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Experimental Evaluation of Flexural Strength and Ductility of One-Way Concrete Slab Panels Reinforced with Welded Wires and Deformed Bars

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Abstract. *This study evaluates the flexural strength and ductility of one-way concrete slab panels reinforced with deformed bar and welded wire under the application of pure bending. An experimental database of flexural strength and ductility for a set of three slab panels reinforced with 10 mm deformed bar, three panels with 6 mm welded wire reinforcement (WWR), and three panels with 8 mm WWR has been developed, with each slab panel having a dimension of 762 mm by 2286 mm. The deflection and ductility factors of the specimens are investigated. The results show that the slab panels reinforced with 6 mm WWR exhibit less vertical deflection at fracture in comparison to those with 8 mm WWR. Welded wire reinforced slab panels demonstrated uniformly distributed crack propagation in comparison to deformed bars. In addition, the slab panels with 8 mm WWR exhibit higher flexural strength than the 6 mm WWR reinforced panels. The 10 mm deformed bar-reinforced slab panels exhibited greater deflection at fracture compared to WWR specimens. The reason for the lower ductile behavior of slab panels with 6 mm WWR is due to the fact that 6 mm WWR, produced locally in Bangladesh, is manufactured by the cold-drawn method and has a lower ductility in compliance with BDS ISO 6935 Class A, which does not conform to ASTM A1064. Both 8 mm WWR and 10 mm deformed bars conform to BDS ISO 6935 Class D and ASTM A1064. Hence, the 6 mm WWR with Class A ductility is not recommended for reinforced concrete (RC) slab panels based on the experimental results conducted in this study, whereas the 8 mm WWR and the 10 mm deformed bar with Class D ductility are suitable for structural use as recommended in ACI 318.*

Keywords: *Welded Wire Reinforcement, Ductility, Flexural Strength, One-way Slab Panels.*

1 Introduction

Welded Wire Reinforcement (WWR) is pre-manufactured reinforcing steel constructed of high-strength, cold-drawn or cold-rolled wire that is welded together in square or rectangular grids (ASTM A1064:2017). Each wire intersection is electrically resistance-welded by a continuous automatic welder. Pressure and heat fuse the intersecting wires into a homogeneous section and fix all wires in their proper positions. Plain wires, deformed wires, or a combination of both may be used in WWR (WRI 2016).

Welded wire reinforcement (WWR) was first used in 1908 in road pavement construction (WRI 2014). After World War II, WWR was extensively used in building construction in Europe because it required less labor and time to place compared to conventional reinforcing bars. Currently, WWR is widely used in various types of structures, such as commercial and residential buildings, parking structures, highways, bridges, airports, walls and barriers, and tunnels, due to its cost-effectiveness and short placement time (Maguire et al., 2013). With outstanding economic growth in Bangladesh, numerous development projects are being undertaken in both the public and private sectors. The country needs the adoption of cutting-edge technology in order to reduce construction costs and time. One such addition would be the incorporation of WWR into RC infrastructure, which provides a variety of advantages. There are numerous benefits to using WWR instead of traditional mild steel reinforcing bars. WWR has a higher yield strength of up to 550 MPa (80 ksi) than traditional rebars. And so, the required amount of steel can be reduced by approximately 33% by using WWR. Use of WWR in concrete slabs ensures labor safety, optimizes construction time, and reduces cost significantly since there is no cutting of bars, no marking and spacing them out, and above all, no laborious tying of binding wires. (WRI 2016).

One of the drawbacks of using WWR is strain localization. The phenomenon due to which the local curvature on a cracked section at the ultimate moment, precisely prior to the fracture of steel, is much larger than the curvature on adjacent sections between the cracks is called strain localization (Gilbert and Smith 2006). Experiments have shown that WWR bonds perfectly with concrete, which eventually facilitates strain localization. The plastic deformation of the steel reinforcement was discovered to be confined to a relatively short length of reinforcing bar near the critical crack section. As a result, the critical portions have an extremely low rotational capacity, and the deflection right before the reinforced concrete section fractures is very minimal (Tuladhar and Lancini 2014). Furthermore, strain localization is prominent in higher-grade and excellently bonded welded wire reinforcements, especially in the case of small diameter wires (Foster and Kilpatrick 2008).

Due to the cold-drawing process, WWR has less ductility than traditional bars. The ductility and tensile strength of the wires are also reduced as they are welded together to produce WWR. This reduction in ductility makes WWR susceptible to brittleness and catastrophic failure. Material characteristics change considerably due to welding, which is the reason behind this reduction in ductility. According to Mo and Kuo, a metallurgical notch is created by the heat of welding. As a result, the stress-strain curve's plastic range is reduced, and WWR becomes susceptible to brittle failure. Annealing helps to reduce the negative effect of the weld and increase ductility by 35%–45% (Mo and Kuo 1995). Cold-drawn wires and WWR formed from cold-drawn wires are included in low-ductility reinforcement (known as Class A) (BDS-ISO-2016). High ductility reinforcement (Class D) includes hot-rolled deformed bars. Class D and Class A reinforcements have minimum specified characteristic values of elongation of 2% and 8%, respectively, and minimum tensile strength to yield stress ratios of 1.02 and 1.25, respectively. Regarding ductility, serious design implications have to be considered while using Class A.

According to the American Concrete Institute's Committee 318 (ACI 318-19 Table 20.2.2.4), WWR is allowed in members resisting flexure, axial, shear, and torsional stress in all structural applications, excluding members composing special seismic resisting systems. Due to its high ductility, ACI 318-19 only authorizes the use of low-alloy Grade 60 steel for the latter. There have been safety concerns raised about the use of WWR as tension reinforcement in concrete construction. The expected mode of failure for low-ductility tensile reinforcement is the sudden fracture of the longitudinal tensile steel, which has been experimentally verified by Eligehausen and Fabritius (1993) for continuous slabs. Gilbert and Smith (2006), Gilbert and Sakka (2007), Foster and Kilpatrick (2008), and Tuladhar and Lancini (2014) expressed their concerns about using WWR due to its' low ductility. As a result, the flexural strength of WWR-reinforced members in the Australian code (AS3600-2009) was reduced by 20%. Researchers Gilbert and Smith have also discovered that the concrete cracks occur at or close to the cross-weld specimens due to the strain localization phenomenon.

The main goal of this study is to examine the ductility, flexural performance, and mode of failure of concrete slabs reinforced with 6 mm WWR, 8 mm WWR, and 10 mm deformed. In this paper, a simple experimental program consisting of nine simply supported one-way slab specimens was executed to investigate the wire diameter and ductility class.

2 Experiments

This study carried out flexural strength tests on nine simply supported, one-way concrete slabs with two-point loading. The slabs were 762 mm by 2286 mm and 114 mm thick, as illustrated in Fig. 1. Two major parameters were investigated: wire diameter and ductility class. The effects of these factors on strength, ductility, and failure mode were also evaluated. Other characteristics were unintentionally changed as a result of the concrete and wire suppliers. The test matrix is summarized in Table 1, which includes the kind of reinforcement (WWR or rebar), lapping, cross-weld spacing, and longitudinal reinforcement area. The WWR specimens are exhibited in Fig. 2.

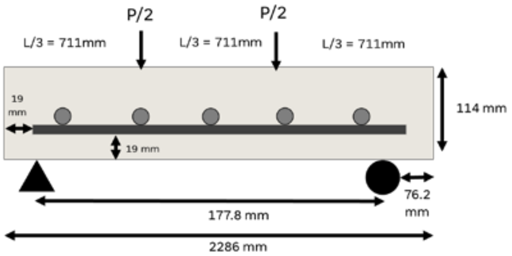


Figure 1. Cross-section of the Specimens

Figure 2. 8 mm Weled Wire Reinforcement (WWR)

Table 1. Test Matrix

Specimen ID	Steel Type	Longitudinal Steel	Cross-weld Spacing (mm)	Steel Ratio, ρ (%)
S1	WWR	D4.7 @ 100mm	150	0.13
S2	WWR	D4.7 @ 100mm	150	0.13
S3	WWR	D4.7 @ 100mm	150	0.13
S4	WWR	D8 @ 150mm	225	0.13
S5	WWR	D8 @ 150mm	225	0.13
S6	WWR	D8 @ 150mm	225	0.13
S7	Rebar	10mm @ 225mm	-	0.13
S8	Rebar	10mm @ 225mm	-	0.13
S9	Rebar	10mm @ 225mm	-	0.13

The American Concrete Institute (ACI) 318-19 and Bangladesh National Building Code (BNBC) 2020 codes were used to design all specimens, with a superimposed dead load of 3.83 kN/m^2 (80 psf) and a live load of 4.79 kN/m^2 (100 psf). The specimens reinforced with WWR were designed with equivalent steel area replacement for conventional deformed bar of similar grade. All specimens were also intended to fail in flexure.

Slabs (S1 to S3) consisted of Class A (BDS-ISO-2016) reinforcing steel of 6 mm diameter. Class A reinforcement used in this study is welded mesh with a longitudinal and cross wire diameter of 6 mm and spaced 100 mm in the longitudinal direction and 150 mm in the transverse direction.

The rest of the slabs (S4–S9) consisted of Class D (BDS-ISO-2016) reinforcement. Slabs S4–S6 were reinforced with a longitudinal and transverse diameter of 10 mm and spaced 225 mm in the longitudinal direction and 325 mm in the transverse direction. Further, Slabs S7–S9 were reinforced with a longitudinal and transverse diameter of 10 mm and spaced 225 mm in the longitudinal direction and 325 mm in the transverse direction.

2.1 Material Tests

The average concrete compressive strength was 28.6 MPa at 28 days. For each steel type (WWR and bars), three steel samples were evaluated. Tensile tests were performed following ASTM A370, and stress-strain data for each WWR tested was obtained using a 2 in. (50 mm) extensometer centered on the cross-weld.

2.2 Specimen Test

The test setup and instrument design used in this work are shown in Fig. 3. The applied load (P) and deflection (Δ) were measured during the experiments. The experimental setup was arranged according to two-point loading configuration. The specimens were loaded at a rate of 38 N/min (218.75 lb/min) using a Universal Testing Machine (UTM) with a custom servo-controlled pump. The applied load was equally distributed using two spreader beams. As indicated in Fig. 2, a dial gauge was used to measure the vertical deflection at the mid-section. An electrical resistance-based full-bridge 112.4-kip (500 kN) load cell was used to measure the applied load at the top of the loading position.

(a)

(b)

Figure 3. Test Setup (a) Loading Configuration (b) Vertical Deflection Measurement

3 Test Results

The focus was to obtain load-deflection data, moment-deflection data, failure mode, and cracking patterns from the nine simply-supported one-way slabs that were tested. Each and every one of the slabs reinforced with WWR behaved similarly until the appearance of the first crack. This was anticipated because the slab behavior is controlled by the concrete before cracking. The key findings of this research are presented in the following sections.

3.1 Material Test Results

Tensile strength tests of the reinforcement specimens were conducted in compliance with ASTM A370. Each of the reinforcements was of similar grade, and the yield strength of 6 mm WWR (585 MPa) was slightly higher in comparison to 10 mm rebar (550 MPa) and 8 mm WWR (528 MPa). The average elongation percentages of 6 mm WWR, 8 mm WWR, and 10 mm deformed bars were 4%, 13%, and 13%, respectively. The weld shear forces for both 6 mm WWR and 8 mm WWR were recorded to be 11.02 kN and 11.52 kN, respectively, at failure. The f_u/f_y ratio was found to be 1.25 for 10 mm deformed rebar, which meets the specification as per ACI 318-19. Consecutively, the f_u/f_y ratio of 6 mm WWR and 8 mm WWR were found to be 1.06 and 1.29, respectively. 6 mm WWR conformed to the specifications of Ductility Class A rebars as per BDS ISO 6935 but it failed to meet ASTM A1064 specifications. In contrast, 8 mm WWR complied to the specifications of ASTM A1064 and Ductility Class D rebars in accordance with BDS ISO 6935.

3.2 Ductility, Deflection and Moment Calculation Results

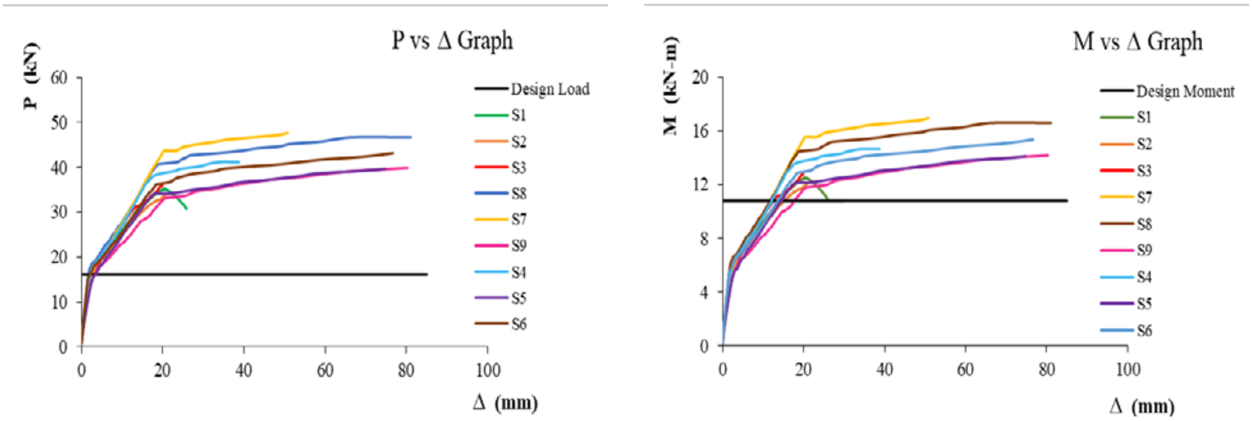
The summary results are exhibited in Table 2, which includes the ultimate deflection and ductility factors. Ductility factor (μ_Δ) is typically expressed in terms of a deflection ratio, $\mu_\Delta = \Delta_u/\Delta_y$, where, Δ_u and Δ_y is the deflection corresponding to peak load and yield load, respectively. To identify the yield load, the peak load from the test is multiplied by a factor of $(1/(f_u/f_y))$. The deflection at yield (Δ_y) is the corresponding deflection at this calculated yield load. This is primarily due to the uncertainty of calculating Δ_y visually from a load-deflection curve, which could significantly affect the μ_Δ value.

Table 2. Summary Results

Specimen ID	Maximum Observed Deflection, Δ_u (mm)	Ductility Factor, μ_Δ
S1, S2, S3	19.86-20.71	1.15-1.28
S4, S5, S6	38.75-76.65	2.95-5.12
S7, S8, S9	50.8-80.28	2.41-4.98

3.3 Load vs Deflection and Moment vs Deflection Graph

From Fig. 4, it is evident that each of the samples satisfied the design load of 16.15 kN and allowable moment capacity of 10.77 kN-m, but the 6 mm WWR samples failed earlier due to low ductility. All the specimens exhibited similar load-carrying capacities up to the yield point.



(a) (b)

Figure 4. Response of Different Slabs for Two-point Loading (a) Load vs Deflection Graph (b) Moment vs Deflection Graph

The deflection-based ductility factor (μ_Δ) of 6 mm WWR samples ranged from 1.15-1.28, which is considerably low. The ductility factors of 8 mm WWR and 10 mm deformed bar specimens ranged from 2.95–5.12 and 2.4–4.98, respectively. 8 mm WWR and 10 mm deformed bar demonstrated greater ductility and vertical deflection due to higher ductility class.

3.4 Mode of Failure

There were differences in failure modes and cracking patterns among slabs reinforced with 6 mm WWR, 8 mm WWR, and 10 mm deformed bar. Fig. 5 shows the complete mode of failure of the specimens. 6 mm WWR-reinforced specimens failed catastrophically due to rupture of the wire reinforcement. In the case of WWR specimens, crack propagation is uniform compared to 10 mm deformed bar. In the case of the 10 mm deformed rebar-reinforced sample, the specimen failed due to the yielding of 10 mm rebar, but there was no instance of concrete crushing at the top compression zone.

(a) (b)

(c)

Figure 5. Mode of Failure of One-way Slabs (a) 6 mm WWR Reinforced Sample (b) 8 mm WWR Reinforced Sample (c) 10 mm Deformed Bar Reinforced Sample

4 Conclusions

The ductility of concrete members reinforced with WWR was investigated using a simple experimental program. The program involved evaluating nine simply-supported slabs with a span-to-depth ratio of 20 using 6 mm WWR, 8 mm WWR, and 10 mm deformed bar. The nine specimens were loaded to failure, with member deformation being recorded constantly as the load increased. The following inferences were obtained as a result of the experimental investigation:

- The flexural strength of slab panels reinforced with 6 mm WWR of ductility class A produced in Bangladesh is significantly lower compared to 8 mm WWR and 10 mm deformed bars of ductility class D.
- 6 mm WWR of ductility class A, which is locally produced in Bangladesh and is referenced in this paper, does not comply with the specifications of ASTM A1064. Therefore, its use in suspended, reinforced concrete slabs is not recommended.
- 8 mm WWR exhibited sufficient ductility prior to failure, and it conforms to the specifications of ASTM A1064. Therefore, it can be utilized in RC slabs.
- The use of WWR ensures uniform crack distribution in RC slabs at failure.

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