the inner GFRP bar and ends at the bottom of the outer GFRP bar, passing through the middle GFRP bar. While for beam B3, the cracks after testing are confined between the inner and middle GFRP strengthening bars. Regarding beam B7, the crack was extended starting from the applied load position toward the supports. For beam B4, in which the failure was concrete compression failure, where the crushing in the compression area was observed, as well as the cracks occurred in the tensile zone at the beam mid-span.
8. Conclusions
The following conclusions are drawn from the results of the experimental test:

- Using some pattern of NSM technique for shear strengthening in concrete beams containing lightweight coarse aggregate replacement restores about 90% of the shear capacity of normal concrete beams without lightweight coarse aggregates replacement.
- The possibility of obtaining an increase in the maximum load of up to 29.6% while replacing the coarse aggregate with a lightweight aggregate of 30% when using the GFRP bars for strengthening the concrete beams with NSM technique.
- The effect of strengthening by GFRP bars on concrete beams is very small in the absence of stirrups and does not exceed 11.1% in the gain of ultimate load.
- Changing the distance between the strengthening GFRP bars has a greater effect as compared to the change of the angle of inclination of the GFRP rods on the ultimate load of the beams. It was found that the percentage of increase in the ultimate load was 7.4% for changing the distance between GFRP bars, while the percentage increase in the ultimate load was 0% for changing the inclination of GFRP bars with presence of stirrups.
- It is possible to use concrete containing 30% replacement of lightweight aggregate for normal work because the density and compressive strength is decreased by only 7.25% and 10.66% respectively, and the density did not reduce to the lower limits of the specifications of lightweight concrete.

9. References

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