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Research Paper

Upgrading of Deficient Disturbed Regions in Precast RC beams with Near Surface Mounted (NSM) Steel Bars

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ABSTRACT

An experimental study has been conducted to explore the response of self-compacting RC drop in- beams retrofitted with NSM steel bars under short-term static loading. Eleven specimens have been tested in this program, each of (200x400x1600 mm) dimensions with two values of (a/d), namely (1.0 and 1.5). Two of these beams have been considered as reference beams (efficiently reinforced). Three beams with reduced reinforcement, two with deficient main nib steel, while the third with reduced steel in hanger. The rest have been upgraded with different arrangements to retire the expected drop in strength. It was found that reduction of nib reinforcements to the half, results in decreasing load failure of about (11% & 5%) for (a/d) values of (1 and 1.5) respectively. Also, it is observed that retrofitting the nib region with horizontal NSM steel bars led to increase the failure load capacity by about (28.7%) for (a/d) =1. While, the enhancement in strength for specimen with (a/d) =1.5 was about (24%). The increase in the strength capacity for upgraded hanger regions vertical and inclined bars for a/d (1) was about (21.4% and 14.2%) respectively.

1 Introduction

The dapped end beams is considered as one of the alternatives for the fabrication of precast beams with corbels or cross l-edge beams [1]. PCI Handbook [2] defined dapped ends as "bearing areas that are recessed or dapped into the end of the member ". Any reinforced concrete member can be divided in two types of regions; the first type in which the flow of stresses is uniform and there is no significant turbulence in stress contour lines. Such regions are called Bernoulli regions and termed as B-regions in which the flexural analysis theory can be applied. The second type of regions where there is some discontinuity of flow of stresses some disturbance of contour lines occur. Such regions are called Discontinuity regions and termed as D-regions. Some example of D-regions are dapped ends, corners, corbels, opening zones, deep beams, pile caps, point of application of concentrated loads . Both types of regions are shown in Fig. 1. To treat such regions two methods were suggested, which are shear friction method (PCI) [2] and STM models. Dapped

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ends allow getting a better fabrication in connection, reducing the height of the structure and enhance lateral stability. They are usually used very heavily in reinforced concrete structures such as prefabricated buildings, parking structures and recently in prefabricated conveyor belts the bridge girder, precast footings [3].

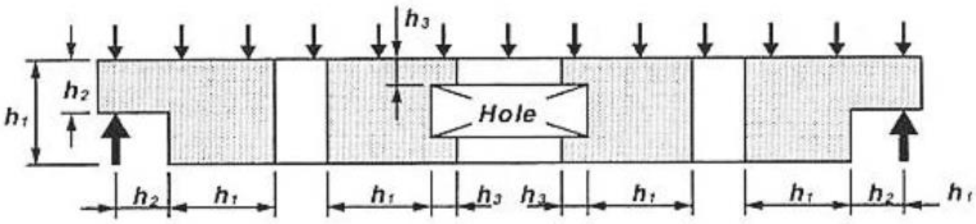


Fig. 1- Disturbed regions in Precast beams [4]

Many researchers studied the behavior and strengthening of reinforced concrete drop-in beams in terms of the most influencing parameters on the shear resistance of such members. Mattock and Chan [5] concluded that the use of the corbel design concepts in design of RC dapped end is valid for Shear slenderness ratio(a/d<1) but with adding hanger reinforcement. Liem [1] showed that the ultimate strength of dapped end with (45º) inclined reinforcement two times the strength of the case with horizontal or vertical reinforcement". Lu, et al. [6] studied several variables that may influence on the behavior, such as the concrete strength, (a/d) value, and amount of main reinforcement. Tests results showed that the shear resistance might be improved with adopting higher grades of concrete, the steel ratio of main reinforcement, and using smaller shear slenderness ratio (a/d). Ahmad, et al. [7] reported that the design for DEs using STM model gave varying results with changes in the angle of strut inclination. Aswin, et al. [8] studied Several variables as amount of the nib reinforcement, main flexural reinforcement, and concrete type at the dapped end region. It was found that the use of such compendious composite in the dapped area improved the failure load by (51.9%). While increasing the amount of nib and main flexural reinforcements enhanced the failure load by (62.2%) and (46.7%) respectively. Herzinger and Elbadry [9] Investigated experimentally the behavior of DEs using three analytical methods which are STM models, shear friction and diagonal bending methods.

Regarding prestressed precast beams with dapped ends, So [10] investigated the response of single stem T-beams. Two STM models were developed…... Double Thin-stem beams were considered in several studies, Nanni and Huang [11] tested two (18.3m) full-scale beams. The validity of using several alternatives of reinforcement to qualify the requirement of DEs design based on PCI method was explored. It was reported that the failure of both specimens was in shear -flexure in the full depth part of the beam and anchorage of the steel reinforcement is critical. Forsyth, [12], discussed several configurations at the disturbed region to improve the performance. It was demonstrated that the mode of failure of all specimens occurred within the full depth of the section across a critical diagonal crack that developed separately from initial cracks. Botros [13], then Botros et al. [14], different reinforcement arrangements have been tested, to study the effect of several parameters on behavior of drop in- beams as the pre-stressing of the nib, grade of concrete, web shear reinforcement, depth of the extended end, and splice length of the hanger steel. it was found that the crack development at service loading stage can be restricted by passing the pre-stressed steel within the extended end.

In some situations, there is a need to retrofit the partially damaged dapped ends and corbels.
Some of these cases are:

1. Errors in design that may occur leading to a reduction in actual capacity from the design value.
2. Severe conditions due to excessive loading resulting from careless uncontrolled use.
3. When it is required to increase the length of the extended end (or a corbel) to satisfy non-previously planned applications.
4. need to increase the capacity of the drop in- beams due to change in use in either load value or type.

Several studies have be published considering this topic. Huang et al. [15] studied the performance of precast prestressed concrete double T- beams upgraded using FRP fabrics. Two arrangements were adopted and compared. An orthogonal wrapping method was adopted. To insure that the failure occurred fiber "rupture" instead "peeling", anchors at ends were added which was obtained in addition to no of plies of FRP to have significant effect on improving capacity. Tan [16] investigated several arrangements for upgrading DEs in shear to accommodate increased loading. Such suggestions were CFRP plates, CFRP sheets (CS), and GFRP fabrics. Beyond failure, the damaged section was repaired with adding more reinforcement and the test was done again to investigate the range of benefit of the FRP system at the other end. Results revealed that the ultimate load was enhanced by 43%, 75% and 80% for the three systems respectively. It was reported that the STM model convenient for the evaluation of the improvement shear resistance. Tahir [17] tested 52 specimens with three defects that are introduced at the disturbed region which were inadequate development length of main steel at the DEs, and ignoring either the horizontally or vertically aligned shear reinforcing steel. Such defects were repaired with twelve strengthening techniques and a comparison was made to identify the best scheme. The arrangements were; steel angle that externally bonded at the re-entrant corner, unbonded bolt anchors, external jackets by steel plate, externally fixed wrapping and/or stripping of carbon fiber have been investigated. Results indicated that the insufficient development length of longitudinal reinforcing steel at the bottom fiber is the most critical parameter. Moreover, that horizontal carbon fiber wrapping in both the reduced and full depth zones with inclined CFRP stripping yielded good enhancement in response.

Huang and Antoni [18] tested three full-scale prestressed concrete (PC) DE beams with double T-sections. One any two externally fixed CFRP sheets were used with and without U-anchors. Tests yielded that the configuration of strengthening have a considerable effect of the behavior of the drop-in beams. The activity of upgrading of DEs using externally bonded carbon fiber reinforced polymers (CFRPs) was demonstrated experimentally and numerically by Nagy-György et al. [19]. Results indicated that capacity could have increased by up to 20% if the plates had been mechanically anchored. A.C. Dăescu1 et al. [20] discussed theoretically strengthening (RC) dapped-end beams by (CFRP) sheets through testing 17 different configurations. The modes of failure ranged from a sudden failure of the scenario of upgrading up to the desired gradual failure of the individual parts of the strengthening regime. Furthermore, a maximum enhancement in strength of 37.6% was obtained. Sas et al. [21] used the non-linear FEM to identify the most effective one of 24 configurations of (CFRP) sheets for strengthening (RC) dapped-end beams. Several variables have been considered which are; the characteristics of the CFRP, the procedure of strengthening and the angle of alignment of the fibers measured from the x-longitudinal axis. Results revealed that two failure patterns were recognized: rupture of CFRP and delamination of the FRP form the concrete surfaces. Moreover, it was reported that that high-strength NSM FRPs could considerably increase the capacity. Atta and Taman [22], tested experimentally eight specimens with various shapes of external prestressing technique directions: horizontally, vertically and inclined alignment. Results revealed that vertical alignment technique is a very effective method to improve the failure load of the tested drop-in beams up to 82%. Shakir and Abd [23] studied the response of RC self-consolidating DEs strengthened by CFRP sheets. it is found that a deficiency of the nib reinforcement by about (60%), leads to deterioration in strength by about (36%) for a/d=1.5 and (15%) for a/d=1.0. Furthermore, the increase in the strength capacity for a/d (1.5 and 1.0) was about (17% and 23%) respectively when using inclined strips to upgrade the hanger region, whereas the enhancement was about (11% and 18%) for a/d (1.5 and 1.0) respectively when using vertical strips. While, the enhancement in strength capacity for specimen strengthened at nib region with (a/d) =1.5 was about (10%) using L-shape sheets with horizontally aligned strips.

One of the first studies that discussed the NSM strengthening technique by CFRP bars was achieved by Al-Mahmoud et. al. [24].in this study, strengthening of cantilever RC beams against flexure have been investigated

Hosen et al. [25] used steel bars in strengthening four RC rectangular beams by the NSM technique. The test results showed that flexural strength increased up to 53.02% and energy and deflection ductility improved. In 2018, Shakir and Kamonna [26] extended using this technique in experimental investigation to improve the performance of deficiently reinforced high strength self-compacting concrete corbels. Ten specimens were tested, two values of shear slenderness ratio (a/d) were considered. It was reported that NSM steel bars retrofitting technique enhanced the load capacity noticeably. An enhancement of 57% and 41% when adopting shear slenderness ratio (a/d) of 0.85 and 1.25 respectively. The upgrading configuration termed as "Upside down V-shaped" is more efficient for (a/d <1). The horizontal bars arrangement is more influential when using large values of (a/d) i.e. a/d >1. Mohammad, and Al-Shamaa[27] studied experimentally the behavior of reactive powder concrete(RPC) corbels, strengthened with NSM CFRP strips with two configurations, inclined and horizontal. It was found that when a/d =0.65, failure load enhanced in range of (10.3% - 15.45%). Whereas for a/d =0.4, the enhancement was in range of (7.1% - 14.6%).

It is clear that there is no previous study considered using the NSM steel bars technique to strengthen the dapped ends. Therefore, steel bars are used in the work instead of CFRP bars in retrofitting drop in- beams including some internal deficiencies in reinforcement, because CFRP bars have several demerits in use as:

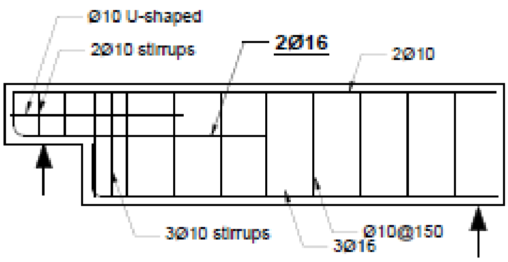
1. The cost of CFRP bars, and all the FRP composites, in general is high relative to the steel bars.
2. Being brittle (against bending) relative to steel bars, makes it difficult in handling and the work should be achieved by well-trained craftsmen.
3. The CFRP bars are not available in market thus some delivery problem may arise if compared with steel, which is widely used in RC concrete structures.
4. CFRP bars have good resistance to pure tensile stresses. However, They have lower resistance to flexural and shear stresses if compared with steel. In disturbed regions, i.e. dapped ends and corbels, there is exist substantial shear stresses in addition to flexural tensile stress.
5. Bent can be done in a steel bar simpler than in a CFRP bar. This is needed for continuous beams and when it is needed to provide bond by hooks at ends.
6. The upgraded member keeps its original aesthetic if compared with other techniques of strengthening.

As the strengthening by NSM steel bars technique is based on making groove in concrete without vibration, conventional ribbed steel bars are fixed using bonding material with strength not less than that of concrete, Steel bars are not prone to buckling nor to aggressive conditions. Thus, the added bars may be considered to produce strength as (or close to) the designed steel provided no slip to occur .This method can be used in straight and bent configurations for different purposes including change in use and design errors. It can be used to resist flexure, shear, axial forces and torsion with elements as beams, deep beams, corbels, dapped ends, one way slabs, staircases, around openings and one direction of two-way slabs. However, this method needs enough space to work, enough cover in concrete of acceptable strength to make the grooves, surfaces with dimension to make successive layers with enough spaces. Moreover, it is not practical to be used in strengthening long-span girders, orthogonal planes nor in two directions.

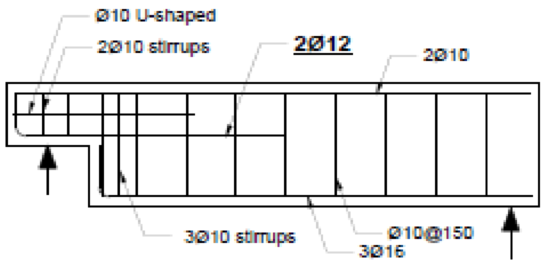
The present work represents utilizing the NSM technique using steel bars instead CFRP bars in repairing the deficiencies in reinforcement detailing .i.e. the ability of making bents(hooks) in steel bars is disregarded.

2 Experimental program

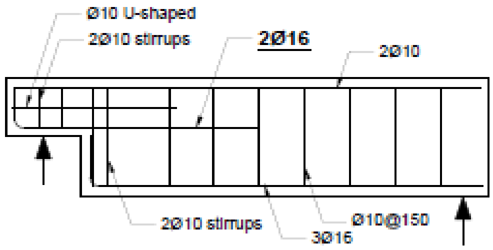
In the present work, an experimental study has been conducted to investigate the behavior of reinforced self-compacting concrete dapped end beams strengthened with NSM steel bars. The program consists of testing 11 specimens with dimensions of 200 mm width, 400mm depth and 1600 mm total length with nib end dimensions of 200mm depth and 250 mm length. Two values of shear slenderness ratio (a/d) are considered namely 1.0 and 1.5. Two specimens served as controlling samples with full-reinforcement designed by STM model. Three beams are intentionally casted with deficient reinforcement in either hanger or extended nib end. In the rest six beams, the NSM steel bars technique have been used to retire the lack in strength caused by these defects. Several suggestions have been investigated. The tested specimens are categorized into two groups according to ratio (a/d). Group one in which a/d=1.0, consists of one control specimen, two specimens with deficient nib or hanger reinforcement. Four specimens strengthened with different configurations. Group two in which a/d=1.5 included one specimen as control, one with reduced nib reinforcement and two others strengthened by NSM steel bars. For all the specimens, the concrete cover is taken as 40 mm to provide enough thickness for making the grooves Fig. 2, shows typical detailing of the tested specimens. All tests are done under the effect of one point static loading at the Structural Laboratory in the Faculty of Engineering / University of Kufa.



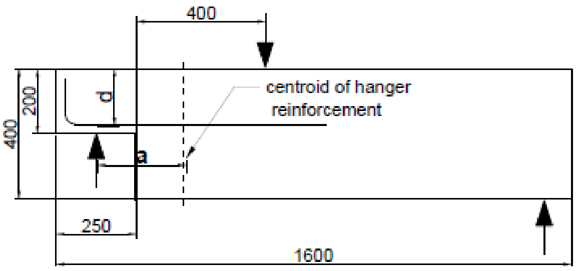
(a) Details of Reinforcement for Control Beam



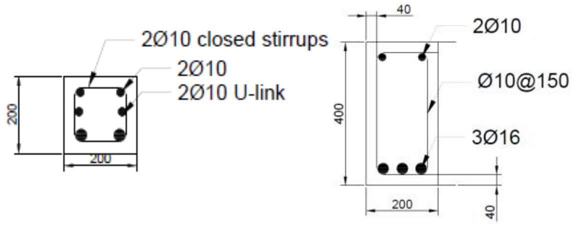
(b) Details for Nib Reduced Reinforcements Beam



(c) Detail of the Hanger Reduced Reinforcements Beam



(d) Dimensions of the tested specimens



(e) Cross sections through nib and full beam depth

(f) Cage of specimen with reduced hanger reinforcement

Fig. 2- Details of Reinforcement Beams

3 Materials properties

The self-consolidating concrete mix used in this work has the same constituent materials as in the conventional mix, which are cement, aggregate, and water with some other material to produce the workability, flowability and high strength. Cement use is Ordinary Portland cement (OPC), Type I. that is manufactured by Najaf cement company –Iraq. The physical and chemical tests are achieved to comply with the requirements of the Iraqi Specifications (IQ.S. 5/1984) [28]. Sand is considered as fine aggregate with maximum size of 4.75 mm that is taken from "Al-Ukhaidur" zone. Crushed gravel as coarse aggregate taken from Al-Niba'ee region having maximum size of 20 mm .Both fine and course aggregate are checked to satisfy the requirements of the Iraqi Standard Specifications NO.45/1984 [29]. Deformed steel of #16 mm served as the main tensile steel at the extended end for the control specimens and for flexural reinforcement of the beam. Bars of #10 mm served as stirrups reinforcement of the beam and hanger zone. Furthermore, bars of #12 mm served as a main reinforcement in extended end for the deficiently reinforced specimens. Tests are achieved according to ASTM A370-2005 specifications [30] and the results are shown in Table (1). All tests except steel are achieved at the Structural Laboratory in the Department of Civil Engineering / Faculty of Engineering / Kufa University. The staff of the laboratory department/ Bureau consultant of university of kufa performed steel tests.

Limestone Powder (LSP) which called locally as "Al-Gubra". The particle size of the LSP product conforms to EFNARC [31]. The superplasticizer HP580 is adopted according to ASTM C494 types F and G [32]. Tap water is used for mixing and curing purposes. Sikadur-30LP [33] Manufactured in Bahrain by (Sika Gulf B.S.C) has been used to fix steel bar segment used for strengthening within the grooves. it consists of two-part epoxy bonding and patching paste with hardener(A)/ Cold (B) of 1:3. the properties of the epoxy used are listed in Table (2), This component is suitable for aggressive weather condition that rebar may exposed.   Several trial mixes have been tested to produce a self-compacting mix with concrete cube compressive strength of 60 Mpa. Mix design of SCC met the requirements of filling and flow abilities, segregation resistance according to ACI Committee 237R-07[34]. Table (3) indicates the mix proportions of the mix adopted in the present study.

Table 1-Properties of Steel Bars

Bar Diameter (mm)	Yield Stress* (MPa)	Ultimate Strength* (MPa)
10	575	674
12	600	694
16	550	665

(***** value is average of 3 segments with length of 50 cm each)

Table 2- Epoxy properties

Color	Setting Time (Min.)	Compressive Strength (MPa)	Bond strength (MPa
A: White	20C>60 min.	>70	2 days >7
B: Black			14 days >10

Table 3- Proportion of Concrete Mixes

Parameters	Self -compacting Concrete (SCC)
Cement (kg/m^3)	400
Fine Aggregate (kg/m^3)	962
Course Aggregate (kg/m^3)	780
Limestone Powder (kg/m^3)	75
Water (kg/m^3)	125
Water/ Cement Ratio	0.31
Superplasticizer (L/m^3)	6

4 NSM Steel Bars Strengthening System

The NSM technique for strengthening by steel bars includes the following steps:

1. Sawing grooves or slits in concrete cover with dimensions of (1.5-2) times the bar diameter, Fig. 3.
2. Cleaning grooves by high pressure jet of water and remove all particle or concrete powder.
3. Ensuring that very dry grooves before applying epoxy resin then lay the first layer of epoxy.
4. Embedded the NSM rebar into the prepared grooves, and then complete filling the groove with epoxy.
5. Specimen is tested after curing time in range (7-10) days depending on climate.

Fig. 4 shows the steps of the process of strengthening. Using NSM steel bars that is adopted in the present work.

In the program of the study, the strengthening arrangement of the specimens are selected carefully depending on the results of the deficiently reinforced specimens. The control and beams with defects are tested first and the cracking pattern is studied. Then, the strengthening schemes are proposed accordingly. Shakir and Abd [23] demonstrated that the reduction of the hanger reinforcement is more effective for small values of (a/d) thus strengthening for defects at hanger zone is considered for Group 1 only. Table 4 and Fig. 5 demonstrate the details of the strengthening configurations for the tested specimens.

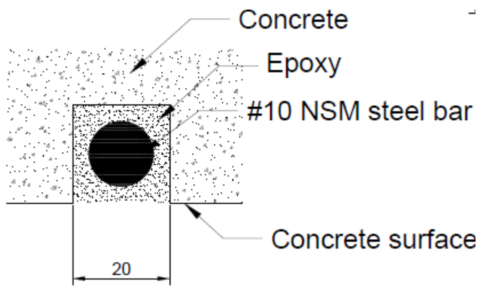


Fig. 3- NSM Groove Details

(a) Making Grooves

(b Groove are cleaned and dried

(c) preparing the paste

(d) Laying the 1st layer of epoxy

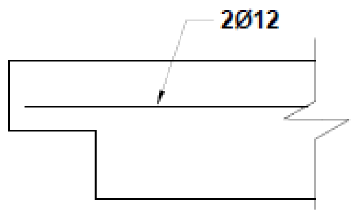
(e) Installing the steel bars

(f) fulling grooves with epoxy

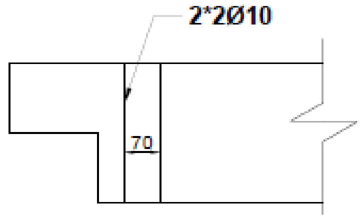
Fig. 4- Step of Strengthening by NSM steel bar technique

Table 4- Characteristics of the Tested drop-in Beams

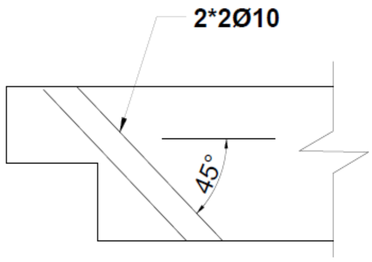
Symbol Beams		**Description of the Strengthening Scheme**
Group-1 a/d= 1.0	C001	Full reinforcement
	RN01	The nib reinforcement is reduced by 44%
	SNH1	1#12 each side at nib reinforcement
	SNI1	1#10 –horizontal +1#10 inclined by 45° each side
	RH01	The hanger reinforcement is reduced by 33%
	SHV1	2#10 vertically at hanger zone-each side
	SHF1	2#10 inclined by 45° at hanger zone-each side
Group-2 a/d= 1.5	C002	Fill reinforcement
	RN02	The nib reinforcement is reduced by 44%
	SNH2	1#12 each side at nib reinforcement
	SNI2	1#10 –horizontal +1#10 inclined by 45° each side



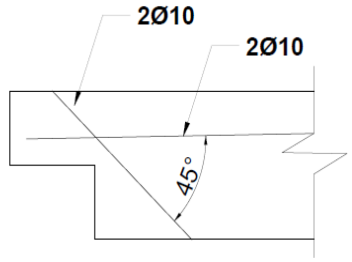
(a) Specimens SNH1and SNH2



(b) Specimen SHV1



(c) Specimens SHF1



(d) Specimens SNI1 and SNI2

Fig. 5- Details of Strengthening Configurations of Dapped end Beams

5 Test setup

All the tests are done by the universal testing machine at "Lab. of Structures", shown in Fig. 6 with maximum capacity of 200ton. The drop-in beam samples are simply supported over c/c span of 1.260m and 1.34m for shear slenderness ratios of 1 and 1.5 receptively, Steel bearing plate of (100mm x 200mm x10mm) used to distribute the concentrated load and supports with the capacity of concrete and avoid the local failure at such regions. Dial gauge (increment of 0.05mm) is positioned under the point load directly to measure the vertical deflection with progress of loading.

Fig. 6- Details of Strengthening of Dapped end Beams

6 Experimental results

In this section, results of the experimental work that is devoted to study effect of lack of amount of reinforcements in hanger and nib zone, effect of the shear span to effective depth ratio and behavior of strengthening configurations by NSM steel bars in both regions on the overall response of dapped end beams. In all specimens, the first cracks initiated around the corner of dapped end beam. Then, the flexural crack initiated with higher loads close to the mid span of the beam with vertical or diagonal trend towards the point load. Moreover, for all of the strengthened specimens, slip occurred close to the final stages of loading because of inadequate provided development length. The first crack loading, failure load, modes of failure and deflection at failure are listed in Table (5). The discussion of results obtained for specimen is presented in the following sections.

Table 5- Ultimate Loads and Failure Modes for the Beam Specimens

(a/d) Ratio	Specimen	Cracking load*(kN)	Ultimate loads (kN)	Max. deflection (mm)	Failure Mode
Group-1 (a/d= 1.0)	C001	65	371	8.60	Diagonal shear at the re-entrant corner
	RN01	60	330	8.10	Diagonal shear failure in the extended end
	SNH1	75	425	9.10	Slip + Diagonal shear failure in the extended end
	SNI1	75	355	7.95	Slip + Diagonal shear at the Re-entrant corner+ crushing in compression
	RH01	70	350	7.85	Direct shear at the extended end
	SHV1	75	425	9.60	Slip + Diagonal shear at the full depth of the beam
	SHF1	70	400	9.15	Small slip+ Diagonal shear at the full depth of the beam
Group-2 (a/d= 1.5)	C002	60	250	8.20	Diagonal shear at the extended end +crushing in compression
	RN02	65	238	7.50	Diagonal shear in the extended end
	SNH2	50	285	7.15	Slip + Diagonal shear in the extended end
	SNI2	70	295	8.40	Slip + Diagonal shear at the extended end +crushing in compression

* initiated at the re-entrant corner.

6.1 Control Specimens (C001, C002)

These specimens are reinforced with full reinforcement, (3-Φ10mm) stirrups for hanger zone and (2-Φ16mm) as tension reinforcement in nib zone to show the effect of (a/d) ratio on the general response. The results are depicted in Fig. 7. It can be concluded that reducing (a/d) value from (1.5) to (1.0) results in increasing the failure load capacity by (48.6%). Fig. 8(a) shows the cracking pattern for the specimen C001 at failure. the first crack initiated within the re-entrant corner at load level of 65kN. With further load, more cracks initiated and developed diagonally to the compression zone. At load level of 120 kN; the first flexural crack initiated. At load level of (180 kN), the first re-entrant diagonal crack penetrates most of the nib depth. The failure occurred at load of (371 kN) following diagonal shear cracking at the re-entrant corner. The cracking propagation at failure instant for the specimen C002 is depicted in fig. (8b). the initiation of the first cracking is recorded at the re-entrant corner as in C001 within nearly the same load level. With further load, cracking penetrates diagonally, and beam takes to rotate as two rigid elements. some crushing is developed at the compression zone within load level of (200 kN). At loading stage of 125 kN; the first flexural crack occurred. The beam failed at load level of (250 kN) due the re-entrant diagonal cracking combined with the spalling of the top compression concrete layer. Furthermore, slight reduction (about 5%) in deflection has been obtained when adopting a/d=1.5 rather than 1.0

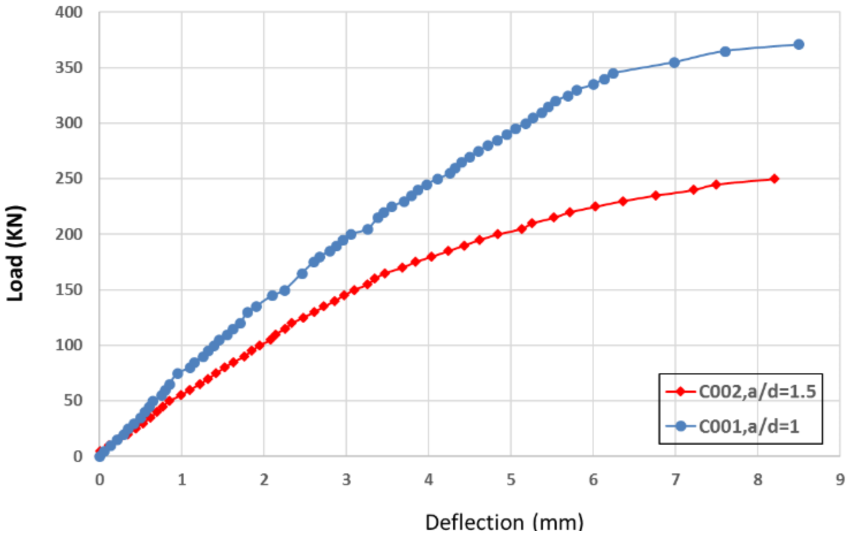
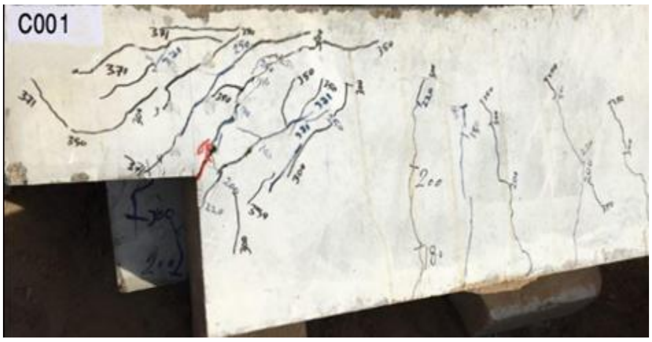


Fig. 7- Load-Deflection Curves for Specimens with Different (a/d) Ratio



(a) specimen C001

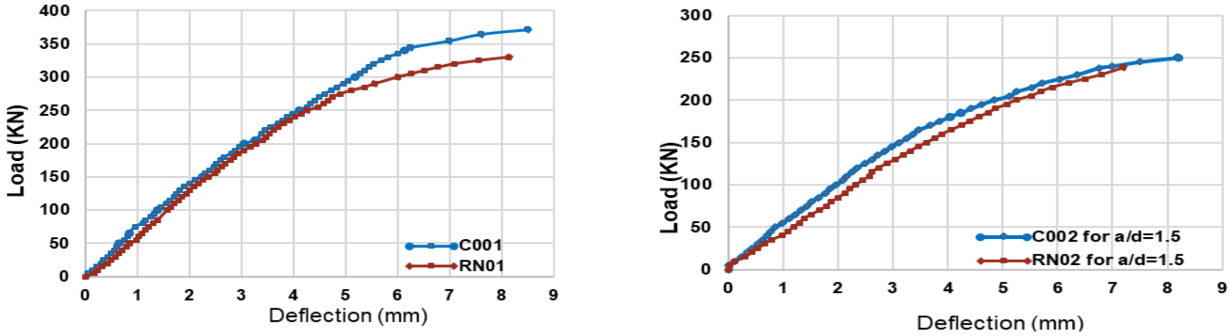
(b) specimen C002

Fig. 8- Cracks Patterns at failure for the Control Specimen

6.2 Specimens Control Reduced Reinforcement (RN01 & RN02)

To study the effect of reducing the nib reinforcement on the behavior of drop in- beams, two specimens RN01 & RN02 have been tested with the same reinforcement of the control specimens C001 & C002, except that (2-Ø12mm) instead of (2-Ø16mm) as a nib reinforcement, i.e. about 44% reduction. Load deflection curves are plotted against the control specimen; the results are shown in Figures 9 and 10. Results reveal that the reduction in nib reinforcement led to a decrease in the failure load and deflection about (11%) and (5%), respectively for specimen RN01. In addition, it is found that for specimen RN02, a reduction in the failure load and deflection about (4.8%) and (12 %), respectively. The small differences between the control specimens (with full nib reinforcement) and the specimens of reduced reinforcement may be attributed as that the PCI method overestimates the steel amounts with self- compacting high strength concrete and there is a need to introduce some modifications on the method to treat high strength concrete dapped end beams. For specimens C001 and RN01 .i.e. Fig. 8(a) with Fig. 10(a), the same pattern of crack propagation can be seen. Also, the two specimens failed by the re-entrant diagonal cracking mode. For specimens C002 and RN02 as in Fig. 8(b) and 10(b) the same can be said and the two beams followed the same mode of failure i.e. re-entrant diagonal cracking accompanied by crashing in compression zone.

This insures the need for more studies to check if the PCI method need to be modified to include the type of concrete (i.e. the self-compacting high strength concrete, reactive powder concrete or steel fiber concrete) in the design steps of the method. Regarding deflection, It is found that the maximum deflection reduced slightly by 6% and 8.5% for a/d=1 and a/d=1.5 respectively.



a) load deflection curve of RNO1 against C001 *b) load deflection curve of RNO2 against C002*

Fig. 9- Effect of Reduction Nib Reinforcement with Different values of (a/d) Ratio

(a) specimen RN01 *(b) specimen RN02*

Fig. 10- Cracks Patterns for the Specimens with Reduced Reinforcement at Nib region

6.3 Specimen Control Reduced Reinforcement (RH01)

In this specimen All the steel is kept the same as control specimen expect that at hanger reinforcement (2-Ø10mm) stirrups were used i.e. about 33% reduction. the load-deflection curve against specimen C001 is shown in Fig. 11 and the cracking pattern is shown in Fig. 12, The failure of this specimen occurred by diagonal shear failure in the re-entrant corner after that change the failure mode to extended end at load about (350 KN) as shown in the Fig. 11. It can be noticed that the reduction in hanger reinforcement results in a slight reduction of the failure load by about (6%), and higher deflection at failure about (9%) as shown in Fig. 12. It can be seen that there is no significant reduction in beam capacity due to deficient amount of hanger steel. This may be attributed to the fact that the concrete resist most of the shear stresses produced within the dapped end region. Then, the steel of the hanger will be less effective than the case of dapped ends with normal strength concrete. Also, comparing crack pattern for specimen RH01, Fig. 12, with that for specimen C001, Fig. 8(a). it is obvious that specimen RH01 failed by the same mode of failure as in C001 but with some crashing at compression face close to the point of load application. Furthermore, it was found that the maximum deflection reduced by 9%.

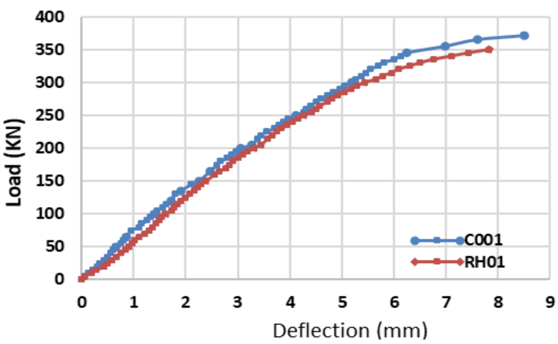


Fig. 11- Load-Deflection Curves of Specimen (RH01) against (C001)

Fig. 12- Cracks Patterns for the Specimen with Reduced Reinforcement at Hanger

6.4 Specimens SNH1 & SNI1; (a/d) =1.0

Although that the reduction of the main nib steel has negligible effect on capacity of specimens of high strength concrete. The reduced amount of area is to be substituted by NSM technique. Two configurations are examined; one consists of (1#12) horizontal bar each side welded with a vertical segment close to the re-entrant corner to form a T-frame shape; beam (SNH1). The second configuration consists of (2#10) bars one horizontal other inclined by 45^0; beam (SNI1). The two bars are welded together at the point of intersection. It can be seen that the maximum enhancement in load capacity is due to the horizontal configuration and it is about 15% with respect to specimen (C001) and 29% with respect to specimen (RN01). The loading history and crack patterns for specimens (SNH1) and (SNI1) are shown in Fig. 13 and Fig. 14 respectively.

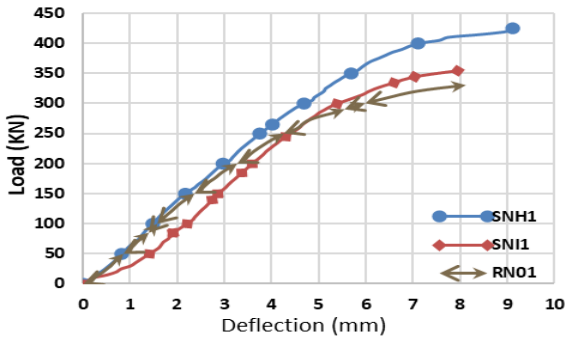
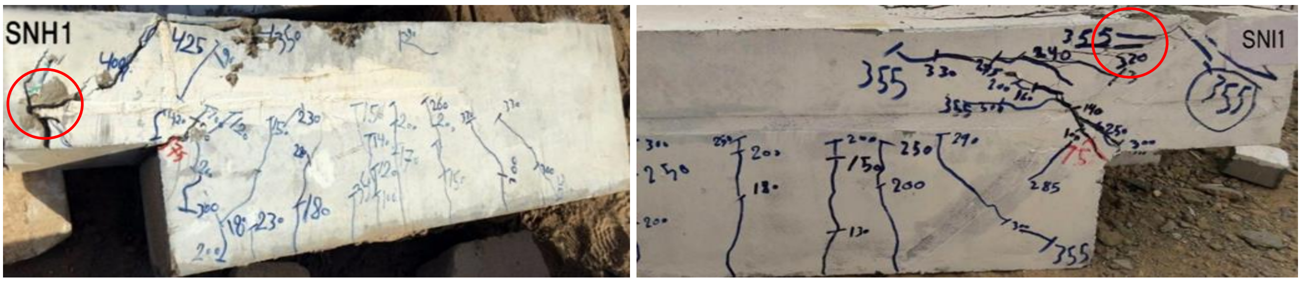


Fig. 13- Effect of Strengthening Nib Region on Response (a/d=1.0)



(a) specimen SNH1 *(b)specimen SNI1*

Fig. 14- Crack Patterns for Strengthening beams SNH1 & SNI1

It is clear that the horizontal configuration, specimen SNH1, yielded a load capacity of 425kN if compared with 355kN for beam SNI1. This may be due to the added vertical segments that made the horizontal steel to act as a framing element that might increase the rigidity of the frame and improved the overall stiffness of the beam. In addition, this segment may act as a shear reinforcement that crossing the path of the diagonal crack developed around the re-entrant corner. It is to be mentioned that more capacity may be obtained if better anchorage is provided for the NSM bars within the extended end. For the specimen with mixed steel reinforcement, it can be realized that the inclined bars have small effect on the capacity due to the early failure in the welding points between the horizontal and inclined segments. Regarding the crack patterns, it can be seen for specimen SNH1, Fig. 14(a), that crack is shifted away from the critical zone to either the extended end or the full depth beam. Cracking continue to develop up to failure following a diagonal cracking at the extended end with some crushing in compression zone. For specimen SNI1, Fig. 14(b), it is obvious that the inclined bar result in shifting some of the diagonal cracks to be horizontal. Failure occurred due to re-entrant diagonal cracking with some crushing at the compression face of the extended end. This crushing increased the rate of penetration of diagonal cracking leading to early failure. Furthermore, it was observed that maximum deflection reduced slightly by 3% for specimen SNI1 whereas an increment of 12.3% for specimen SNH1 has been obtained.

6.5 Specimens SNH2 & SNI2;(a/d) =1.5

The same detailing for specimens SNH1 and SNI1 were adopted for SNH2 and SNI2 but the tests are achieved under (a/d) value of 1.5. The goal of testing such beams is to check the range of influence of this variable on the strengthening process and activity of this technique with a/d value larger than 1.0.The loading history and maps of crack propagation at failure are shown in Fig. 15 and Fig. 16 respectively.

It can be seen for specimen with horizontal configuration of strengthening, the initial crack developed vertically at reentrant corner at load level of (50 kN). With further loads, cracks spread around this this corner within the dapped ends. Failure occurred at load of (285kN) due to deboning of the horizontal bar followed suddenly by penetration of the vertical reentrant crack to cross most of the cross section. For specimen SNI2 with mixed (vertical + inclined) strengthening bars, it is obvious that the inclined bars restrict formation of cracks close to the reentrant corner. Thus, major cracks formed horizontally within the compression zone, this crack extend within the nib end beyond the support. The width of such cracks developed dramatically up to failure, which occurred at (295kN) due to horizontal crack within the extended end, it can be concluded that, for values of (a/d) greater than 1.0, the anchorage detailing of the strengthening bars has a dominant effect on determining the better method of either the nib (horizontal) strengthening or the mixed configuration. If sufficient anchorage is provided for the horizontal bars, then, the horizontal configuration still the better one. Otherwise, the mixed configuration yields higher performance. Comparing the load-deflection curve of these specimens against the specimen (RN02) as shown in the Fig. 13, it can be seen that the ultimate load capacity has improved by about (20% and 24%), respectively.

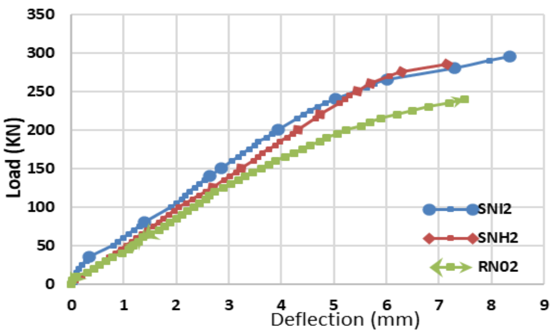


Fig. 15 Effect of Strengthening Configuration (at Nib Region) Against RN02

(a)specimen SNH2 *(b) specimen SNI2*

Fig. 16- Crack Patterns for Strengthening for specimens strengthened against main nib steel (a/d=1.5)

6.6 Specimens strengthening of the Hanger region (SHV1, SHF1)

Two configurations have been suggested to substitute the reduction of hanger reinforcement. The specimen (SHV1) was strengthened with two 2#10 vertical bars per side in hanger region while the other specimen (SHF1) was strengthened with 2#10 bars at angle 45º at both side in hanger region. Fig. 17 shows the load deflection-curve that obtained for the two strengthened specimens against the specimen (RH01). It can be seen that the specimen (SHV1) yielded stiffer response and load capacity (425kN) than the (SHF1) that yielded a load capacity (400kN) i.e. the increments in the failure load capacity were recorded by about (21%, 14.3%) respectively. Fig. 18 (a) & (b) show the crack patterns for the specimens SHV1 and SHF1 respectively. It can be seen that the vertical strengthening restricts the propagation of cracks close to the re-entrant corner at which the first crack occurred. Failure occurred due to the vertical cracking at the hanger side that penetrate diagonally towards the point of load application. Therefore, it is expected the cracks may be further delayed by shifting the strengthened bars in the direction of load application or making the strengthening bars as in C shape.

Fig. 18(a) shows the crack pattern for the specimen SHF1. It can be seen that adopting angle 45 may not substitute the lack in reinforcement in the bottom corner of the full depth beam, which may be separated as a single unit (secondary failure). Also, it is obvious that from Fig. 18(b) that failure didn't occur at the re-entrant corner, Thus, it is expected that the capacity can be enhanced using more number of steel bars with smaller sizes within the disturbed regions of the beam. Regarding deflection,It can be seen that the maximum deflection increased by 22% and 16.5% for specimens SHV1 and SHF1 respectively.

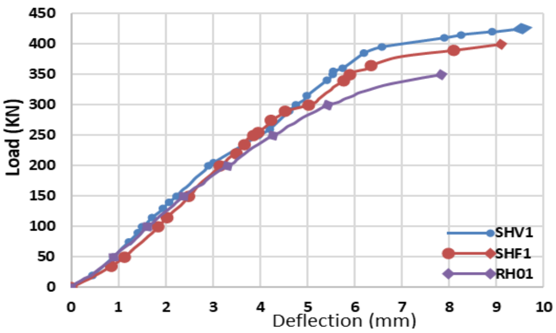


Fig. 17- Effect of Strengthening Configuration (at Hanger Region) Against RH01

(a)specimen SHV1 *(b) specimen SHF1*

Fig. 18- Crack Pattern for the Specimens (SHV1 & SHF1)

It can be seen from the previous discussion that all of the strengthened specimens in either nib or hanger zone, the failure is caused largely by slip of the strengthening bars due to the insufficient development length for such bars to develop the required bond strength with concrete. This can be seen at the final stages of load history curves for such specimens to be nearly flat. Thus, it is expected higher failure loads can be obtained with using hooks at ends of the strengthening bars. This aspect will be the subject of the next work.

6.7 Crack width for the strengthened specimens

The rate of development of the first cracking for the specimens of (a/d) =1.0 strengthened against nib deficiencies in reinforcement compared to the control specimen C001 are shown in Fig. 19. It can be observed clearly that the strengthened specimen SNH1 yielded less crack width relative to specimen C001. this may be attributed to the manner of distribution of the total area of the horizontal nib reinforcement in which it is 4#12 for specimen SNH1 corresponding to 2#16 for specimen C001. For specimen SNI1, the inclined arrangement added some tension strength and improved the shear resistance (bars are orthogonal with crack).thus, the crack width is the least compared with SNH1 and C001.

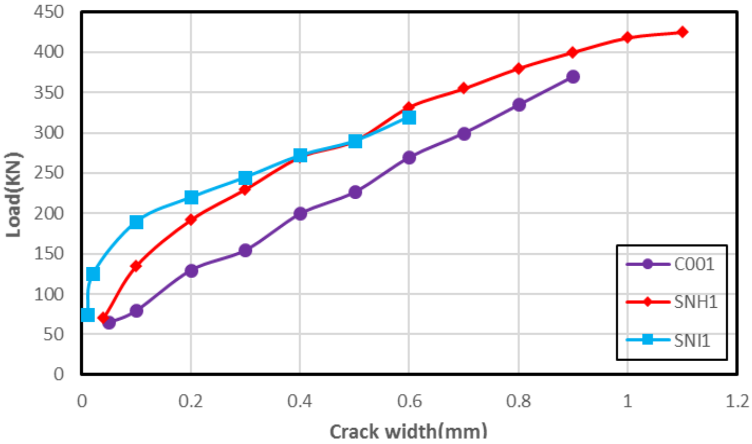


Fig. 19- Variation of Crack Widening For Specimens SNH1 and SNI1 against Specimen C001

Fig. 20 shows the rate of widening of cracking for specimens with (a/d)=1.0 and strengthened at hanger zone against the specimen C001. It can be seen that specimen SHV1, which strengthened with 2# 10 each side, produced lower rate of crack widening relative to C001.this may be attributed to the effect distribution of the total amount of hanger reinforcement and that the excess steel area served as steel fibers in delaying crack propagation. However, with progress in loading the rate of widening increased gradually. This may be caused by the slip occurred at final stages of loading. The lowest rate of crack widening can be seen for specimen SHF1, because that the added bars are orthogonal with the path of cracks. Thus, maximizes the shear strength of the section in addition to the efficient distribution of strengthening bars.

The effect of NSM strengthening bars for specimens included deficiencies in reinforcement at nib zone with a/d=1.5 is shown in Fig. 21. Results revealed that the control specimen C002 yielded the highest rate of crack widening whereas for the specimen SNH2 lower rate has been obtained. This may be due to the same reason discussed above which is using more number of bars with smaller diameter. For specimen SNI2, results confirmed that the inclined orientation of strengthening bars might result in good enhancement in performance in terms of rate of cracking and ultimate capacity.

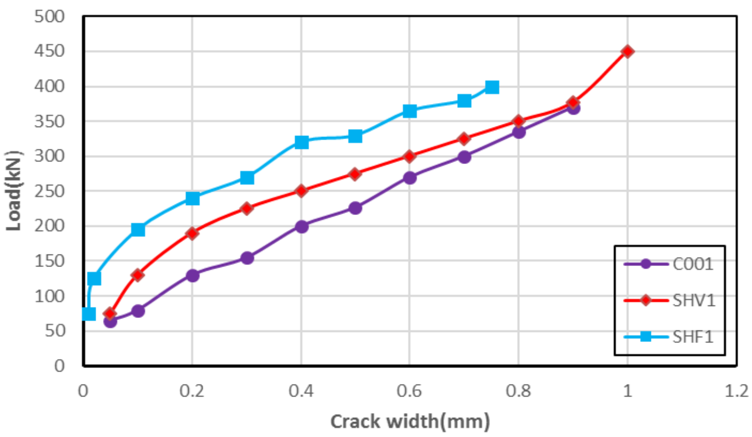


Fig. 20- Variation of Crack Widening For Specimens SHV1 and SHF1 against Specimen C001

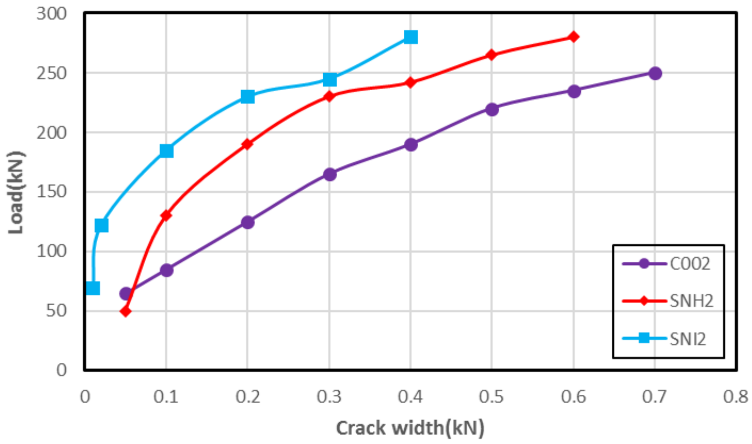


Fig. 21- Variation of Crack Widening For Specimens SNH2 and SNI2 against Specimen C002

7 Conclusions

It found that (a/d) ratio has a noticeable effect on behavior of dapped end beam. For the tested controlling samples (with design-reinforcement), adopting (a/d) ratio from (1.0) instead of (1.5),enhancement in failure load by about (48.4%) and shifting the mode of failure from "Diagonal tension in the extended end" to "Diagonal shear failure at the re-entrant corner accompanied with crushing in compression zone". Furthermore, it was observed that the reduction of nib reinforcement by (44%) resulted in reducing capacity by 11% and 5% for a/d=1 and a/d=1.5 respectively. In addition, the reduction of hanger reinforcements by about (33%), has decreased failure load by about (9%) in the (a/d) ratio (1.0). This may be attributed that the PCI method yields some overestimation when design the self-compacting high strength concrete.

Regarding the strengthening results, it is found that strengthening the nib region with NSM steel bars led to increase the failure load capacity by a range of (8% -28.7%) with (a/d) =1. While, the range of enhancement in strength for specimen with (a/d) =1.5 was about (19.7%-24%). The increase in the strength capacity for strengthened hanger regions vertical and inclined bars for a/d (1) was about (21.4% and 14.2%) respectively.

The present study revealed that using NSM strengthening technique by straight segments of steel bars within the disturbed zones of dapped ends has moderate significance if compared with bent bars or bars with hooks at end. This may be because of the lack in development length provided to develop bond with concrete.

Regarding rate of Crack widening, it is found that distribution the design steel area in more number of bars but with smaller diameter yield better performance and may produce relatively low rate of crack propagation. Furthermore, adopting strengthening configurations that include bars crossing the crack path with largest angle(close to 90°) might result in highest shear strength. Moreover, the increased rate of widening of cracking with progress in loading makes it is necessary to take the development length and providing efficient methods to produce the required both to reach yield point of the strengthening bars as using some bents or hooks at the bar ends.

Appendix A. Coding of the specimens

The general designation of a specimen is ABCD
A: either C: control, R: reduced, S: strengthened.
B: represent defect type; either N:nib end or H:hanger
    For Non-strengthened specimens B=0
C: type of arrangement of strengthening;
    For nib strengthening; H: horizontal; I: inclined by 45o+horizontal
    For hanger strengthening; V: vertical; T: inclined by 30o
    For Non-strengthened specimens C=0
D: represents the group; either 1 for a/d=1; or 2 for a/d=1.5

REFERENCES

[1]- S.K. Liem, Maximum Shear Strength of Dapped-end or Corbel. MSc Thesis, College of Engineering, University Concordia, Montreal, Quebec, Canada, August 1983

[2]- Precast Prestressed Concrete Institute. PCI Design Handbook: Precast and Prestressed Concrete 7th Ed. Chicago, IL., 2010.

[3]- A.M. Atta, T.F. Al-shafiey, Strengthening of RC dapped end beams under torsional moments. Mag. Concrete Res. 66(20) (2014)1065-1072. doi:10.1680/macr.14.00067

[4]- J.G. MacGregor, Reinforced concrete mechanics and design. 3rd Ed. Prentice-Hall, Upper Saddle River, N.J., 1997

[5]- A.H. Mattock, T.C. Chan, Design and Behavior of dapped End Beams. PCI J. 24(6) (1979) 28-45.

[6]- W.-Y. Lu, I.-J. Lin, S.-J. Hwang, Y.-H. Lin, Shear strength of high strength concrete dapped-end Beams. J. Chinese Inst. Eng. 26(5) (2003) 671-680. doi:10.1080/02533839.2003.9670820

[7]- S. Ahmad, A. Elahi, J. Hafeez, M. Fawad, Z. Ahsan, Evaluation of the Shear Strength of Dapped Ended Beam. Life Sci. J. 3(10) (2013) 1038-1044.

[8]- M. Aswin, B.S. Mohammed, M.S. Liew, Z.I. Syed, Shear Failure of RC Dapped-End Beams. Adv. Mater. Sci. Eng. (2015) 309135. doi:10.1155/2015/309135

[9]- R. Herzinger, M. Elbadry, Alternative Reinforcing Details in Dapped Ends of Precast Concrete Bridge Girders : Experimental Investigation. J. Transp. Res. Board 2028(1) (2007) 111-121. doi:10.3141/2028-13

[10]- K.M.P. So, The Behavior of Thin Stemmed Precast Prestressed Concrete Members with Dapped Ends. Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Canada, June 1989.

[11]- A. Nanni, P.C. Huang, Validation of an Alternative Reinforcing Detail for the Dapped Ends of Prestressed Double Tees. PCI J. 47(1) (2002) 38-45.

[12]- M.B. Forsyth, Behavior of Prestressed, Precast Concrete Thin-Stemmed Members with Dapped Ends. MSc thesis. Department of Civil, Construction and Environmental Engineering, North Carolina State University (NCSU), Raleigh, NC., 2013.

[13]- A.W. Botros, Behavior and Design of Dapped Ends of Prestressed Concrete Thin-stemmed Members. PhD Thesis, North Carolina State University, 2015.

[14]- A.W. Botros, G.J. Klein, G.W. Lucier, S.H. Rizkalla, P. Zia, Dapped Ends of Prestressed Concrete Thin-Stemmed Members : Part 1, experimental testing and behavior. PCI J. 62(2) (2017) 61-82.

[15]- P.C. Huang, J.J. Myers, A. Nanni, Dapped-End Strengthening in Precast Prestressed Concrete Double Tee Beams with FRP Composites. In: Prococeedings pf the 3rd International Conference on Advanced Composite Materials in Bridges and Structures, Ottawa, Canada, 2000, 545-552.

[16]- K.H. Tan, Shear Strengthening of Dapped Beams Using FRP Systems. In: FRPRCS-5: Fibre-reinforced plastics for reinforced concrete structures, Vol.1, 2001. pp. 249-258.

[17]- S.E.-D.M.F. Taher, External Strengthening of Critically Designed RC Girders with Highly Stressed Dapped-Ends. Concrete Struct. J., Institution of Civil Engineering, U. K., 2004.

[18]- P.C. Huang, Dapped-End Strengthening of Precast Concrete Double Tee Beams with FRP Composites. MSc thesis, University of Missouri-Rolla, Missouri, 2000.

[19]- T. Nagy-György, G. Sas, A.C. Dăescu, J.A.O. Barros, V. Stoian, Experimental and numerical assessment of the effectiveness of FRP-based strengthening configurations for dapped-end RC beams. Eng. Struct. 44(2012) 291–303. doi:10.1016/j.engstruct.2012.06.006

[20]- A.C. Dăescu, T. Nagy-György, B.G. Sas, J.A.O. Barros, C. Popescu, Numerical Assessment of Dapped Beam Ends Retrofitted with FRP Composites. In: Proceedings of the 11th International Symposium on Fiber Reinforced Polymer for Reinforced Concrete Structures / [ed] Joaquim Barros; José Sena-Cruz, Universidade do Minho , 2013.

[21]- G. Sas, C. Dăescu, C. Popescu, T. Nagy-György, Numerical optimization of strengthening disturbed regions of dapped-end beams using NSM and EBR CFRP. Compos. Part. B-Eng. 67(2014) 381-390. doi:10.1016/j.compositesb.2014.07.013

[22]- A. Atta, M. Taman, Innovative method for strengthening dapped-end beams using an external prestressing technique. Mater. Struct. 49(8) (2016) 3005–3019. doi:10.1617/s11527-015-0701-8.

[23]- Q.M. Shakir, B.B. Abd, Strengthening of Self-Consolidating High Strength RC Dapped Ends with CFRP Fabrics. J. Mater. Eng. Struct. 6(3) (2019) 359-374.
[24]- F. Al-Mahmoud, A. Castel, R. François, C. Tourneur, RC beams strengthened with NSM CFRP rods and modeling of peeling-off failure. Compos. Struct. 92(8) (2010) 1920–1930. doi:10.1016/j.compstruct.2010.01.002
[25]- Md.A. Hosen, M.Z. Jumaat, Kh.M. Ud Darain, M. Obaydullah, A.B.M. Saiful Islam, Flexural Strengthening of RC Beams with NSM Steel Bars. In: Proceedings of the International Conference on Food, Agriculture and Biology (FAB-2014), Kuala Lumpur, Malaysia, 2014
[26]- Q.M. Shakir, H.H.H. Kamonna, The Behavior of High Strength Self-Compacting Reinforced Concrete Corbels Strengthened with NSM Steel Bars. Int. J. Adv. Sci. Eng. Inf. Technol. 8(4) (2018) 1022-1028. doi:10.18517/ijaseit.8.4.5905
[27]- A.K.Q. Mohammad, M.F.K. Al-Shamaa, Experimental Study of Behavior of Reactive Powder Concrete Strengthening by NSM-CFRP Corbels. Civ. Eng. J. 4(5) (2018) 980-992. doi:10.28991/cej-0309150
[28]- Iraqi Specification No. 5, Portland Cement, Baghdad, 1984
[29]- Iraqi Specification No. 45, Natural Sources for Gravel that is Used in Concrete and Construction. Baghdad, 1984.
[30]- ASTM A 370-05 (2005), Standard Test Method and Definite on for Mechanical Testing of Steel Products. Annual Book of ASTM Standards, Vol.01.01, ASTM, Philadelphia, PA.
[31]- EFNARC, Specification and Guidelines for Self-Compacting Concrete. Association House, 99 West Street, Farnham, Surrey, London, UK, February 2002, 32pp.
[32]- ASTM C-494/C 494M –01, Standard Specification for Chemical Admixtures for Concrete. Annual Book of American Society for Testing and Materials, 2001.
[33]- Sika, Sikadur 30 LP. Technical data sheet, 2017 edition, Sika Corporation.
[34]- ACI Committee 237R-07, ACI 237R-07, Self-Consolidating Concrete. Farmington Hills, Michigan; American Concrete Institute. 2007.