



Journal of Materials and Engineering Structures

Research Paper

Strengthening of Deficient Drop-in-ends with Near Surface Mounted (NSM) Steel Bars under static and repeated loading

Qasim M. Shakir *, Ruqyia Alliwe

University of Kufa, Faculty of Engineering, Al-Najaf, Iraq

ARTICLE INFO

Article history:

Received : 25 February 2024

Revised : 28 June 2025

Accepted : 5 July 2025

Keywords:

Drop-in -ends

Repeated loading

Residual deflection

NSM steel bars

ABSTRACT

The present research work, presented the results of eight drop-in-ends specimens tested experimentally to investigate the efficiency of the NSM steel bars technique in strengthening the drop-in-ends including internal miss detailing, under static and repeated loading. Two values of shear span/depth (a/d) ratios was considered, namely are 1.0 and 1.5. Several variables were considered including (a/d) ratio, effect of deficient nib reinforcement (by about 40%) and the type of loading. The behaviour has been discussed in terms of cracking load, failure load, cracking pattern, load-deflection curve and failure mode. Results revealed that increasing a/d ratio from 1.0 to 1.5 yielded a reduction in capacity and the corresponding deflection by 33% and 5%. Moreover, capacity reduced by 11% and 5% for the two values of a/d respectively, when the nib steel reduced by (40%). Furthermore, it was found that the strengthening by NSM steel bars resulted in increasing capacity by 15% and 14% for the two a/d ratio respectively. Regarding the repeated loading tests, it was found that capacity and corresponding deflection reduced by 25% and 42% for $a/d = 1.0$ relative to the strengthened specimen and tested under static loading, while the respective values for $a/d = 1.5$ were 5% and 3%. In addition, it was obtained that the capacity was reduced by 16% when increasing a/d ratio from 1.0 to 1.5.

1 Introduction

In the modern world, precast buildings appeared just prior to World War I when the British city engineer John Alexander Brodie were patented in 1905 in Liverpool, England, for his invention of the process of construction of precast buildings. However, precast industry was not developed significantly and used in a large extent until the early of the 1950s [1]. beyond this stage, its applications increased globally due to the construction revolution that followed the World War II and its several merits represented by good quality control of the products, rapid production, cost aspects and being non affected by calamite conditions. The idea of precast concrete is based on casting the elements of the structure individually. Then, by a suitable linking method they are assembled in site to form the structure. The basic types of connections use corbel and beams with Dapped-end (DEB) [2]. With columns connected, the use of dapped ends produces a better fabrication, they are also used to reduce overall height of the structure and increase stability for load bearing support. They

* Corresponding author. Tel.: +964 7832460562.

E-mail address: qasimm.alabbasi@uokufa.edu.iq

are very commonly used in reinforced concrete structures such as precast buildings, parking structures and recently in precast strap footings as it furnishes greater lateral stability and minimize the floor height; also, they are frequently found in bridge girder [3] Fig.1 shows some of the applications of dapped ends. Dapped end beams can be defined as a beam with notched ends at the bottom corners. Thus, severe change in geometry results in stress concentration within the junction between the nib end and the full depth beam. Such an area of disruption and irregularity in the flow of stress is known as the "disturbed zone" (D-zone). In zones of discontinuity as in dapped-ends beams, corbels and around openings etc. The traditional methods of design (Bernoulli theory) [4, 5] are not efficient to treat such problems. Two of these are adopted widely in concrete structure which are the shear friction (PCI) method [2] and the Strut and Tie Model (STM) [6]. failure for the dapped ends are characterized to be as "flexural and axial tension, direct shear at the junction of the dap and main body of the member, Diagonal tension from the reentrant corner, Diagonal tension at the extended end or Diagonal tension at the undapped portion" [PCI handbook [2].

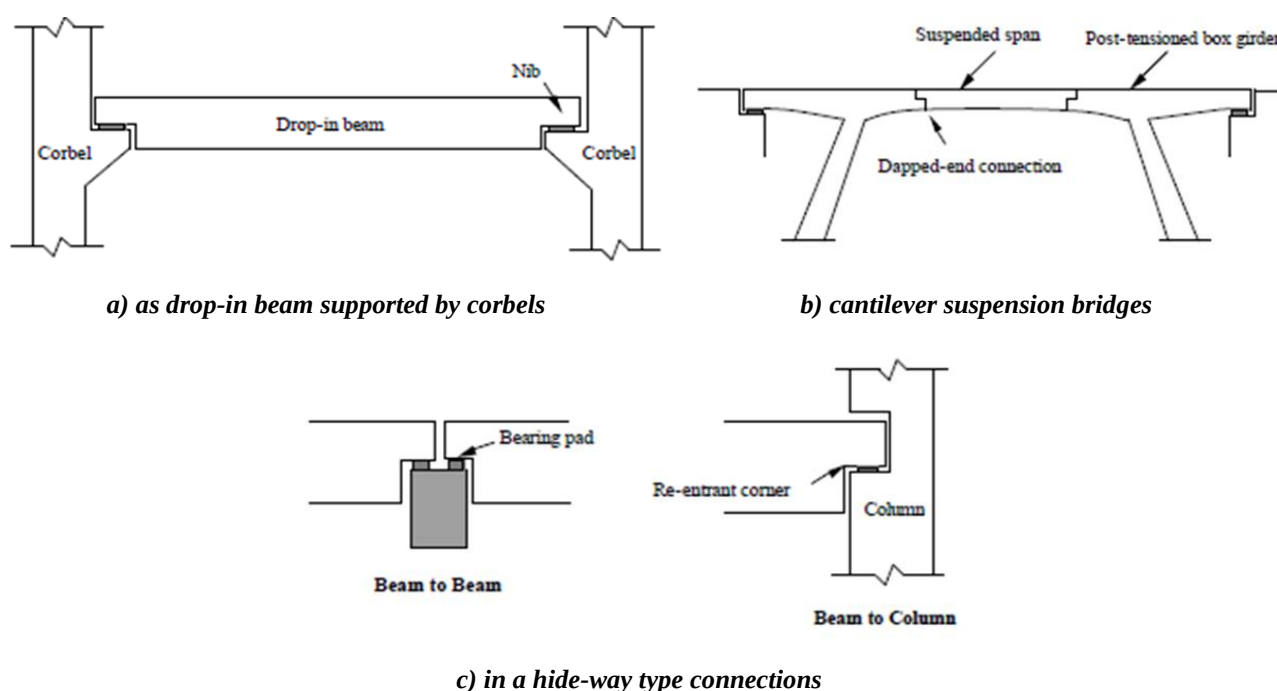


Fig. 1- Some applications for the Dapped End [7]

There are various circumstances in which upgrading dapped ends is necessary, some of these are [8]:

- 1- Error in calculating the forces of the virtual truss, which is the keystone of the STM models. This may result in errors in analysis results of the Struts and tie forces.
- 2- Errors in calculating the shear span (a) of the dapped end, shifting of the vertical forces or the vertical force may be accompanied by some non-predicted horizontal force.
- 3- Differences between methods and change in codes to be adopted may difference in designed steel at nib end and hanger zone.
- 4- Change in use form residential to industrial building or change in load values to be applied and its type (static, dynamic or cyclic).
- 5- Lack of knowledge of the behavior of the dapped ends.

Studies regarding strengthening of dapped (drop-in) ends can be categorized in two groups. Namely are internal strengthening and external strengthening. the internal strengthening is represented by the prestressing technique. Where as the external strengthening was presented by external bonding (EB) of FRP composites and the post-tensioned bolts. Regarding the prestressing method of strengthening Werner & Dilger [9] studied the behavior of prestressed dapped ends.. Mattock, and theyro [10] studied the behavior of thin stemmed prestressed concrete members with or without horizontal forces and developed a method of design for the required reinforcement. So [11] investigated the behaviour of thin stemmed

precast prestressed concrete members with dapped ends. Nanni and Huang [12] validated experimentally an alternative reinforcing detail for the dapped ends of prestressed double tees, which satisfies the requirements of the shear friction method adopted by PCI institute. Botros, et.al. [13] studied experimentally the behavior of dapped ends of prestressed concrete thin-stemmed members. Strands with Straight profile were used. Several reinforcement configurations for dapped ends were considered which are the vertical L- and Z-shapes, inclined L, welded-wire reinforcements, vertical C, and CZ forms. Masénas and Valivonis [14] studied the influence of prestressed strand placement in the dapped end and the intensity of prestressing. Proposed a method of external prestressing bars to strengthen the dapped-end beams. Mohammad and Abbas [15] considered the behavior of prestressed RC dapped beam with openings strengthened using CFRP sheets

Regarding the second category, using FRP composites has several merits as bendability for sheets, low weight, simplicity of installation, high tensile strength provided (especially by using CFRP sheets). However, the demerits are low shear strength, cost of composites and epoxy, low resistance to aggressive acids used to be considered. It should be noted that steel plates are rarely used in strengthening since that are difficult to bend, Fixing problems and low strength provided by thick bendable plates, being prone to corrosion.

Huang et.al. [16] focused on strengthening of precast prestressed concrete double tee beams with CFRP sheets. Huang and Antonio Nanni [7] investigated the efficiency of Strengthening of full-Scale Prestressed Double Tee Beams with FRP Composites Nagy-Gyorgy et al. [17] studied experimentally and numerically the adequacy of strengthening by CFRP sheets. Afefy et.al. [18] considered the rehabilitation of steeped beams by CFRP. Taher [19] conducted an experimental research regarding the external strengthening of RC girders with dapped-ends of internal deficiencies. Atta and Taman [20] proposed an external prestressing technique as a method for strengthening dapped-end beams. An experimental study has been conducted by Shakir and Abd [21], to scrutinize the response of reinforced self-compacting concrete half joints upgraded with CFRP strips. The study was extended by authors [22] to discuss effect of a/d ratio on stiffness and ductility of the dapped ends strengthened by CFRP sheets. Quadri and Fujiyama [23] considered the behavior of repaired RC dapped-end beams cyclic loading.

During the last decades, the NSM strengthening technique have been proposed. In this method, surface groove are introduced, filled partially with epoxy then FPR bars are embedded [24]. This method has some advantage compared to other techniques as eliminating buckling, environmental, and appearance aspects. This method was used with steel bars in strengthening RC beams by Hosen et.al. [25]. Shakir and Alliwe [26] explored experimentally the response of self-compacting RC drop in- beams retrofitted with NSM steel bars under short-term static loading. In 2021, Shakir and Alliwe [27], investigated the effect of a/d ratio of dapped ends strengthened by NSM steel bars on toughness, stiffness and the ductility. The posttensioned bolts has several difficulties related with need for special bolts and tools, experienced crew, enough space and affecting on the external appearance of the strengthened member.

As the previous review shows that non many researches take into account how internal deficiencies of reinforcement affect the performance of the dapped ends. Moreover, few studies adopted the NSM steel bars technique in strengthening the dapped ends. Furthermore, the effect of repeated loads on the dapped ends was not considered and investigated thoroughly such that the range of effect based on number of cycles of the repeated load would be determined.

The present study focused on the following issues:

- 1- Studying the effect of reducing the nib end on the general performance and studying efficiency of strengthening of deficient dapped ends,
- 2- The range in performance of dapped using NSM steel bars to substitute the missed steel bars is investigated.
- 3- Studying the response of the strengthened dapped end under the effect of repeated loads.

2 Experimental program

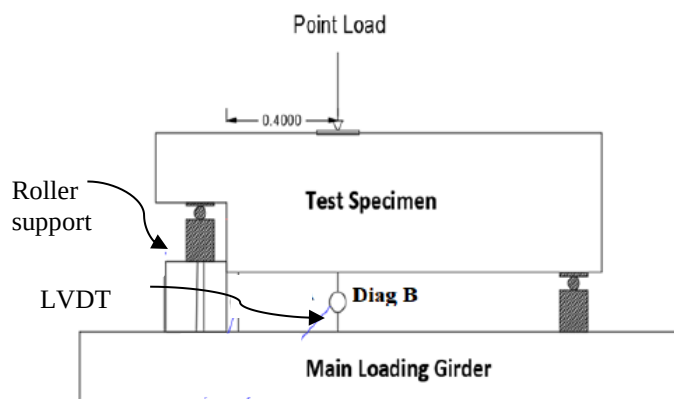
2.1 2.1 Description of specimens

In the present work, an experimental study has been conducted to investigate the behaviour of reinforced self-compacting concrete dapped end beams strengthened with NSM steel bars. The program consists of investigation the behaviour of 8 dapped end beams 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 span/effective depth of the nib (a/d) ratio are

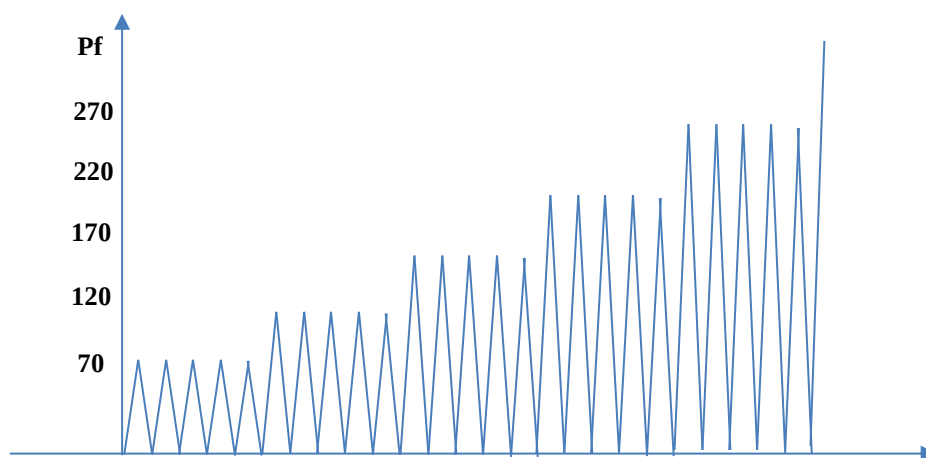
considered, namely 1.0 and 1.5. For each group, one specimen served as reference samples with full-reinforcement designed by PCI method and one specimen was intentionally casted with deficient reinforcement in the extended nib end. Two specimens were strengthened by horizontally aligned NSM steel bars; one was tested under static load, Fig. (2b). While, the other was tested under repeated load with regime shown in Fig.2 c. the NSM steel bars technique have been used to retire the lack in strength caused by these defects. For all the specimens, the concrete cover is taken as 40 mm to provide enough thickness for making the grooves.



(a) Details of Reinforcement for Control Beam



(b) Loading setup of the present work



(c) Regime of the repeated loading

Fig. 2- Details of Reinforcement and regimes of loading

2.2 Materials properties

The concrete mix constituents are Ordinary Portland Cement (OPC) Type I manufactured by KAR Company. The Chemical Composition of Cement (Type 1) are indicated in Fig. 3. Sand is considered as fine aggregate (FA) with maximum size of 4.75 mm that is taken from "Al-Ukhaidur" zone. Crushed gravel as coarse aggregate (CA) taken from Al-Niba'ee region having maximum size of 20 mm. Both fine and coarse aggregate are checked to satisfy the requirements of the Iraqi Standard Specifications NO.45/1984 [28]. Fig. 4 shows the distribution of grain size of the aggregate. Limestone Powder (LSP) which called locally as "Al-Gubra". Limestone powder (LS) as the filler and GILENIUM ®54 as superplasticizer (SP). Clean tap water (W) is used for mixing and curing. The mechanical properties for steel bars are listed in Table (1). Sikadur-30LP [29] has been used to fix steel bar segment used for strengthening within the grooves. Several trial mixes have been tested to produce a self-compacting mix with concrete cube compressive strength of 60MPa. Table (2) indicates the mix proportions of the mix adopted in the present study. Furthermore, Fig. 5 shows the material test of (SCC) fresh concrete mix, hardened concrete and steel bars.

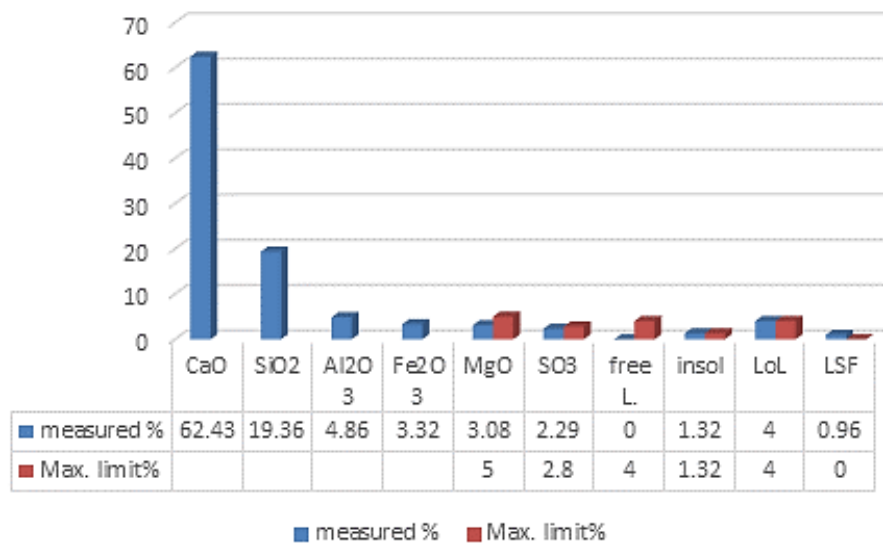


Fig. 3- Chemical Composition of Cement (Type 1)

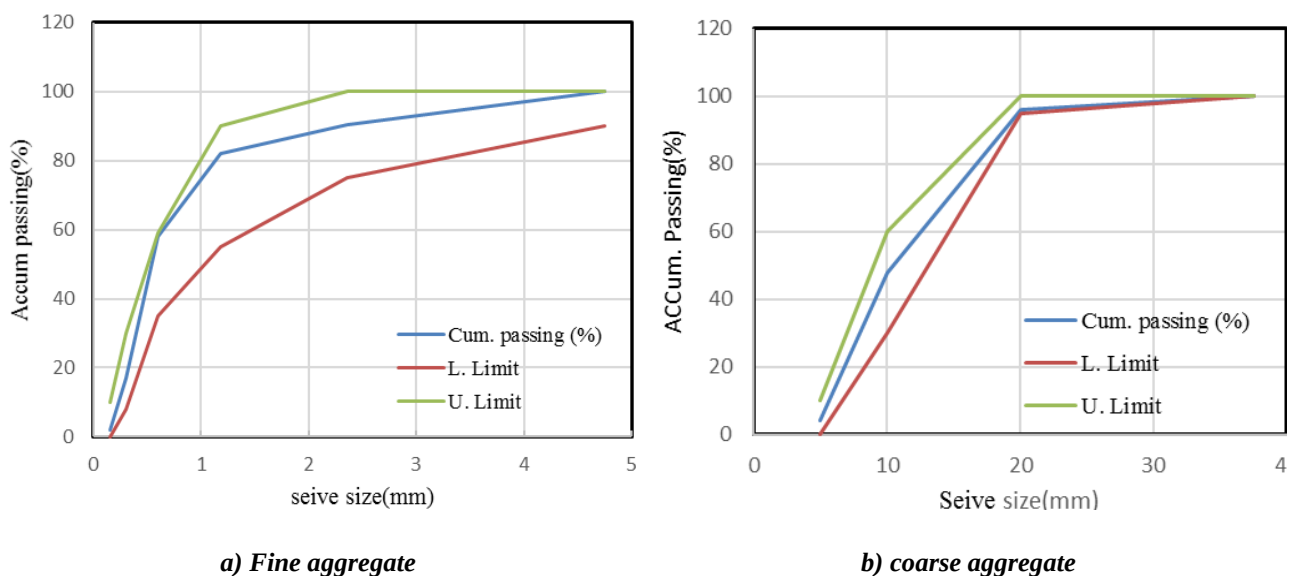


Fig. 4- Grain size analysis of aggregate

Table 1-Properties of Steel Bars [30]

Bar Diameter (mm)	Yield Stress* (MPa)	Ultimate Strength* (MPa)	Use
10	575	674	Stirrups, Hanger& strengthening
12	600	694	Red. Nib
16	550	665	Main steel & Nib

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

Table 2- Proportion of Concrete Mixes

Material	C	FA	CA	LP	W(L)	W/C	SP(L)
Quantity-kg (Relative)	400(1)	962(2.4)	780(1.95)	75(0.18)	125	31%	6

**(a) Slump flow test****(b) J-ring test****(c) compressive strength test****(d) Tensile splitting test****(e) tensile strength of steel****Fig. 5 –Material tests of fresh concrete mix, hardened concrete and steel bars**

2.3 NSM Steel Bars Strengthening System

The following procedure summarizes the NSM technique for strengthening with steel bars:

- 1- Creating slits or grooves in the concrete cover that are 1.5-2 times the diameter of the bar. In the present work, as the bar diameter is 10 mm, then the square section of grooves was (15-20) mm
- 2- Concrete powder or all particles are removed from the grooves using a high-pressure water jet cleaning method.
- 3- Prior to applying epoxy resin, make sure the grooves are completely dry before applying the first layer of epoxy.
- 4- Place the NSM rebar into the grooves that have been prepared, and then finish epoxy filling the groove.
- 5- Based on the recommendation of the manufacture, the specimen is tested seven to ten days after curing (dry whether+ room temperature). In the present work the test was done in during June inside the structural Lab., thus, ten day was enough to develop the strength reported by the manufacturer.

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. Table 3 demonstrate the details of the strengthening configurations for the tested specimens.

Table 3- 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
	SNH1-R	1#12 each side at nib reinforcement
Group-2 a/d= 1.5	C002	Full reinforcement
	RN02	The nib reinforcement is reduced by 44%
	SNH2	1#12 each side at nib reinforcement
	SNH2-R	1#12 each side at nib reinforcement

2.4 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 (100 mm x200 mmx10 mm) used to distribute the concentrated load and supports with the capacity of concrete and avoid the local failure at such regions. LVDT is positioned under the point load directly to measure the vertical deflection with progress of loading.



Fig. 6- Dapped end beam under test

3 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 behaviour 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 4.

Table 4- Ultimate Loads and Failure Modes for the Beam Specimens

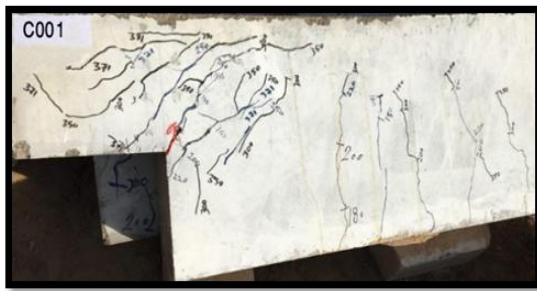
(a/d) Ratio	Specimen	Cracking load (kN)	Ultimate loads (kN)	Max. deflection (mm)	Failure Mode
Group-1 (a/d=1.0)	C001	65	371	8.60	Diagonal shear at the re-entrant corner
	RN01	60	330	8.10	Diagonal shear in the extended end
	SNH1	75	425	9.10	Slip + Diagonal shear failure in the extended end+ some crushing
	SNH1-R	70	320	5.31	Diagonal shear at the re-entrant corner
Group-2 (a/d=1.5)	C002	60	250	8.20	Diagonal shear at the extended end +Diagonal crack at the re-entrant corner
	RN02	65	238	7.50	Diagonal shear in the extended end
	SNH2	50	285	7.15	Slip + flexure failure at the junction
	SNH2-R	70	270	6.94	Diagonal shear in the extended end+Slip

3.1 Control Specimens (C001, C002)

The two specimens represent the control dapped ends that have been designed with full reinforcement using the Shear friction (PCI Handbook) method. It is aimed to observe the effect of a/d ratio on many indicators including cracking pattern, cracking load, failure load, mode of failure, load-deflection curves, toughness, ductility and stiffness. The two specimens are identical in detailing of the internal reinforcement and the difference is only in the shear span/ effective depth of the dapped end ratio. For specimen C001, the ratio was 1.0. While, for the specimen C002, the a/d ratio was 1.5.

It can be seen from Fig. 7 that the first crack initiated at the re-entrant corner within a load stage of 65 and developed diagonally with an angle of 45°. As the re-entrant corner represents the most critical point of the dapped end, severe disturbance in stress contour lines may occur at the location. This is due to the sudden change in the section combined with the effect of the high shear force at the dapped end reaction that acted as an obstacle that may result in a severe concentration of stresses around the re-entrant corner. It is to be mentioned that the value of angle of the diagonal crack is affected by several variables including the a/d ratio, detailing of steel reinforcement within the dapped end, inclusion of fibers within the concrete at the D-region, grade of concrete, etc. Further loading resulted in propagation of crack on both sides and around the nib end. Curvature increased leading to crushing at the compression face close to the top CCT nodal point. Such crushing restricts the development of the diagonal crack to the compression face. Consequentially a semi-plastic hinge is developed at the contact between the tensile and compression regions. 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 formation of this hinge resulted in a tendency to a rigid body rotation of the dapped end and the beam around this hinge. The value of the tensile forces that may be provided in the tie elements and the strength of the compressive struts play a significant role in determining the range of resistance.

As the bending moment/shear force ratio increased, i.e. higher a/d value was adopted, as in specimen C002, higher stresses may be induced in the tensile tie, or horizontal nib reinforcement. Consequently, with the same load, greater deflection can be recorded and some cracks can be started at the nodal point (CTT). Furthermore, the extended end acts as though it is hinged at the imaginary hinge. More curvature is therefore visible. As a result, the horizontal cracking continues to the nib's tip end. In contrast, specimen C001 exhibits a situation where shear may be predominant and the horizontal crack's ability to develop is limited. It should be noted that the first crack, which developed at a load of 60 kN, also started at the re-entrant corner. But with a smaller inclination angle. With further load, cracking penetrates diagonally, and the beam tends 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 to the re-entrant diagonal cracking combined with the diagonal crack developed at the extended end. It is expected that more capacity can be obtained when the compressive force is improved by using higher grade of concrete or using steel fibre concrete within the dapped end region.



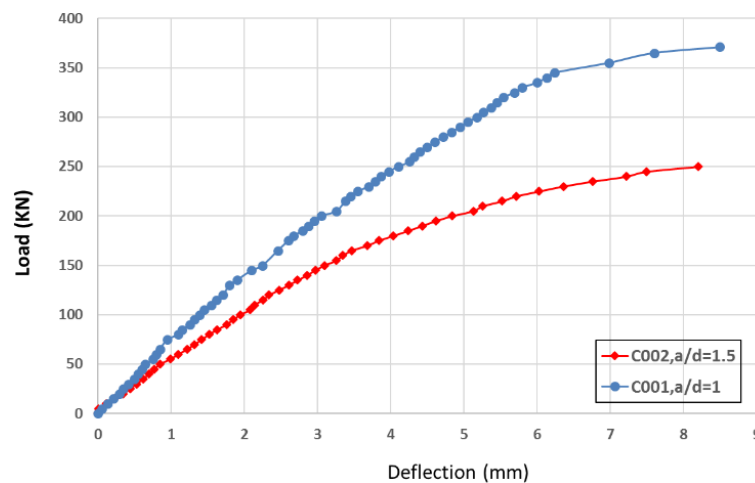
(a) Specimen C001



(b) Specimen C002

Fig. 7- Cracks Patterns at failure for the Control Specimen

Fig. 8 shows the load-deflection curves for the two control specimens. It can be observed that the specimen C001 yielded higher initial stiffness than the specimen C002. This may be attributed to the dominance of the bending moment relative to the shear force compared to the specimen with smaller a/d ratio. Then, lesser stiffness may be observed. Thus, cracking may be shifted to the nib end rather than the full-depth beam as in specimen C001. It can be observed that more ductile failure may be acquired for the specimen with higher values of a/d ratio. Because that, it is controlled mainly by the strength of the tie element which is represented by the main steel of the nib end. Regarding failure loads and corresponding deflections, results reveal that a reduction in capacity by 33% was recorded while a slight increase in maximum deflection (5%) was observed when increasing a/d value from 1.0 to 1.5.

**Fig. 8 Load-Deflection Curves for Specimens with Different (a/d) Ratio**

Regarding flexural toughness, which represented as the sum of areas under the load-deflection curve, Specimen C001 yielded toughness of 1950 kN.mm while specimen C002 yielded a toughness of 1222 kN.mm. i.e. an enhancement in toughness of 60% was acquired when the a/d ratio reduced from 1.5 to 1.0. Stiffness was affected also, by reducing the a/d ratio. In this work the effective stiffness was adopted and is represented by the slope of the secant to the service load which is considered as $0.75P_F$. Then, effective stiffness equal the ratio of $0.75P_F$ to the corresponding deflection. Results revealed that the effective stiffness of specimen C001 was 63 kN/mm while for specimen C002 was 45 kN/mm. This means that an enhancement of 40% was acquired when the dapped end become shear dominant rather than being bending moment. Regarding ductility which represents the margin in safety that a member may provide. Several proposals were suggested in literature [31-33]. In the present work, Energy based method [34] was adopted and may be calculated using eq. (1). The recorded ductility were 1.95 and 1.82 for the two specimens respectively with an enhancement of 7%.

$$D = \mu = 0.5 ((E_t / E_e) + 1) \quad (1)$$

In which E_t is the total energy dissipated up to failure; E_e is the energy that proposed to be dissipated within the elastic stage.

3.2 Specimens with Reduced Reinforcement (RN01 & RN02)

Fig. 9a shows the map of crack propagation of the specimen RN01. This specimen included some internal defect represented by the reduced 40% nib reinforcement. It can be observed that the first crack initiated at the re-entrant corner and developed diagonally with an angle of about (45°) . Further loading resulted in propagation of cracks around the re-entrant corner because it is the most critical point in the stressed D-region. The relatively small (a/d) ratio or small B.M. /S.F. ratio maintain rigidity of the dapped end without severe drop when reducing the nib reinforcement. Thus, cracking propagated within the body of the full-depth part away from the highly disturbed region towards the neighboring B-region.



(a) specimen RN01



(b) specimen RN02

Fig. 9- Cracks Patterns for the Specimens with Reduced Reinforcement at Nib region

The strength of the junction interface between the nib end and the full-depth beam reduced in a gradual trend as cracking penetrated throughout the depth of the beam. Due to the higher stiffness of the nib end, more stresses were accommodated. It can be observed that the nib end was sufficient in resisting the applied load and the full beam acted as a simply supported beam loaded by a mid-span concentrated force. This is clear from the vertical cracking directed towards the point load within the body of the beam. With progress in loading, concentration of stress around the critical corner increased resulted in widening of cracks and increasing of the curvature that resulted in initiation of spalling behavior of concrete within the final stage of loading. Regarding the nib end, more cracks developed towards the end of the beam and results revealed that the system of reinforcement of the nib end is sufficient to restrict sudden drop in capacity and then brittle failure to occur. Failure occurred beyond a gradual process of loss in energy (ductile failure) up to the instant of failure at a load level of 330kN following a mode of diagonal cracking at the re-entrant corner. It can be concluded that the design by the PCI method yielded a considerably conservative results (main nib steel) that the reduction in steel area has no significant effect on response to some limit.

For the specimen RN02 at which the same reduction in nib reinforcement was adopted, but tested under a/d ratio of 1.5, the cracking pattern at failure is shown in fig. 9b. It can be seen that the first crack initiated at the re-entrant corner, because it represents the most critical point of the highly disturbed region. However, the crack developed with an angle of 30° compared the specimen RN01 at which the angle was about 45° . This may be attributed to the effect of the B.M. that dominated the behavior. As in specimen RN01, cracking propagated around the re-entrant corner with progress in loading more than other regions. However, it can be seen that due to the weakness in the junction of the dapped end, cracking did not propagate in same rate away from the re-entrant corner. Another diagonal crack emanated from the dapped end support in addition to the diagonal crack from the re-entrant corner. Furthermore, it can be observed that most of cracks in the full-depth body are directed towards the applied force, this refers to the high tensile stresses induced normal to the re-entrant diagonal crack. While for specimen RN01, cracking directed vertically, due to effect of shear close to the ends and interaction of B.M. at mid span. Based on the previous discussions, it can be seen that the configurations of inclined nib reinforcement with several configuration discussed in [3], [10], [34–36] may be advantageous to reduce the weakness at the re-entrant corner. In these proposals, inclined reinforced that extended from the full-depth part diagonally to the nib end.

This reinforcement resists the tensile stress normal to the crack from re-entrant corner in addition to the horizontal steel at nib that resists the B.M. at junction interface. A modified configuration that may satisfy the strength and cost requirement is indicated in Fig. 10. This proposal assumes passing half of the main steel of the full-depth part diagonally to the nib end. While the other is aligned as usual with a smaller sizes (same area) to the CTT nodal point.

To provide hanger reinforcement, these bars are passed vertically to the CCT at top and bent in a manner to provide a suitable hook. the nib steel may be reduced to a value equivalent to the resistance provided by the horizontal component of the inclined steel. Consequently, smaller amount of steel is needed as hanger reinforcement and main reinforcement at the nib end.

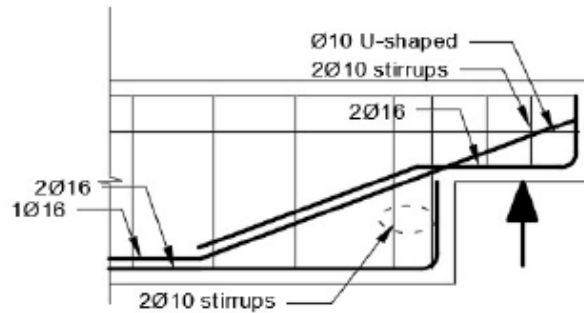


Fig. 10 The modified dapped end reinforcement with reduced nib steel [34, 35]

Fig.11a shows the load-deflection curve for the specimen RN01 against that for the corresponding control one (C001).it can be observed that reducing the nib steel by 40% resulted in a reduction of capacity by 11%. This may be attributed that the reduction in tension steel reduced the rigidity of the RC section and increased the flexibility. Then, the specimen behaved as it was tested under higher value of (a/d) but with same steel area of the nib. The dominance of the shear force reduced compared to C001.However, this reduction is less than that for the specimen RN02. i.e. the equivalent a/d ratio is in the range (1-1.5) or it is close to be of (1.25)

in other words:

$$(a/d)_{C001} < (a/d)_{RN01} < (a/d)_{C002}$$

it is to be mentioned that the maximum deflection also reduced by (5%) when reducing the value of nib steel and the reduction is in the range

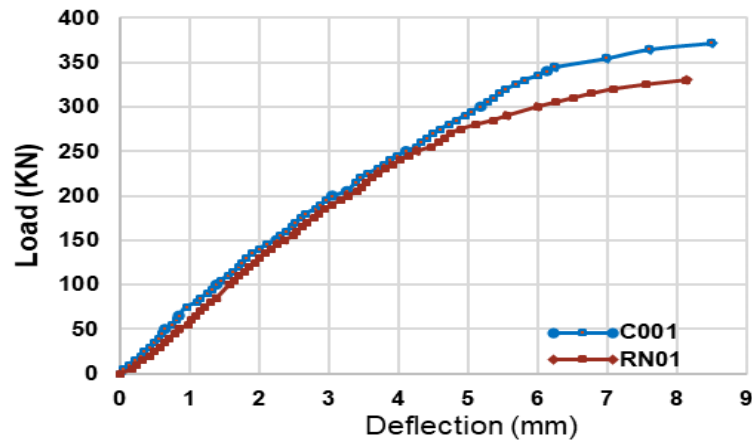
$$(\Delta_{max})_{C001} < (\Delta_{max})_{RN01} < (\Delta_{max})_{C002}$$

Regarding the flexural toughness, a value of 1622 kN.mm was obtained for the specimen RN01 which is less than that of the control dapped specimen (C001) by 17%. This value lies between the toughness of the control specimens C001 and C002, or:

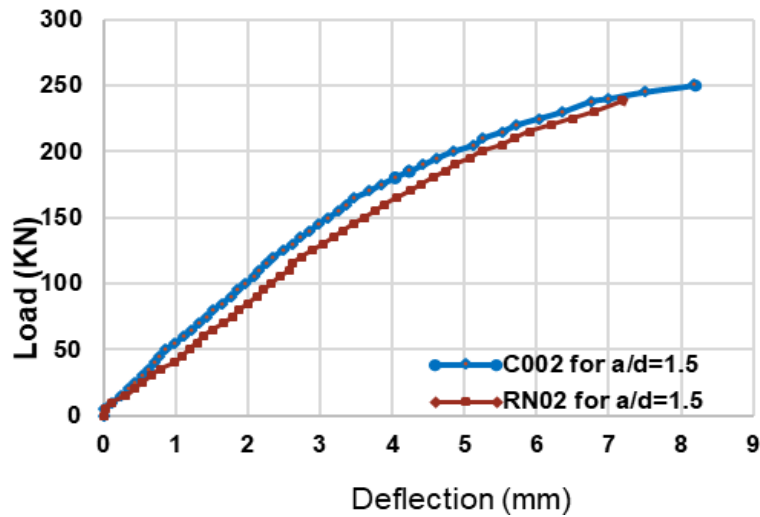
$$(T)_{C001} > (T)_{RN01} > (T)_{C002}$$

Regarding ductility, a value of 1.72 was recorded compared to 1.95 for specimen C001. Furthermore, the calculated effective stiffness was 56kN/mm while that for the control specimen C001 was 63kN/mm.

Fig. 11b shows the load-deflection curve for specimen RN02 against specimen C002. It can be noted that a slight reduction in capacity was recorded by 4.8% relative to the control specimen C002. Moreover, a reduction in the maximum deflection by 12% was recorded. It can be concluded that the response of the specimens with high values of (a/d) ratio is sensitive to any internal defects of miss-detailing in the nib reinforcement. Thus, the use of the inclined configuration may reduce this effect by increasing the efficiency of steel to resist direct tensile stresses normal to the crack emanating from the re-entrant corner.



a) Load deflection curve of RNO1 against C001



b) Load deflection curve of RNO2 against C002

Fig. 11- Effect of Reduction Nib Reinforcement with Different values of (a/d) Ratio

Regarding toughness, results revealed that calculated value was 901 kN.mm compared to a value of 1222 kN.mm for the specimen C002. Moreover, Ductility recorded a value of 1.50 compared to a value of 1.82 for the control specimen. Furthermore, the calculated value for effective stiffness was 42kN/mm compared to that of the specimen C002, which was 45kN/mm.

It can be concluded that the variation in capacity, maximum deflection and toughness for the specimens with reduced nib steel and the corresponding control one can be expressed as follows:

Failure load:

$$(P_u)_{C001} > (P_u)_{RN01} > (P_u)_{C002} > (P_u)_{RN02}$$

Maximum deflection:

$$(\Delta u)_{C001} > (\Delta u)_{RN01} > (\Delta u)_{C002} > (\Delta u)_{RN02}$$

Flexural toughness:

$$(T)_{C001} > (T)_{RN01} > (T)_{C002} > (T)_{RN02}$$

3.3 Specimen SNH1 and SNH2:

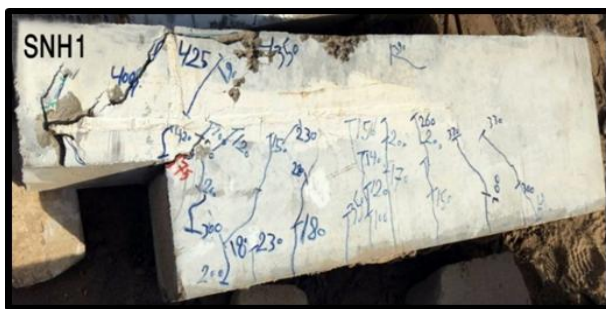
The crack pattern for specimen SNH1, Fig. 12a reveals that the first crack initiated at the re-entrant corner within a load level of 75 kN. However, the propagation of the horizontal cracking at the top fibre is interrupted by the vertical

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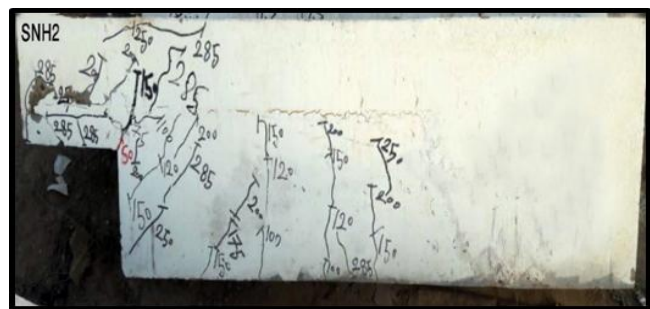
welded bar segment. Furthermore, the formation of the fictitious hinge and then the rotation about it is combatted by the additional tension force provided by the NSM steel bars. It can be observed that the horizontal cracks at the compression face occurred within a thin concrete layer. At load of 400 kN, bond failure occurred resulting in rapid extension of cracks followed by a diagonal path to join the horizontal crack developed at the compression face. Full collapse occurred at load level of 425kN.

Regarding the beam portion, it can be seen that more cracks developed close to the anchorage of the main steel of the beam(nodal point CTT) relative to specimen C001 and RN01, referring to the high strength developed within the half-joint region. No cracks extended above the embedded horizontal bars. It is expected that if the bond failure is avoided by using suitable anchors or welding the bars to plate fixed on face of nib end, higher capacity may be obtained.

The crack pattern for the specimen SNH2 is shown in Fig.12b. The first crack initiated at the re-entrant corner with load of 50 kN and angle of 60°. The addition of the horizontal NSM bars restricted the formation the fictitious hinge to some extent and the rigid body rotation of the two portions. It can be noticed that due to the existence of tension ties, the two parts acted as one unit within service loading. Thus, cracks propagated from the nodal point (CTT) and developed towards the zone of maximum moments. At load level of 250 kN, horizontal cracks propagated at the compression face. At load level of 285 kN, Bond failure occurred between the steel bars and epoxy. At this instant, the resistance to rotation about the fictitious hinge dropped leading to formation of a new vertical crack at the re-entrant corner that penetrated rapidly caused violation of equilibrium and causing full collapse of the specimen. The load-deflection curves for the two specimens are included in Fig. 17 and 18 respectively for the sake of comparison with other specimens in the group.



(a) Specimen SNH1

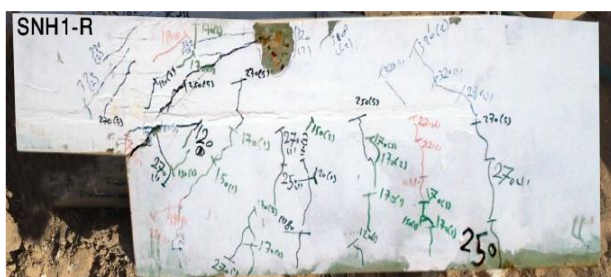


(b) Specimen SNH2

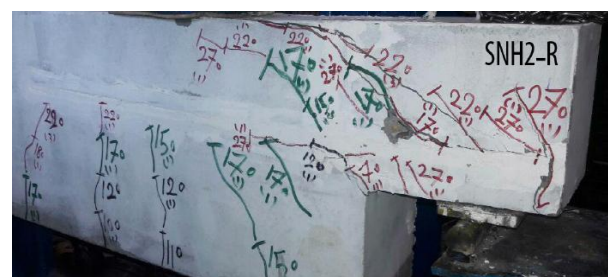
Fig. 12- Cracks Patterns for the Specimens with SNH1 and SNH2

3.4 Specimen SNH1-R and SNH2-R:

The aims of testing the two specimens are to observe the effect of the type of loading on the response under two different values of a/d ratio. Fig. 13 shows the cracking patterns of the two specimens under the successive stages of loading. For more understanding the propagation of cracking, Fig. 14 shows the cracking development at each stage individually. Moreover, Table 5 shows detailed descriptions to the behaviour of the two specimens within each stage (5 cycle) of loading.



a) Specimen SNH1-R



b) Specimen SNH2-R

Fig. 13- Cracks Patterns for the Specimen with Reduced Reinforcement at Hanger

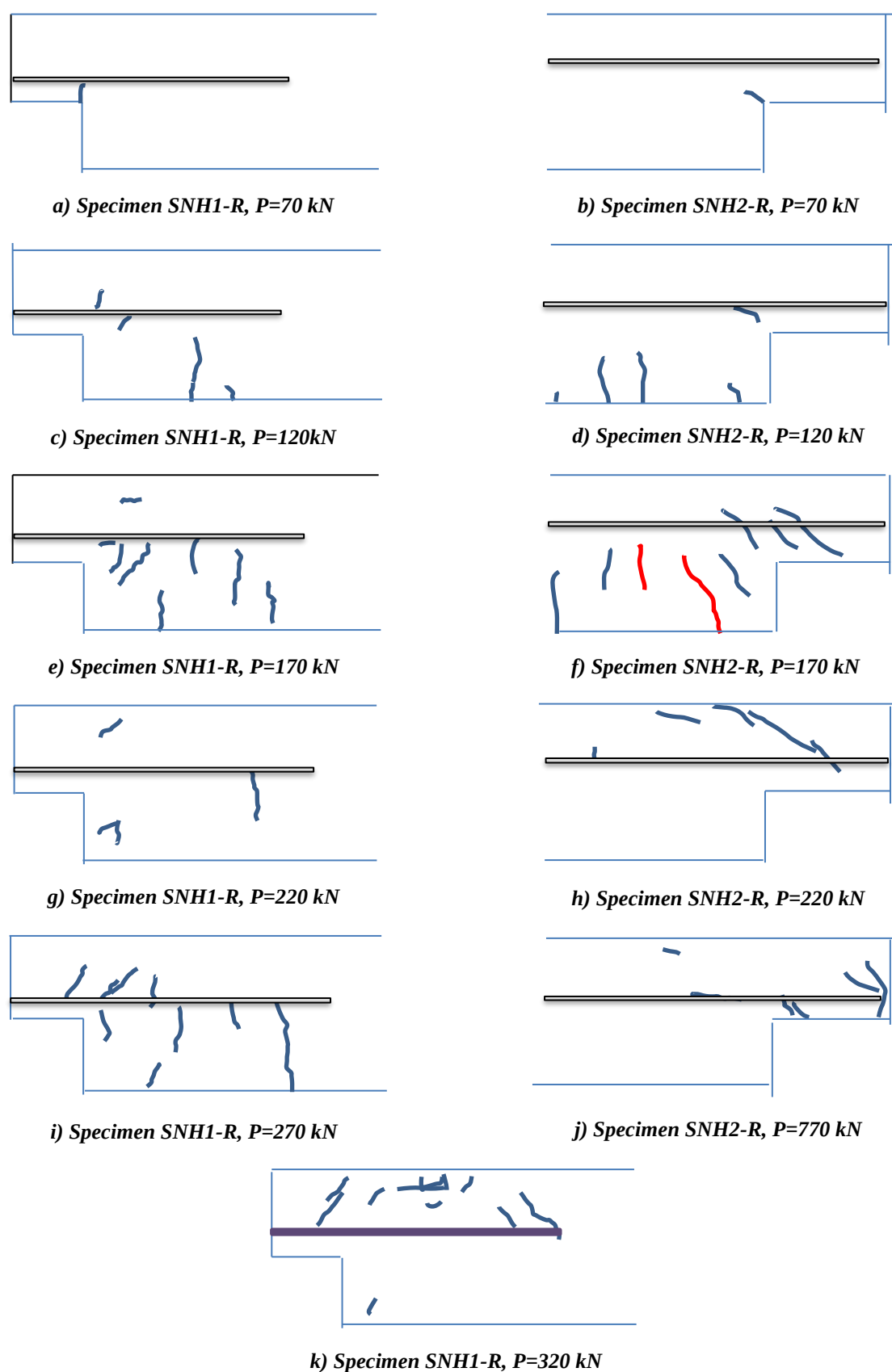


Fig.14-Crack development at the end of each stage of the repeated loading

From table 5, it can be concluded that the shear is dominant in specimen SNH1-R which is represented by the development of crack in a large angle. While, for the specimen with high a/d ratio at which the B.M. is dominant and resulted in making the angle of diagonal crack to be small ($<30^\circ$). Moreover, it can be seen that more cracks in specimen SNH1-R developed around the re-entrant corner while crack in the other specimen was recognized to occur within the nib end. Some diagonally emanating from the support the extend beyond the NSM steel bars. Furthermore, specimen SNH1-R, did not show large curvature which is clear from the cracks around the NSM steel bars. While, in specimen SNH2-R, several cracks developed horizontally around the NSM bars and some spalling behavior was recognized. The specimen SNH1-R failed by diagonal crack from the re-entrant corner after five stages of repeated loading followed by a stage of a monotonic loading up to failure. While, specimen SNH2-R failed by diagonal cracking at the nib end after four stages of repeated loading followed by a stage of monotonic loading up to failure.

Table 5 - Detailed descriptions for the behavior of specimens SNH1-R and SNH2-R

Stage	Specimen SNH1-R	Specimen SNH2-R
70	First crack developed from the re-entrant corner in steep angle ($>60^\circ$). Fig (15a)	First crack developed from the re-entrant in a small angle ($<30^\circ$). Fig (15b)
120	- The first crack developed up across the NSM bars. - Some cracks developed vertically within the D-region. Fig (15c)	- The first crack developed along the NSM steel bars. - Some cracks developed close to the bottom CTT node, others within the full-depth part. Fig (15d)
170	- More cracks developed from the re-entrant corner. - The vertical cracks in stage above developed towards the NSM bars. Fig (15e)	- Several cracks developed diagonally at the nib end, others developed from the bottom CTT node. - Some cracks developed near the end of NSM bars within the D-region. Fig (15f)
220	- Some stabilization of intensity of cracking occurred with some development to the top compression face. Fig (15g)	- The cracks in stage above developed to the loading point, cracks widened considerably. - Some spalling behavior was noticed. Fig (15h)
270	- More crack developed around the NSM steel bars. - More cracks developed out of the D-region. Fig (15i)	(Static loading up to failure) - More cracks developed at the nib end. - Diagonal cracks extended beyond the end of the NSM bars to develop the diagonal cracking failure - Some slip occurred. Fig (15j)
320	(Static loading up to failure) The diagonal crack from the re-entrant corner widened, curvature increased and some crushing at the compression face occurred. Fig (15k)	

It can be concluded that adopting small values of a/d ratio may yield optimum results when testing the strengthened dapped ends under repeated loads. If high values are to be adopted (but less than 2.0), the bond between the NSM bars and concrete should be improved by using welded steel plates at ends or suitable hooks. Also, using steel fiber concrete within the dapped end (D-region) may improve the behavior by reducing the rate of cracking propagation and better distribution and transfer of stress throughout the body of concrete may be obtained. Moreover, the disturbance of stress contour lines may be reduced. Consequently, the concentration of cracks around the NSM steel bars may be controlled and the bond with concrete may be maintained to large extent on the loading regime.

Fig.16 shows the envelope curves for the specimens SNH1-R and SNH2-R. It can be observed that increasing the a/d ratio resulted in significant increase in the recorded deflection and residual deflection at the end of each stage of loading. Values of 50%, 76%, 65%, 60% are recorded. This may be attributed to the high rate of cracking propagation for the specimen with high a/d ratio that resulted in higher values of the accumulated permanent deformation. Results also revealed that higher a/d ratio resulted in flatter envelope compared to the case with small a/d ratio, which may be attributed to the bond-slip effect that developed within specimen SNH2-R. It can be concluded that the interaction effect between the B.M. and shear may control the dissipation in potential energy for the upgraded dapped end. otherwise, a suitable means should be used to control the slip of NSM bars by welded cap plates, hooks, CFRP jacketing, Welded steel plate jacketing, combination of horizontal and inclined bars, etc.

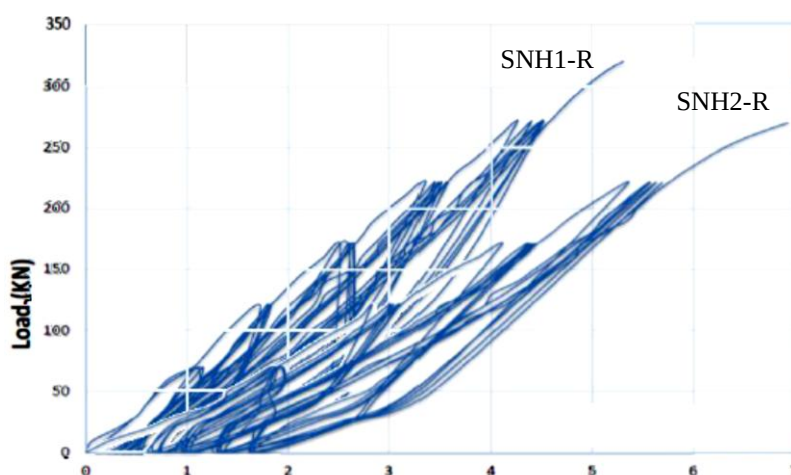


Fig. 16- Load-Deflection Curves of Specimen (SNH1-R) against (SNH2-R)

Fig. 17 shows comparisons between the load-deflection for the tested specimens in the first group ($a/d=1.0$). It can be seen that the strengthened dapped end (SNH1) yielded higher failure capacity than the control specimen (C001). This may be attributed to better resistance to stresses when the main steel was spread with the concrete rather than being concentrating in specific locations (in few bars). Moreover, the bond resistance to slip increased with increasing number of bars. This is attributed to the fact that using 4#12 bars for the main nib steel rather than 2#16 bars increased the contact area by 50%.

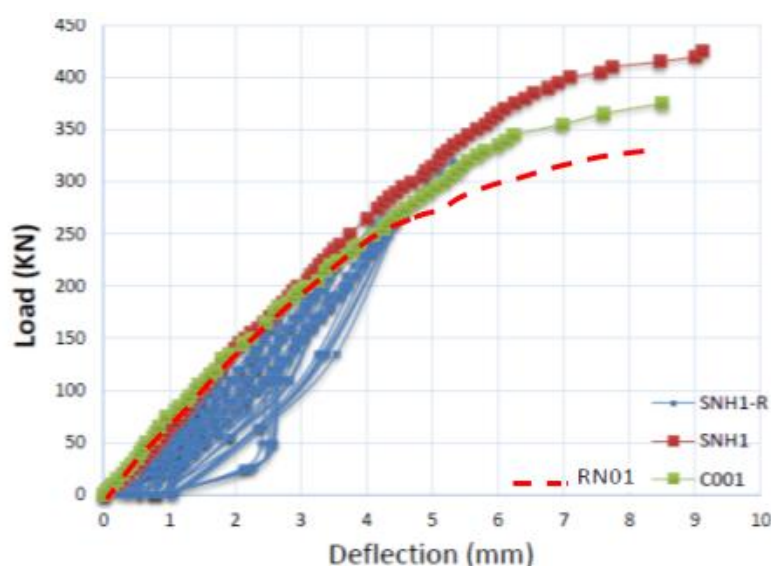


Fig. 17 - Load deflection curve of Group 1 ($a/d=1.0$)

Results also, shows that the repeated loading and number of cycles have significant effect on response. Reduction in capacity and deflection of 25% and 42% are observed compared to specimen SNH1. This reduction may be attributed to the residual stresses and displacement which increased with the successive stages of loading and that each stage started with smaller stiffness than the preceding one. It is to be mentioned that the effect of fatigue is negligible due to the few number of cycles (25 cycles).

Fig. 18 shows the load-deflection curves for the specimens with ($a/d=1.5$) tested under static loading against that tested under repeated loading. It can be seen that the strengthened specimen under the static loading yield higher capacity than the corresponding control specimen (C002). This may be attributed to the same reason above, related with the contact area of the main steel of the nib end. Furthermore, it can be observed that testing the upgraded specimen under repeated loading (4 stages only) resulted in reduction in capacity by 5% and the corresponding deflection by 3%.

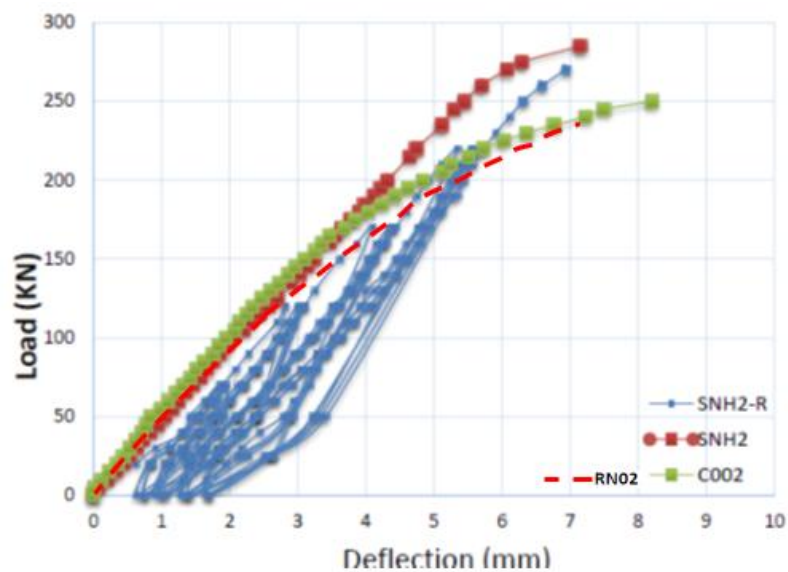


Fig. 18- load deflection curve of Group 2 ($a/d=1.5$)

LIST OF NOTATIONS

B-zone: Bernoulli zone (region)

CFRP: Carbon fibre reinforced polymers.

D: Ductility

D-zone: Disturbed zone (region)

DEB: Dapped end beam

EB: Externally bonded

Et: Total energy

Ee: Elastic energy

FRP: Fiber reinforced polymers

NSM: Near surface mounted

PCI: Precast Concrete Institute

Pu; ultimate capacity

SCC: self-compacting concrete

T: Toughness

Δ : Deflection

CODING OF THE SPECIMENS

The general designation of a specimen is ABCD

A: either C: control, R: reduced, S: strengthened.

B: represent defect type; N:nib end

For Non-strengthened specimens B=0

C: type of arrangement of strengthening;

For nib strengthening; H: horizontal;

or Non-strengthened specimens C=0

D: represents the group; either 1 for $a/d=1$; or 2 for $a/d=1.5$

4 Conclusions

It was discovered that the dapped end beam's behavior is significantly impacted by the (a/d) ratio. Adopting a (a/d) ratio of 1.0 instead of 1.5, increasing the failure load by approximately 48.4%, and changing 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. Additionally, it was noted that for $a/d=1$ and $a/d=1.5$, respectively, a reduction in nib reinforcement of about 40% led to a reduction in capacity of 11% and 5%. This could be explained by the fact that when designing self-compacting, high-strength concrete, the PCI method tends to be conservative.

With respect to the strengthening outcomes, it is discovered that using NSM steel bars to strengthen the nib region increased the failure load capacity by 15% with $(a/d) = 1$. Conversely, the specimen with $(a/d) = 1.5$ had a strength enhancement range of roughly 14%. In comparison to bent bars or bars with hooks at the end, the results of this study indicate that the use of the NSM strengthening technique on straight segments of steel bars within the disturbed zones of dapped ends has a moderate significance. This might be as a result of the short development period for bond development with concrete.

It has been discovered that spreading out the design steel area across a larger number of bars with a smaller diameter results in better performance and may lead to a relatively low rate of crack propagation. Moreover, the highest shear strength may be obtained by using strengthening configurations with bars that cross the crack path at the largest angle—close to 90 degrees. Furthermore, because of the faster rate at which cracks widen as loading increases, it is vital to consider the development length and provide effective ways to generate the necessary both to reach the strengthening bars' yield point, such as by using bents or hooks at the ends of the bars. More configurations of NSM strengthening may be explored under repeated loading. The efficiency of NSM steel bars technique may be examined under more cycles of repeated loads to simulate the fatigue effect.

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