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# REPAIRED REINFORCED CONCRETE BEAMS WITH NORMAL AND HIGH STRENGTH CONCRETE

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**ABSTRACT:-** In resent years several attempts were undertaken to repair damaged reinforced concrete structures. Studies on the effectiveness of repaired and strengthened reinforced concrete elements which fail primarily due to formation of major flexural cracks are same what limited for normal strength concrete (NSC) and very limited for high strength concrete (HSC).

The overall objective of the present work is to investigate the strength and deformation characteristics in flexure of reinforced HSC and NSC beams repaired with either with concrete alone or with fiber reinforced concrete or with Welded Wire Mesh (W.W.M).

From the results obtained, it was found that the beams were adequately repaired and the general mode of failure was flexural. The repaired beams had higher strength than the original beams. All repaired beams exhibited significant decrease in deflection than the original beams.
***Keywords-:*** beam, concrete, fiber reinforced concrete, flexural, high strength concrete, repair, welded wire mesh.

## 1. INTRODUCTION

The repairing and replacement of structure members become necessary when the performance of the structure with respect to ultimate loads and serviceability become unsatisfactory(1). In resent years emphasis is being given to repair and strengthening of structures in preference to demolition and reconstruction because of social and economic reasons(2).

There are many techniques for repairing reinforced concrete members such as:

### 1.1 Epoxy Resins Technique

The method of repairing concrete crack by epoxy injection has been successfully applied to numerous structures of various types. The repair process not only eliminates the unsightly appearance but also restores the intergraded of the damaged concrete.

There are limitations to the effectiveness of the technique. If crack widths are too great, the resin repair will than not hold under load. Conversely, if the cracks are too narrow for proper resin penetration, there will be no improvement to the beam stiffness(3).

### 1.2 The Plate Bonding Repair Technique

The development of glues based on synthetic resin has opened up another method of structural repair in which steel plates are bonded to the structural element with epoxy glue. These glues have adequate bonding strength and it has been shown that they can provide effective composite action between the steel plate and the concrete element to be strengthened.

A short coming of the method is the danger of corrosion at the epoxy-steel interface, which adversely affects the bond strength(5).

### 1.3 Ferrocement Technique

Ferrocement is a type of thin reinforced concrete, whose cement-sand mortar is reinforced with closely spaced small diameter wire mesh with or without steel bars of small diameters called skeletal steel bars. Ferrocement has a very high tensile strength to weight ratio and superior cracking behavior comparison with reinforced concrete(6,7).

### 1.4 Repairing By Conventional Methods Technique

In this method all cracked concrete was chipped out. The yielded tension steel was cut out and replaced with new steel bars. Finally new concrete was placed in the repaired zone. The structural behavior of the repaired beam was similar to the original one(8).

## 2. EXPERIMENTAL PROGRAM

The experimental program consists of casting eight specimens under reinforced concrete beams of 200mm by 300mm in cross – section and 2000mm in length. The concrete cylinder strength was 60 MPa for HSC beams and 30MPa for NSC beams. The beams were reinforced with two 16mm dia. bars at the bottom with yield strength of 450 MPa . The beams were provided with stirrups of 8mm dia. at 100mm center to center in shear zones. The cover to the reinforcement from all sides was 30mm. Fig 1 gives all beam details.

## 3. REPAIR PROCEDURES

Four methods of repairing are out lined as follows :

### 3.1 Method One ( Group A)

The methods of preparations are out lined as follows:

1. After failure the damaged concrete in the failure zones was chipped out in both the tension and compression zones of the beam. The limits of the repair zone were 500mm from the center line of the beam Fig.2.
2. The longitudinal steel in compression zone was straightened
3. The longitudinal steel in tension zone was cut out original tension steel.
4. New steel bars of 16mm diameter were welded to original 16mm diameter bars. The new steel having similar tensile strength.
5. All damaged stirrups were replaced with equal strength stirrups. The first stirrup was placed (S/2=50mm) away from the interface between the old and the new concrete. The remaining stirrups were spaced (S=100) mm as in the original beams.
6. At the interface between the old and the new concrete the aggregates of the old concrete were exposed and wire –brushed to remove any loose material. These procedures ensure a good bond between the two concerts.
7. A thin coat of low viscosity P.V.A was applied to the old concrete at the interface between the old and the new concrete to ensure a good bond between them.

The beams were placed in molds and new concrete with similar mix proportions to the old concrete was placed in the repair zone. Fig. 2 shows the details of repairing of this Method.

### 3.2 Method Two (GROUP B)

The same steps indicated in method one were carried out with step 3 being replaced by the following:

3. Keeping the old tensile steel bars.

### 3.3 Method Three (Group C)

The same steps, which indicated in method one were carried out with step 4 being replace by the following step:

4. New steel bars of 8 and 12mm diameter were welded to the original (16mm) diameter bars plus a W.W.M of 4mm diameter to ensure the same percentage of longitudinal steel ( $\rho = 0.0102$ ).

### 3.4 Method Four (Group D)

The same step indicted in method one but replace step 8 by the following step:

8. The beams were placed in molds and new concrete consisting of 1% (by volume) steel fibers was placed in the repair zone. Besides steel fibers, the new concrete had similar mix proportions to the old concrete. The same details used in Fig 2 could be used in this method of repairing with fibers in new concrete.

## 4. RESULTS AND DISCUSSION

The structural behavior of the repaired beam was similar to the original. Failure in both cases was characterized by yielding of the tension reinforcement, extensive cracking in the tension zone and crushing of the concrete on the compression face. All beams had a ductile failure mode. Typical characteristics of the test beam response to loading are summarized below:

### 4.1 Strength And Efficiency

The efficiency of repairing is defined as the ratio of repaired beams strength to its original strength ( on percentage basis).

Groups A,B,C and D of HC repaired beams recorded 90.8 %, 112.6%, 117% and 136.3% respectively.

Groups A,B,C and D of NC repaired beams recorded 96.2 %, 122%, 103.7% and 120.4% respectively as shown in Table 3.

The best efficiency recorded was indicated in group B (Method 2 of Repair) with efficiency of 122% and the other methods are prepared as follow:

Group (A) (Method 1 of Repair) recorded the lowest efficiency about 96.2%, this may be due to the effect of welding on the properties of steel bars specially the reduction of the yield strength.

Group (B) (Method 2 of Repair) recorded the highest efficiency (122%), and the difference in efficiency between method 1and 2 was (25.8%). There are two possible reasons behind this difference. Firstly there is no cutting in the old reinforcement and secondly the strain hardening the old reinforcement may also be a factor.

Group (C) (Method 3 of Repair) recorded the efficiency of (103.7%). The difference between method 1 and method 3 was about (7.5%).

Group (D) (Method 4 of Repair) recorded high efficiency of (120.4%). The strength capacity of the repaired beam in this group increased because the addition of steel fibers, the maximum load is controlled primarily by fibers gradually pulling out, and the stress in the fiber at the ultimate load is substantially less then the yield stress of the fiber. From the above information it exhibited that group (D) has a higher efficiency than groups (A and C), and approximately equal to group B. The deference in efficiencies between them were (24.2%, 16.7% and 1.6%) respectively. Table 2 shows the test results for strength and efficiency for all tested beams.

### 4.2 Deflection

Deflection of the beams was measured at mid span of the beam and plotted versus the load as shown in Figs 3(a – d)

Load –deflection curves of the beams through all loading stages up to failure consisted of several parts:

The first part, from zero load up to formation of the first flexural crack, is of relatively steep slope which means that the beam at this stage is of higher flexural stiffness.

The second part, extend from cracking load to yield point and is of smaller slope, the beam at this stage is of less flexural stiffness because of the development of concrete cracks.

Final part, extend from the point of steel yield up to failure of the beam. This part is relatively flat and the beam at this stage has little stiffness in flexure.

For Group (A), similar relations are observed between repaired and original beams.There was a small rise in deflection in the repaired beam compared to the originals one, up to failure. At a load of (120kN) there was a 3.3% rise, as shown in Fig(3-a). It may be noted that the rise in deflection for repaired beam is acceptable (span/360) as recommended in ACI 318-05[10]. This point is very important in repaired beams to avoid damaging nonstructural elements which attached to the beams.

Group (B), this group recorded the largest rate of drop in deflection in all stage of load. At cracking load the drop was of (16.7%) and at load of (120kN) the drop was of (21.5 %) compared to the original beam see Fig(3-b). There are two reason behind this difference, firstly because the enhancement of the strength of the repaired beam in flexure, and secondly

In group (C) the repaired beam showed a similar Load-deflection relations with the original beam. There was a small drop in deflection in the repaired beam up to failure load.

At a load of (120kN) recorded this drop was (2.1%) less than the original beam Fig. (3-c).

Group (D), the repaired beam has shown a little drop in deflection up to cracking load. After this point the difference between the repaired and original beam increased up to failure. At a load of (120kN) the drop was (8.3%) less in the repaired beam than the original beams Fig(3-d). The cause of this drop in deflection is the addition of steel fibers.

It may be noted that for load-deflection curves for repaired group B,C and D that an enhancement in stiffness and ductility occurred in the repaired beams compare to the original ones.

At point before yielding, groups A,B,C and D of HC repaired beams recorded 108.7 %, 70.7%, 79.3% and 72.3% respectively.

At point before yielding, groups A,B,C and D of NC repaired beams recorded 98.9 %, 74.7%, 93.9% and 87.8% respectively as shown in Table 4.

In group "A", similar relation are observed between repaired and original beams. Their enhancement in deflection of repaired beam until the failure load. At load (135kN) recorded an enhancement in the deflection about (13.3%) than the original as shown in Fig(3-a), and the new deflection agree with the calculating the allowable deflection as recommended in ACI 318-05[(10)]. Group "B", this group recorded the largest rate of decreasing in deflection in all stages of load, at cracking load recorded an decreasing of (32.5%) than the original beam, and at load (135kN) recorded an decreasing of (25.6%). As shown in Fig(3-b). The reason behind this behaviors retain to the high strength efficiency of the repaired beam comparison with NC Repaired beam as shown in plate No ( 6).

Groups "C", the repaired beam show a similar load-deflection relation, and a little decreasing in deflection up to failure load. At load (135kN) recorded an decreasing of (16.7 %) than the original beam, as shown in Fig(3-c).

The different between the decreasing in deflection for repaired NC beam and the high rage decreasing for repaired HC beam of this group may be because the good bond between the material of HSC beam and the flexural steel.

Group “D”, the repaired beam shown a decreasing in deflection up to failure. At load (135kN) recorded a decreasing of (24 %) than the original beam as shown in Fig (3-d). The reason behind this retain to the enhances the bond strength between the HC and the flexural steel when adding steel fiber .

### 4.3 Concrete Strain

The maximum concrete strain was found at the mid – span the beam.

At point before yielding, groups A,B,C and D of HSC repaired beams recorded 129.4 %, 59.2%, 70.9% and 85.6% respectively.

At point before yielding ,groups A,B,C and D of NSC repaired beams recorded 108.2 %, 81.4%, 81.4% and 67.3% respectively.

### 4.4 Crack Width

At point before yielding ,groups A,B,C and D of HSC repaired beams recorded 141.1 %, 59%, 74.4% and 72.1% respectively.

At point before yielding ,groups A,B,C and D of NSC repaired beams recorded 146.8 %, 67.4%, 95.1% and 57.1% respectively.

## 5. CONCLUSIONS

From the experimental results, the following conclusion may be drawn:

1. For HSC beams, Method four (group A) (Repairing beams by addition steel Fibers) gave the best method of repairing while Method one group (A) (repairing beam by using the same properties of original beams) is the least advantageous for repairing.
2. From the economic point of view, the Method two (group B) may be considered the best method of repairing for two reasons firstly because method two recorded satisfactory results for strength and behavior. Secondly because this method is the easiest in practice.
3. Comparison between effect of method of repairing on NSC and HSC beams shown that the HSC repair were more efficiency than NSC.

4. The treatment used for the interface joints between the old concrete and the new repairing concrete was satisfactory.
5. The repaired beams worked as a composite member and the separation was not observed.

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**Table (1) :** Details of groups.

| No. | Group No. | Beam No. | Type of concrete | Repaired beam No. | Type of concrete | Material used in repair | |
|---|---|---|---|---|---|---|---|
| | | | | | | Fiber % | W.W.M |
| 1. | A | NB1 | NC | RNB1 | NC | - | - |
| 2. | | HB1 | HC | RHB1 | HC | - | - |
| 3. | B | NB2 | NC | RNB2 | NC | - | - |
| 4. | | HB2 | HC | RHB2 | HC | - | - |
| 5. | C | NB3 | NC | RNB3 | NC | - | 4mm dia. 75*75 mm open |
| 6. | | HB3 | HC | RHB3 | HC | - | 4mm dia. 75*75 mm open |
| 7. | D | NB4 | NC | RNB4 | NC | 1.0 | - |
| 8. | | HB4 | HC | RHB4 | HC | 1.0 | - |

**Table (2) :** Subsidiary test result.

| Group No. | Beam No. | $f_c$` (Mpa) | $f_{sp}$` (Mpa) | $f_r$` (Mpa) |
|---|---|---|---|---|
| A | NB1 | 30.09 | 3.30 | 3.63 |
| | RNB1 | 30.42 | 3.25 | 3.60 |
| | HB1 | 61.30 | 3.60 | 9.06 |
| | RHB1 | 63.05 | 3.62 | 9.10 |
| B | NB2 | 31.05 | 3.17 | 3.60 |
| | RNB2 | 31.60 | 3.13 | 3.51 |
| | HB2 | 60.05 | 3.61 | 10.20 |
| | RHB2 | 62.20 | 3.58 | 10.80 |
| C | NB3 | 29.87 | 3.42 | 3.72 |
| | RNB3 | 30.05 | 3.32 | 3.68 |
| | HB3 | 59.03 | 3.70 | 9.65 |
| | RHB3 | 61.02 | 3.65 | 10.10 |
| D | NB4 | 30.03 | 3.30 | 3.55 |
| | RNB4 | 32.53 | 4.02 | 4.38 |
| | HB4 | 59.60 | 3.80 | 9.20 |
| | RHB4 | 63.31 | 5.60 | 14.72 |

**Table (3) :** Test result for strength efficiency & cracking load.

| Group No. | Beam No. | Type of failure | Cracking load (KN) | Repaired / original (%) | Ultimate failure loading (KN) | Repaired / original (%) |
|---|---|---|---|---|---|---|
| A | NB1 | Flexural | 2.7 | 77.8% | 158 | 96.2% |
| | RNB1 | Flexural | 2.1 | | 152 | |
| | HB1 | Flexural | 3.1 | 77.4% | 207 | 90.8% |
| | RHB1 | Flexural | 2.4 | | 188 | |
| B | NB2 | Flexural | 2.6 | 92.3% | 162 | 122% |
| | RNB2 | Shear | 2.4 | | 198 | |
| | HB2 | Flexural | 3.1 | 109.7% | 207 | 112.6% |
| | RHB2 | Shear | 3.4 | | 233 | |
| C | NB3 | Flexural | 2.3 | 104.3% | 162 | 103.7% |
| | RNB3 | Flexural & Shear | 2.4 | | 168 | |
| | HB3 | Flexural | 2.6 | 111.5% | 182 | 117% |
| | RHB3 | Flexural & Shear | 2.9 | | 213 | |
| D | NB4 | Flexural | 2.6 | 111.5% | 162 | 120.4% |
| | RNB4 | Flexural | 2.9 | | 195 | |
| | HB4 | Flexural | 3.1 | 142% | 182 | 136.3% |
| | RHB4 | Flexural | 4.4 | | 248 | |

**Table (4) :** Results of test parameter.

| Beam No. | Deflection at point before yielding (mm) | Repaired / original (%) | Concrete strain at point before yielding (x $10^{-4}$) | Repaired / original (%) | Max-crack width at point before yielding (mm) | Repaired / original (%) |
|---|---|---|---|---|---|---|
| NB1 | 4.65 | 98.9% | 14.7 | 108.2% | 4.7 | 146.8% |
| RNB1 | 4.6 | | 15.9 | | 6.9 | |
| HB1 | 4.6 | 108.7% | 10.9 | 129.4% | 5.6 | 141.1% |
| RHB1 | 5.5 | | 14.1 | | 7.9 | |
| NB2 | 4.75 | 74.7% | 16.1 | 81.4% | 4.3 | 67.4% |
| RNB2 | 3.55 | | 13.1 | | 2.9 | |
| HB2 | 4.6 | 70.7% | 10.3 | 59.2% | 3.9 | 59% |
| RHB2 | 3.25 | | 6.1 | | 2.3 | |
| NB3 | 4.85 | 93.9% | 16.1 | 81.4% | 4.1 | 95.1% |
| RNB3 | 4.55 | | 13.1 | | 3.9 | |
| HB3 | 4.6 | 79.3% | 11.7 | 70.9% | 3.9 | 74.4% |
| RHB3 | 3.65 | | 8.3 | | 2.9 | |
| NB4 | 4.9 | 87.8% | 17.1 | 67.3% | 6.3 | 57.1% |
| RNB4 | 4.3 | | 11.5 | | 3.6 | |
| HB4 | 4.7 | 72.3% | 9.7 | 85.6% | 4.3 | 72.1% |
| RHB4 | 3.4 | | 8.3 | | 3.1 | |

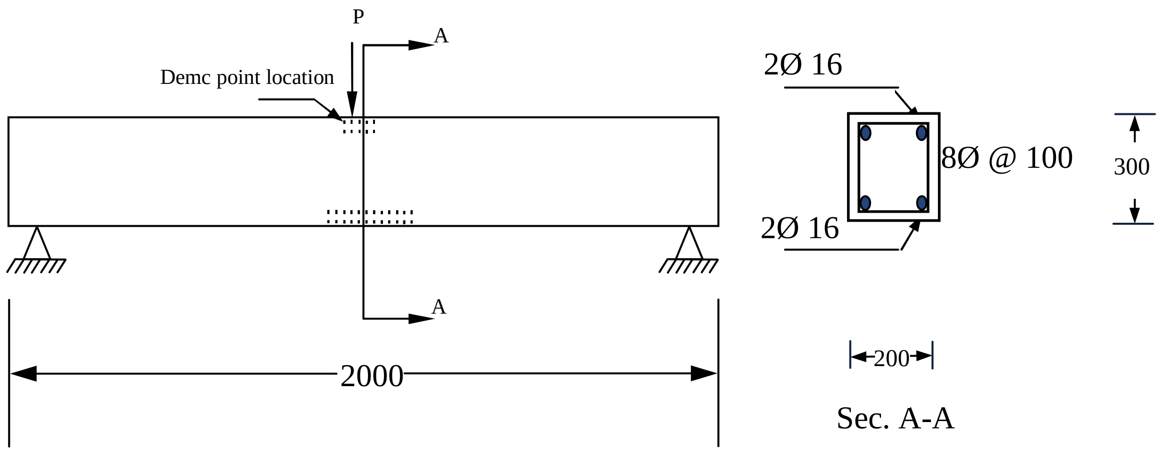
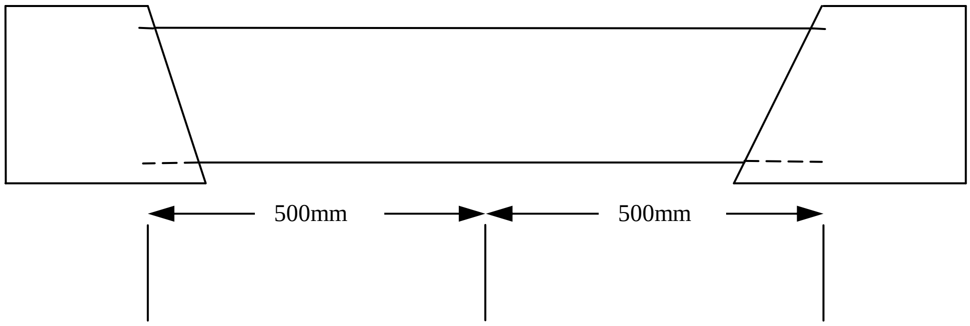
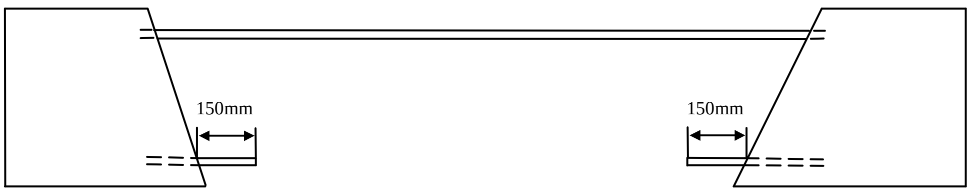


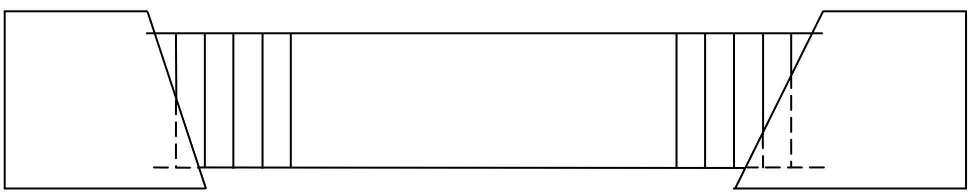

**Fig. (1) :** Dimension (mm) of test specimens.




**Step (1) :** Remove concrete in critical section, remove all stirrups.

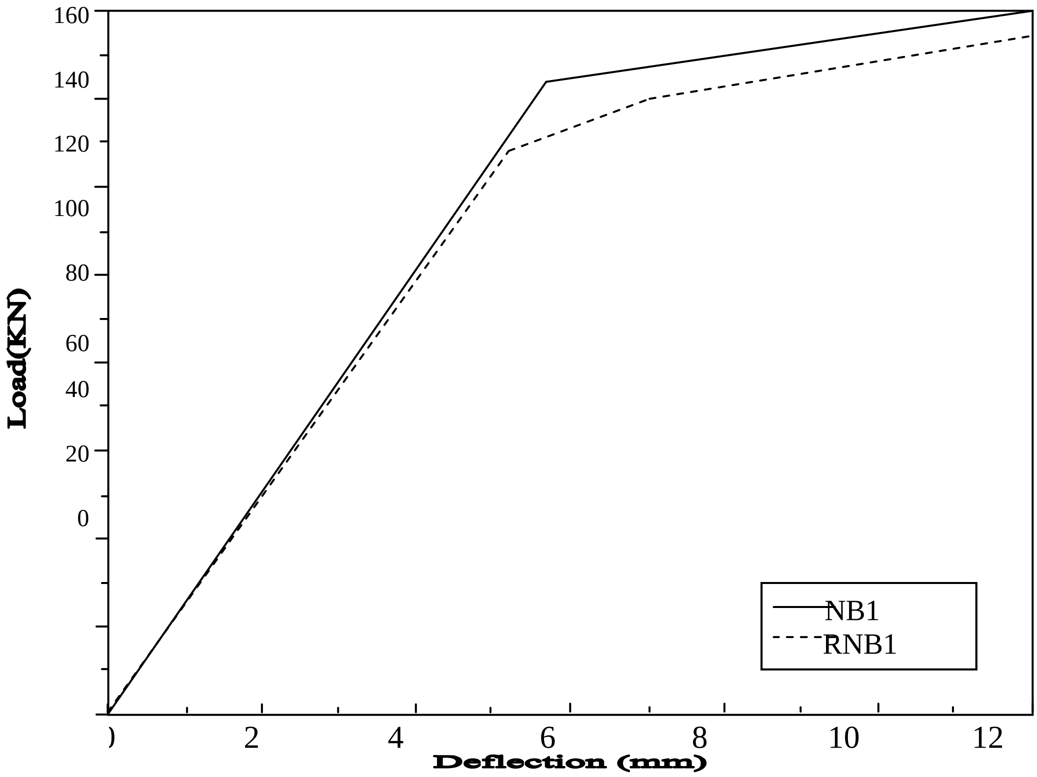



**Step (2) :** Tension steel cut out original tension steel.



**Step (3 ):** Weld new bars to original and provide stirrups.

**Fig. (2) :** Repair procedure for method one, two, three and four.



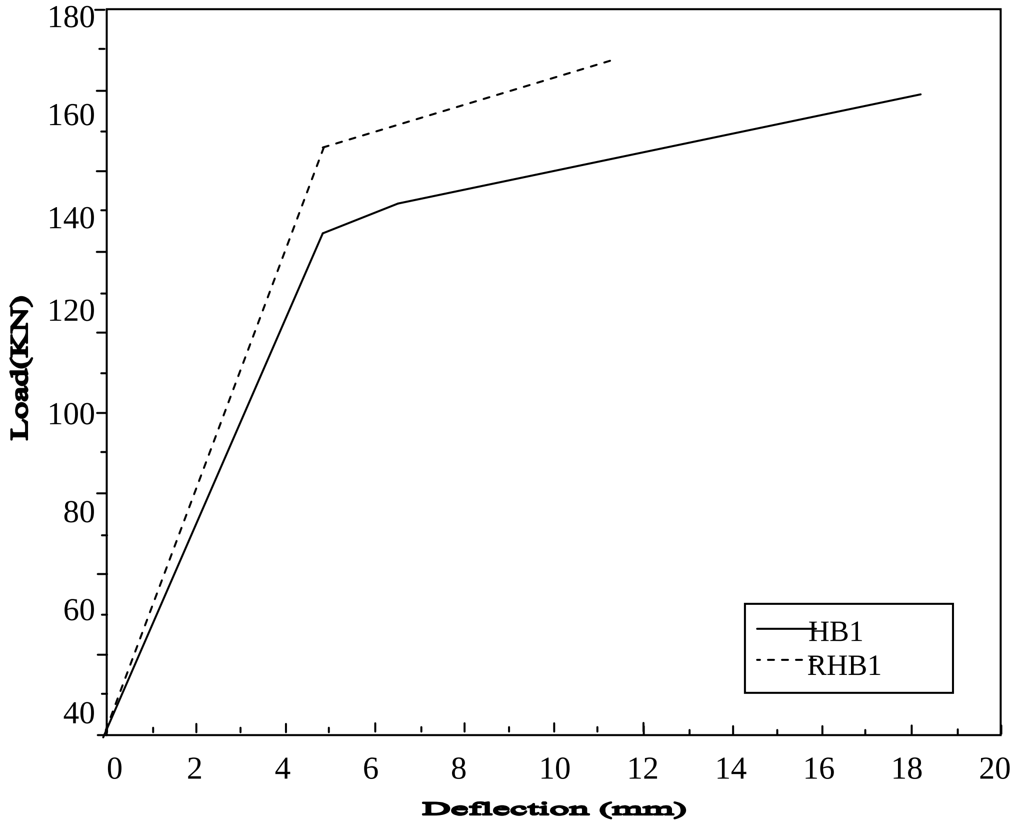
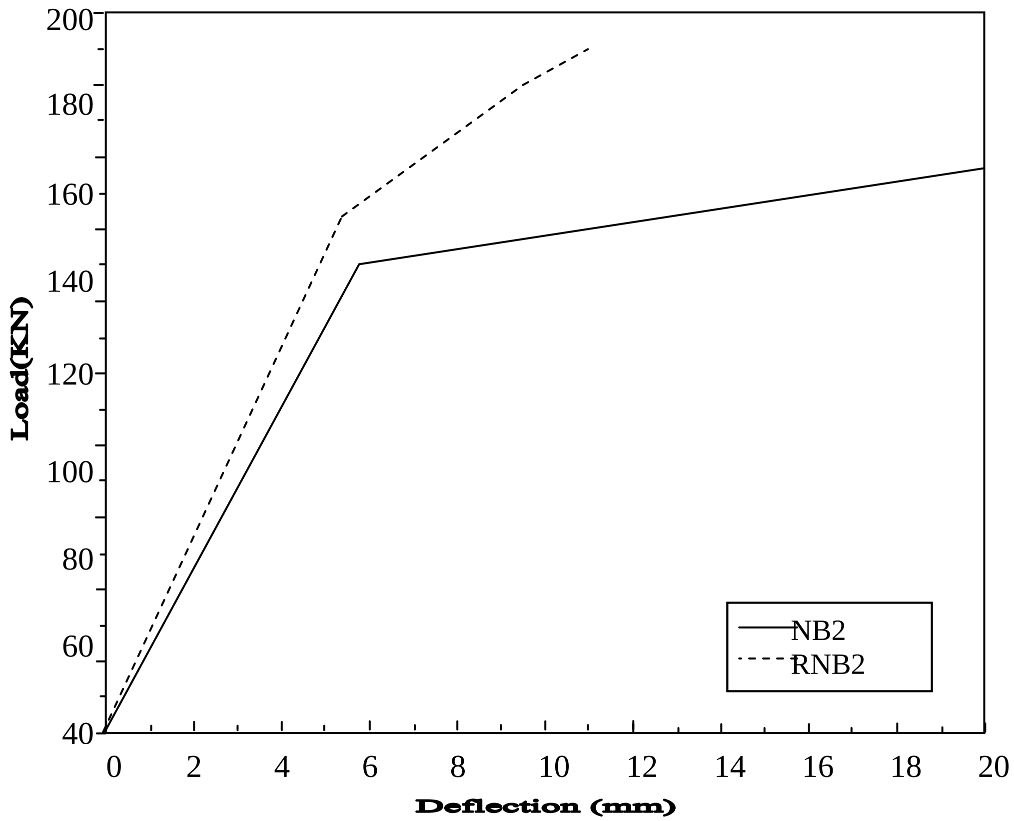




**Fig. (3-a):** Load-deflection curve for method one.



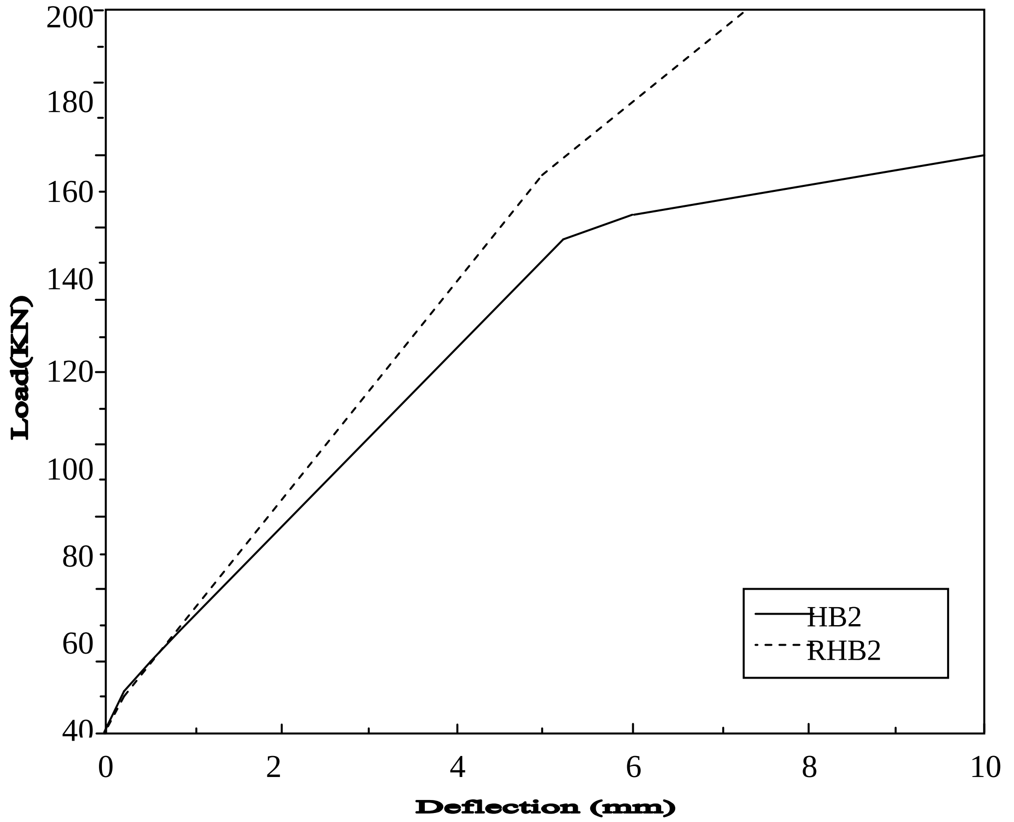
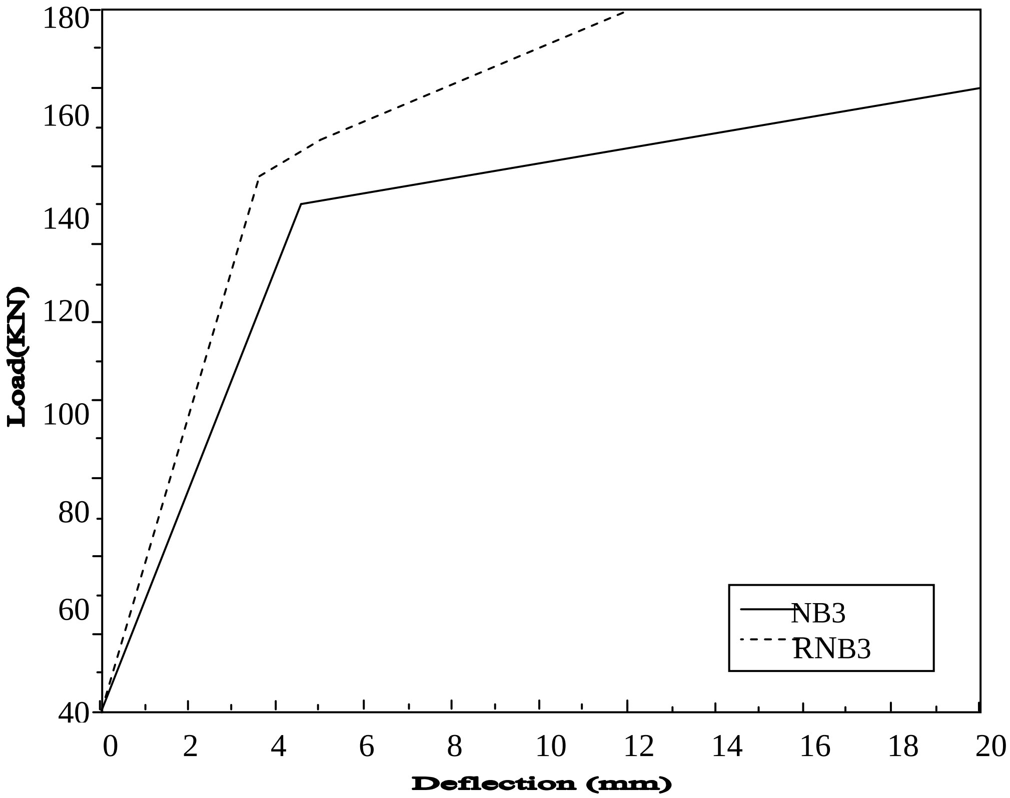




**Fig. (3-b):** Load-deflection curve for method two.



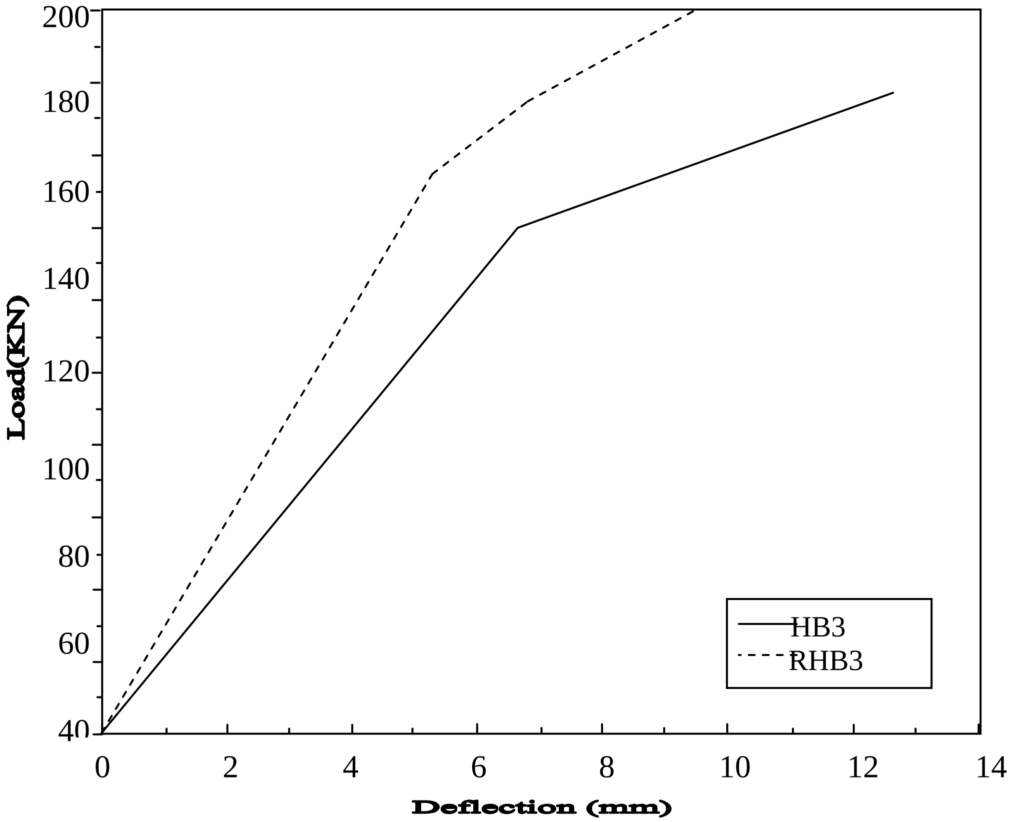
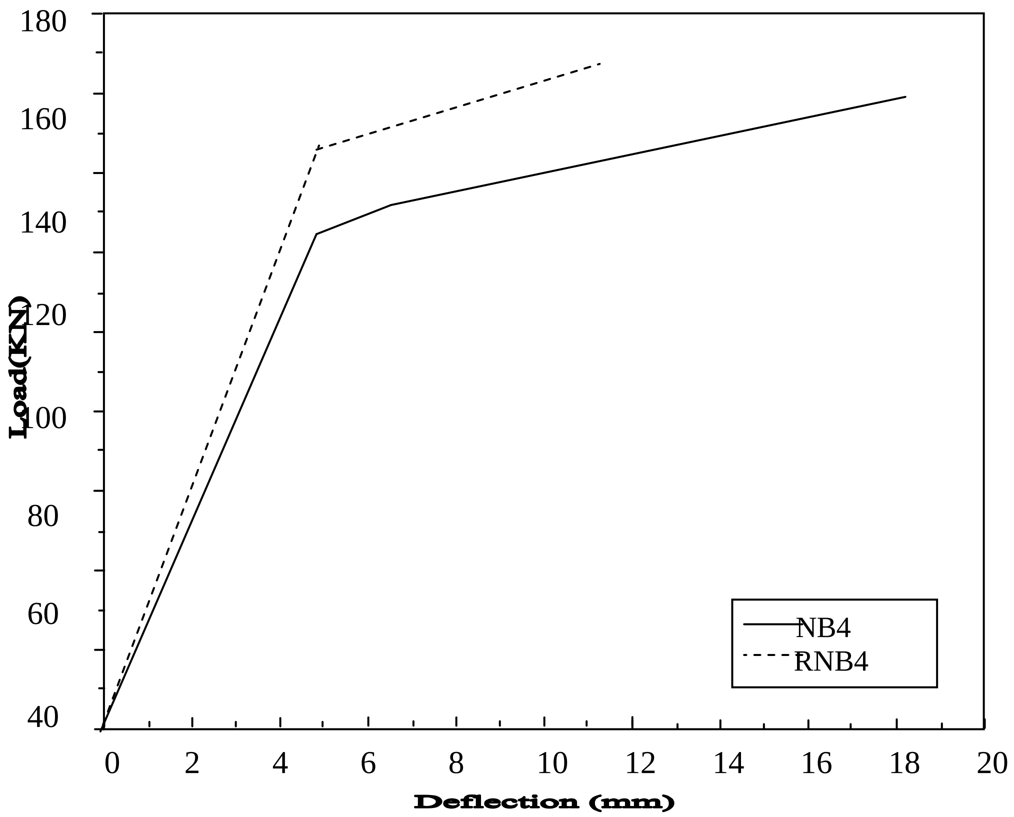




**Fig. (3-c):** Load-deflection curve for method three.



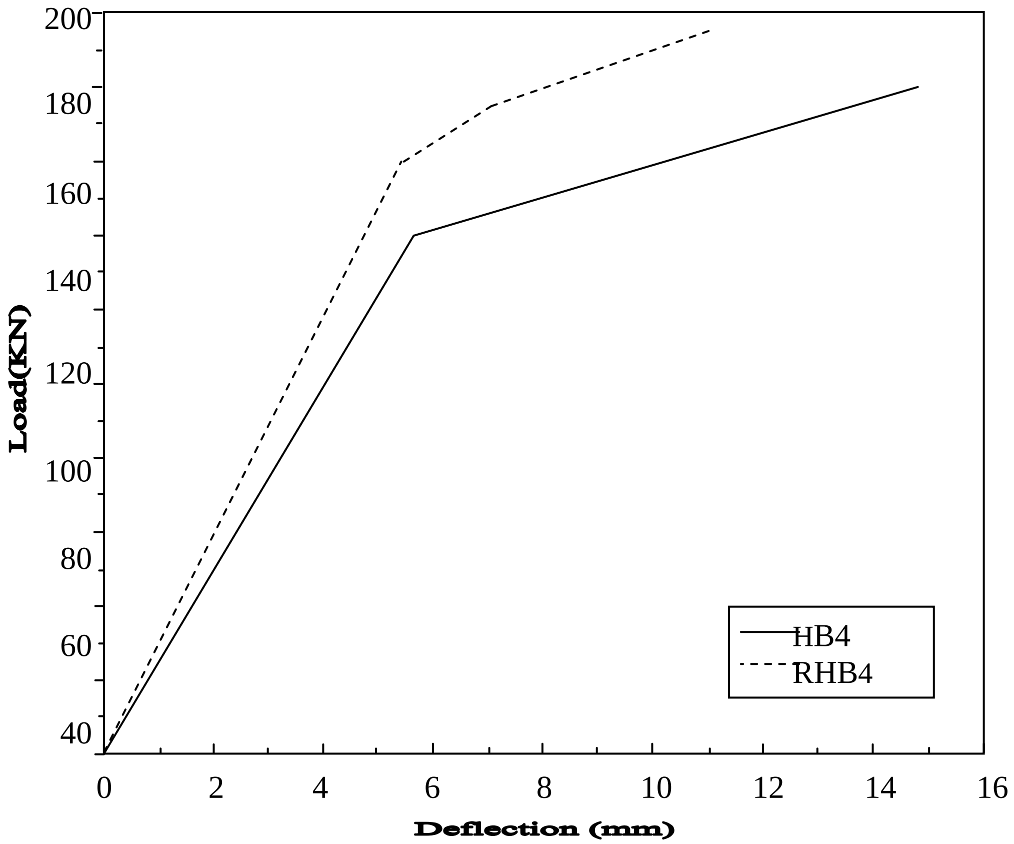




**Fig. (3-d):** Load-deflection curve for method four.

# تصليح الأعتاب الخرسانية المسلحة لخرسانة ذات مقاومة اعتيادية وخرسانة ذات مقاومة عالية

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**الخلاصة**

حديثا هناك عدة محاولات اجريت على تصليح المنشآت الخرسانية المسلحة. هذه الدراسات تركزت على فعالية التصليح والتقوية للعناصر الخرسانية المسلحة التي كان الفشل بسبب الشقوق الإنشائية الرئيسية وهذه الدراسات قليلة بالنسبة للخرسانة ذات المقاومة الاعتيادية وأكثر قلة بالنسبة للخرسانة ذات المقاومة العالية.

الهدف من البحث لدراسة السلوك والتشوهات في العتبات الخرسانية ذات خرسانة اعتيادية وخرسانة ذات مقاومة عالية والتي تم تصليحها بالطريقة الاعتيادية أو بإضافة ألياف الحديد أو باستخدام المشبكات الحديدية.

من نتائج الدراسة، وجد بان الأعتاب الخرسانية تم تصليحها بشكل جيد وكان نمط الفشل عن طريق الانثناء. الأعتاب المصلحة كانت ذات مقاومة أعلى من الأعتاب الأصلية وكل الأعتاب المصلحة أبدت نقصان واضح في الأود بالمقارنة مع الأعتاب الأصلية.