The 2015 World Congress on
Advances in Civil, Environmental, and Materials Research (ACEM15)
Incheon, Korea, August 25-29, 2015

Effect of glass-fiber rods on the ductile behaviour of reinforced concrete beams

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ABSTRACT

Ductility can be defined as the "ability of material to undergo large deformations without rupture before failure". The use of Glass-fiber reinforced polymer (GFRP) bars instead of steel in reinforcing concrete structures is currently encouraged by many structural engineers, especially for their lightness and non-corrosive properties. The study compared the ductile behaviour of beams reinforced with glass-fiber to beams reinforced with steel reinforcement. A total of eighteen beams divided into six groups, each group consists of three identical beams were tested to investigate the ductile behaviour.

Diana finite element program was used before the experimental work to investigate the theoretical behaviour of the beams such as the moment-curvature relationship and distribution of stress and strain along the beam cross-section. The study concluded that, the conventional definitions of ductility are inappropriate for evaluation of the ductility of glass-fiber reinforced (GFR) concrete beams because the GFR have no yield point. Thus, there is a need for both quantitative and qualitative evaluations of ductility when using GFR in reinforced concrete members. The deformability factor and the performance factor for steel reinforced beams have the same trend as conventional ductility, especially the deformability factor. Thus, the deformability factor may be used as a comparison index for both steel reinforced beams and GFR beams. The deformability factor showed that, the ductile behaviour of GFR beams enhanced by increasing the GFR ratio. This may be referring to the increasing in the deformation capacity before failure for the beams that reinforced with high GFR ratio.

Keywords: ductility; bond behaviour; deformability; glass-fiber reinforcement bars; RC beams.

1. Introduction

A little experimental data is available for the flexural behaviour of reinforced concrete beams reinforced with glass-fiber rods (GFR), and even less for a ductile behaviour of these beams. Thiagrgarajan (2003) presented the result of an experimental and analytical comparison of a study on the flexural behaviour of concrete beams reinforced with sandblasted carbon fiber-based composite rods. He concluded that the bonding of sandblasted rods is not the major concern. However, excessive deformation in achieving the predicted moment capacity could be a limiting factor in the design of these beams. Nanni et al. (1995) published a state-of-the-art review of the test methods of fiber reinforced polymers (FRP) systems, which is a very comprehensive compilation. Nanni (1993) discussed the issues involved in the flexural behaviour and design of reinforced concrete (RC) members using FRP reinforcement and observed that deflection control is as important as flexural design. Many researchers have worked on glass-fiber reinforcement rods. Martinola et al. (2010) studied the strengthening and repair of RC beams with fiber reinforced concrete. From the experimental results, the application of a 40mm thick HPFRC jacket on a RC beam provides an increase of the ultimate load. The proposed technique provides a significant structural enhancement at the serviceability limit state due to the remarkably increase of the beam stiffness under service load, the mid-span deflection can be remarkably reduced.

Huanzi and Belarbi (2011) studied the ductility characteristics of fiber reinforced concrete beams reinforced with FRP rebar. They concluded that addition of polypropylene fiber has been proved to be an effective way to enhance the ductility of FRP reinforced system. Sadaatmanesh et al. (1991) conducted experiments on concrete beams using GFR rods and made precise predictions of maximum loads using GFR properties.

In designing a flexural member for structural safety, both the flexural strength and ductility have to be considered. For this purpose Francis et al. (2010) studied the flexural ductility of reinforced concrete sections. His study analyzed the full-range flexural responses of reinforced and prestressed concrete sections taking into account the nonlinearity and stress-path dependence of constitutive materials. Francis et al. concluded that whilst the concept of flexural ductility in terms of the ductility factor works well for reinforced sections, it can be misleading when applied to prestressed concrete sections. For prestressed concrete sections, the concept of flexural deformability in terms of ultimate curvature times overall depth of section may be more appropriate.

Almusallam (1997) presented an analytical model for the prediction of the flexural strength, compared it with experimental tests on concrete beams reinforced with GFR rods, and demonstrated a good correlation between analytical and experimental values for moment-curvature and load-deflection relationship. Satoh et al. (1991) conducted tests using carbon-fiber reinforcement polymers (CFRP), GFRP and steel bars.

Dong et al. (2012) conducted experimental research on the fatigue and post-fatigue static behavior of reinforced concrete beams strengthened with glass or carbon fiber reinforced polymer sheets placed either vertically or obliquely. The test results have shown that externally bonded CFRP or GFRP to the lateral and bottom faces of a beam can increase the first crack load and ultimate strength greatly, arrest concrete crack extension and enhance the rigidity of strengthened beams. The CFRP strengthened beam has the highest ultimate strength but the lowest deflection and the diagonal GFRP reinforcing arrangement is more effect than the vertical arrangement in enhancing the shear strength and stiffness. For CFR rods, they reported a failure ratio of 0.876 for the experimental to predicted failure loads. Other related research has focussed on the effect of varying the strength of concrete, reinforcement ratio, etc. using FRP rods in concrete beams. Theriault et al. (1998) studied the effect of reinforcement ratio and concrete strength on the flexural behaviour of concrete beams with GFRP reinforcement. They concluded that the effect of reinforcement ratio and concrete strength on crack spacing appeared to be negligible and that a higher reinforcement ratio decreased the width and height of cracks. Faza et al. (1991) studied the bending response of beams with different concrete strengths using FRP rods. They have shown that the ultimate moment capacity of beams made with high strength concrete (50 MPa) increased by 90 % when FRP bars of ultimate strength of 90 MPa are used, when compared to mild steel bars. An ultimate moment capacity increase of 100 % has been noted when the concrete strength was increased from 34.5 MPa to 51.7 MPa.

Mahalingam et al. (2013) presented the results of an experimental investigation conducted on Steel Fiber Reinforced Concrete (SFRC) beams with externally bonded Glass Fiber Reinforced Polymer (GFRP) laminates with a view to study their strength and ductility. The variables considered included volume fraction of fiber reinforcement and stiffness of GFRP laminates. The static responses of all the beams were evaluated in terms of strength, stiffness and ductility. The test results show that the beams provided with externally bonded GFRP laminates exhibit improved performance over the beams with internal fiber reinforcement.

The objectives of this paper are to study the effect of using GFR on the ductile behaviour of RC beams. A total of eighteen beams divided into six groups, each group consists of three identical beams were tested for flexural strength capacity and ductile behaviour. A total of nine concrete beams were reinforced with glass-fiber reinforcement (GFR) and the others were reinforced with steel reinforcement (SRFT). The behaviour of all beams was observed for different reinforcement ratios. The details of these beams will be presented in the following section.

2. MATERIAL PROPERTIES

2.1. Reinforcement Bars

High alkalisation of cement matrix, low permeability and concrete cover play the main role to protect steel reinforcement from corrosion. Another good alternative is the using of glass-fiber reinforcement (GFR), which is a non-corrosion material and has a high linear axial tensile strength equal or more than steel reinforcement. Strength and rigidity of GFR bars can be defined according to the kind, number and adjustment of the glass-fibers. GFR behaves linear-elastic up to fracture, if the maximum tensile strength of the material is exceeded. Yielding of the material does not happen. The material has relatively low compression and tensile strength perpendicular to the fibers. Table 1 shows the properties for the used reinforcement in the study. The stress-strain curves for the used reinforcement are shown in Fig. 1.

Table 1 Properties of reinforcement bars used in the study

Material properties of Straight bars	Steel reinforcement SRFT 36/52	Glass-fiber rods GFR
Maximum tensile strength f_t (N/mm²)	520	1000
Characteristic yield strength f_y (N/mm²)	360	1000
Designed yield strength f_{yd} (N/mm²)	313	313*
Strain at yield stress ε_y (mm/m)	1.80	6.0**
Modulus of elasticity E (N/mm²)	200000	60000
Concrete cover (mm)	20 - 30	d_b + 10
Density (g/cm³)	7.85	2.20
Thermal conductivity λ (W/mk)	60	0.50
Magnetism	yes	no

*: Design stress for the glass-fiber reinforcement bars (no yield).
**: Design strain for the glass-fiber reinforcement bars at design stress of 360 N/mm^2.

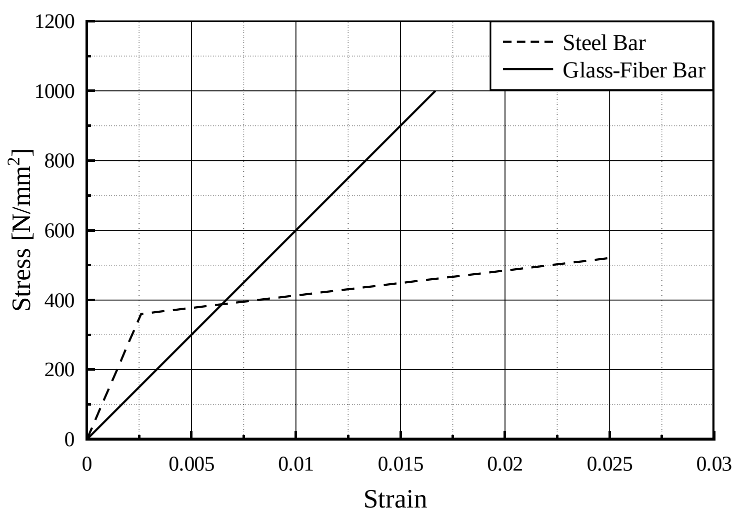


Fig. 1 Stress-strain curve for SRFT and GFR bars

2.2. Concrete Properties

Three cubes 150*150*150 mm were taken during casting of each tested beam to find its characteristic concrete compression strength after 28 days. The properties of concrete used in the study are presented in Table 2.

Table 2 Properties of concrete used in the study

Characteristic cube compression strength, $f_{ck,cube}$ (N/mm²)	32.00
Flexural strength, f_{ct} (N/mm²)	2.35
Calculated tensile strength, f_{cto}, ECP 203 (2007) (N/mm²)	1.94
Modulus of elasticity, E_c (N/mm²)	25000

3. SAMPLE PREPARATION, TESTING, AND MONITORING

Bond behavior is a key aspect of Tension-Stiffening since it controls the ability of the reinforcement to transfer tensile stresses to the concrete (Materschläger et al. 1998). The ductility of structural concrete may be considerably affected by construction details. This is due to their impact on bond properties between reinforcement and concrete which, in turn, have an impact on plastic deformation capacity (Daia Zwicky, 2013). Therefore, it was important to investigate the bond behavior between concrete and glass-fiber reinforcement GFR and steel reinforcement SRFT before beginning the test of beams. For this purpose, 6 specimens (3 with GFR and 3 with SRFT) were tested as pull-out test. A concrete specimen of 150 x 150 x 150 mm was used with contact length between the reinforcement bar and the concrete of five times the bar diameter (d_b). A special mould was produced to ensure the rebar position during casting. The bar was extended with 300 mm from the active end and with 50 mm from the dead end. The concrete specimen was fixed in the machine as shown in Fig. 2, and the bar was pulled out with a loading rate of 0.005 mm/s. The slip was measured from the dead end of the bar. Fig. 2 shows the results of pull-out test. The anchorage length was calculated depend on the results obtained from pull-out test and anchorage length equation in ECP 203 (2007).

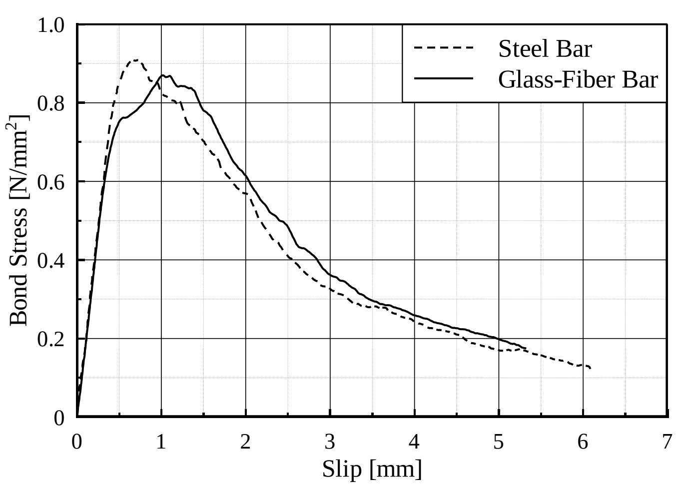


Fig. 2 Bond stress-slip diagram for GFR and SRFT with concrete

As shown in Fig. 3, the used reinforcement has the same length as beam length with straight anchorage length of 150 mm after the support point. Four displacement gauges were fixed at ends of the beam during test; in order to verify that no slip occurred between GFR or SRFT and the surrounding concrete. The beams were loaded with a displacement-controlled actuator at a constant rate of 0.02 mm/s. Loading continued after maximum load was reached so that the post peak behavior of the beams could be observed. All tests were monitored in different ways. The deflection of the beam was measured using displacement gauges at five points under each beam. The slip between the longitudinal compression and tensile reinforcement and concrete was monitored with four displacement gauges; two gauges at each end of the beam. The compression and tensile strains in the longitudinal reinforcement were measured with strain gauges. In order to determine the depth of compression zone at mid span of the beam, 6 strain gauges were attached to concrete surface in compression zone every 20 mm.

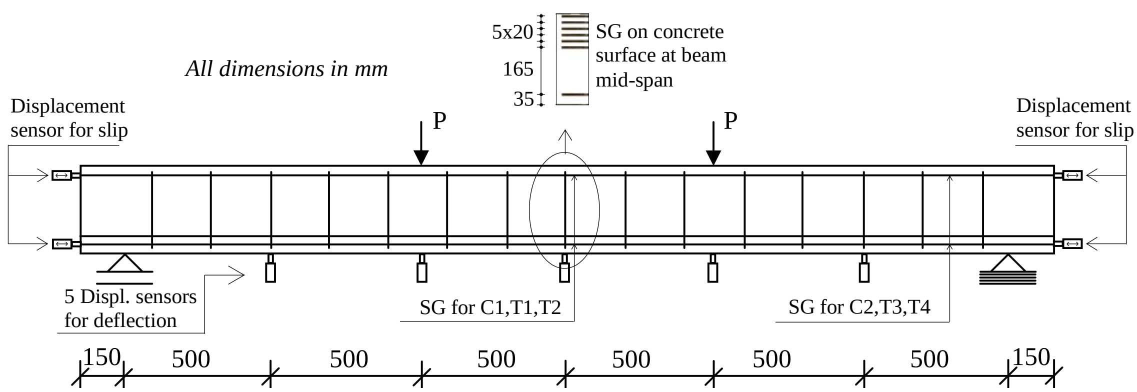
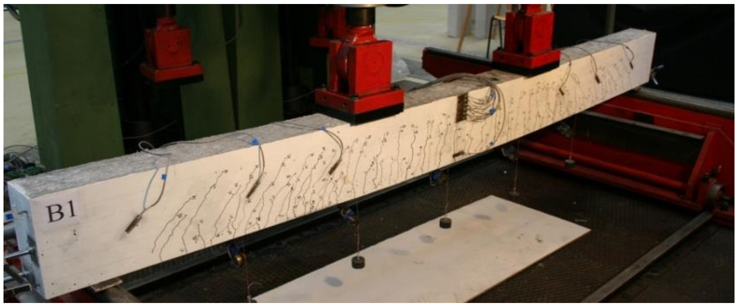


Fig. 3 Positions of displacement sensors and strain gauges on the tested beams

Fig. 3 shows the positions of strain gauges on concrete surface and reinforcement bars. The Finite Element Programs (Diana) were used before the experimental work to design the cross section of the tested beams according to ECP 203 (2007) and to investigate the theoretical prediction for the behaviour of the beams such as the moment curvature relation and distribution of stress and strain along the cross section of the beams. The crack pattern and the principle stresses for a tested beam are shown in Fig. 4. The details of the tested beams are shown in Table 3 and Fig. 5.



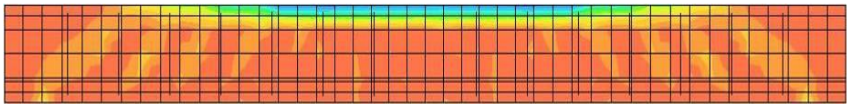


Fig.4 Crack pattern and principal stresses for a tested beam according to experimental and finite element analysis

Table 3 Details of the tested beam groups

Beam No.	Cross-Sec. Dimensions	Cube Concrete Strength, f_{cu}	Stirrups RFT.	Type of Reinforcement	Top RFT.	Bottom RFT.	Reinforcement Ratio[%], ρ_s
GB1	150*300	32.00	ϕ 8/100	GFR	2 ϕ 8	2 ϕ 12	0.50
GB2	150*300	31.62	ϕ 8/100	GFR	2 ϕ 8	3 ϕ 12	0.75
GB3	150*300	33.50	ϕ 8/100	GFR	2 ϕ 8	4 ϕ 12	1.00
GB4	150*300	32.28	ϕ 8/100	SRFT	2 ϕ 8	2 ϕ 12	0.50
GB5	150*300	31.40	ϕ 8/100	SRFT	2 ϕ 8	3 ϕ 12	0.75
GB6	150*300	34.25	ϕ 8/100	SRFT	2 ϕ 8	4 ϕ 12	1.00

GB1: Beam Group No. 1, GFR: Glass-Fiber Reinforcement, SRFT: Steel Reinforcement

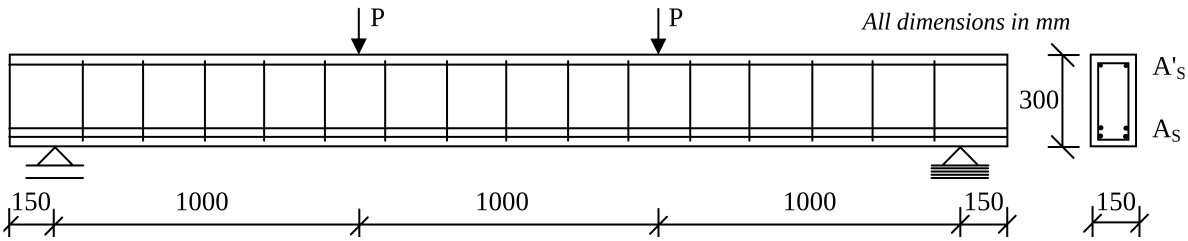


Fig. 5 Schematic detail of beam layout

4. MOMENT-CURVATURE COMPARISON

A variety of results are presented, both from the experimental observations and the finite element analysis. Theoretically, the experimental curvature can be calculated from curvature of the deflection line of the beam during test, but this may be accepted according to the linear elastic behaviour of the beam, but it may not accurate according to nonlinear behaviour. Therefore, the experimental curvature (ϕ_{ex}) was accurately calculated from the following equation.

$$\phi_{ex} = \varepsilon_c / x \qquad (1)$$

where (ε_c) is the maximum compression concrete strain and (x) is the depth of compression zone corresponding to (ε_c).

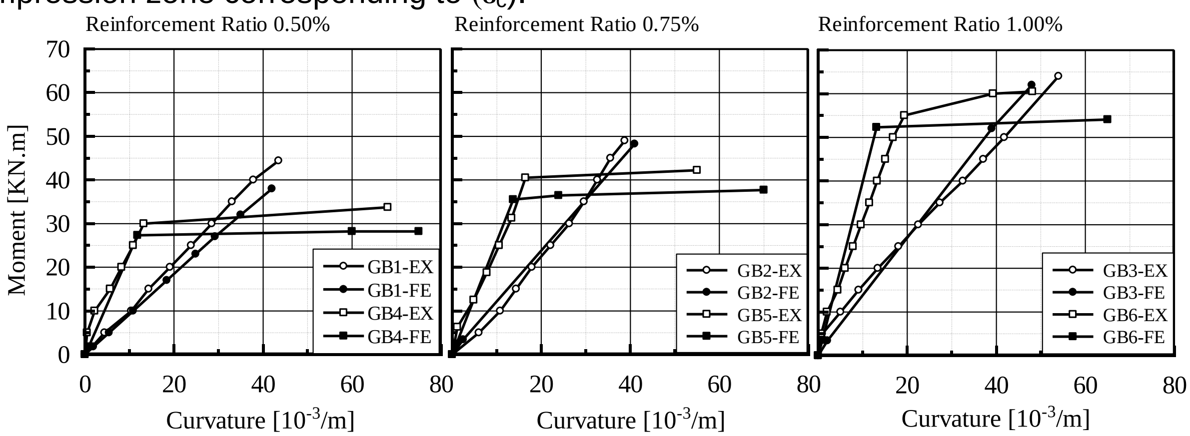


Fig. 6 Experimental and analytical moment-curvature relations

Fig. 6 shows the finite element analysis (FE) and experimental (EX) moment-curvature curves for beam groups (GB1 and GB4), (GB2 and GB5) and (GB3 and GB6) with reinforcement ratios of 0.50%, 0.75% and 1.0 %, respectively. In general, there is no significant difference between the analytical and experimental moment-curvature curves. Using of GFR bars increase the beam curvature capacity at the same moment level. From Fig. 6, it can be noted that the steel reinforced beams reach a certain moment capacity and sustain it for a long increase in curvature before failure. On the other hand, the GF reinforced beams reach their peak moment capacity just before failing. However, it should be stated that the GF reinforced beams are capable of exhibiting deformation characteristics comparable to that of steel reinforced beams before failure, the only difference being that they are unable to sustain peak capacities for long before failing. This can be attributed to the elastic ideal plastic behaviour of steel and a purely elastic behaviour of GFR rods. The high deformation of GF reinforced beams can be attributed to the rods being capable of fairly large strains before reaching an ultimate strength of 1000 MPa.

5. LOAD-DEFLECTION RELATION

Fig. 7 shows typical load-deflection relationships for the tested beam groups (GB1 and GB4), (GB2 and GB5) and (GB3 and GB6). As expected, the steel reinforced beams became nonlinear after yielding of SRFT, with a large deflection increase but with little load gain. However, the GF reinforced beams behaved differently, the load continued to increase with an increase of deflection, and the load deflection relationship was almost linear till failure.

At yielding limit of SRFT beams, the mid span deflection in GF reinforced beams was approximately 3 times as that in steel reinforced beams. At the ultimate state, the steel reinforced beams with reinforcement ratio of 0.5 % (GB4) show more ductile behaviour than that with reinforcement ratio of 1.0 % (GB6), which led to that at reinforcement ratio of 1.0 %, the maximum deflection in GF reinforced beam (GB3) was 1.5 times as that in steel reinforced beam (GB6), while at reinforcement ratios of 0.5 % and 0.75%, the maximum deflection in GF reinforced beam (GB1 & GB2) was approximately equal to that in steel reinforced beam (GB4 & GB5), respectively. In other words, it can be stated that at high reinforcement ratio the difference between the maximum deflection values for GF reinforced beams and steel reinforced beams will be more than that at low reinforcement ratio. Another worthy note here is the residual deflection of the GF reinforced beams is very small comparable to that for steel reinforced beams. This may attribute to the pure elastic behaviour of the GFR bars.

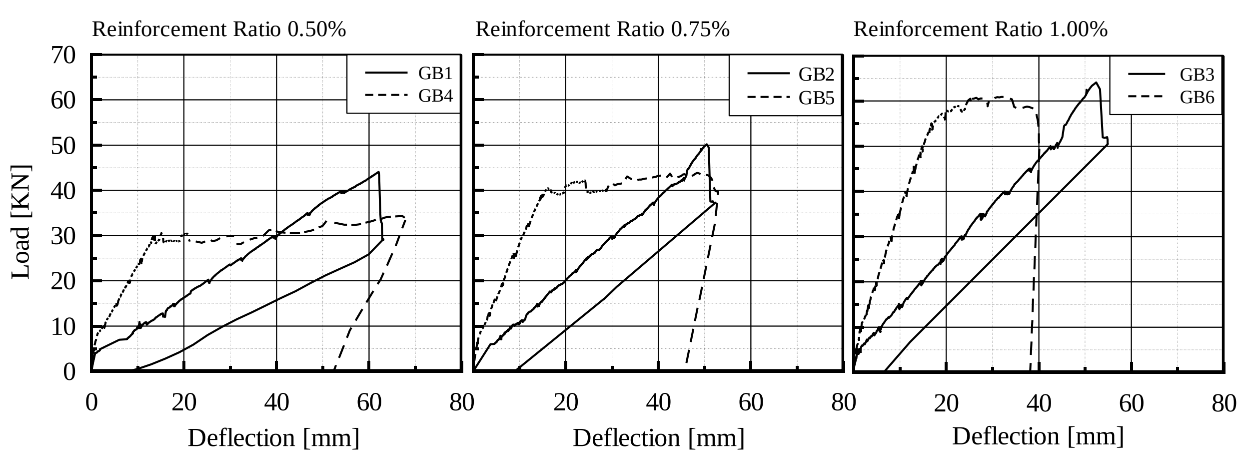


Fig. 7 Load-deflection diagram for tested beams reinforced with GFR and SRFT rods

In steel reinforced beams, the deflection prior to failure was 1/45 and 1/75 of the span for reinforcement ratios of 0.50 % and 1.00 %, respectively. However the deflection prior to failure in GF reinforced beams was 1/50 and 1/55 of the span for reinforcement ratios of 0.50 % and 1.00 %, respectively, which is considered to be sufficient deflection to provide warnings before failure. Abdelrahman (1995) also arrived at a similar conclusion for beams with CFRP.

6. CONVENTIONAL AND MODIFIED DUCTILITY

Ductility is a desirable structural property because it allows stress redistribution and provides warning of impending failure. In general, high ductility ratios indicate that a structural member is capable of undergoing large deflections prior to failure. Steel-reinforced concrete beams should be designed, so that failure is initiated by yielding of the steel reinforcement, followed, after considerable deformation at no substantial loss of load carrying capacity, by concrete crushing and ultimate failure. This mode of failure is ductile and is guaranteed by designing the tensile reinforcement ratio to be substantially below (ACI-318-99 requires at least 25 % below) the balanced ratio, which is the ratio at which steel yielding and concrete crushing occur simultaneously. Conventional definitions of ductility involve the onset of yield of steel reinforcement and, therefore, are inappropriate for evaluation of the ductility of concrete beams reinforced with fibre reinforced polymer (FRP) because FRP has no yield point. Unless ductility requirements are satisfied, FRP cannot be used with confidence in structural members. Thus, there is a need for both quantitative and qualitative evaluations of ductility when using FRP in reinforced concrete structural members (Naaman and Jeong 1995). This includes computation of a suitable ductility index and comparison of behaviour at ultimate between beams containing FRP reinforcement and those containing steel reinforcement.

Ductility factors have been commonly expressed in terms of the various parameters related to deformation, i.e., curvatures, rotations, and displacements (Bachmann et al. 1992; Park 1992). These conventional ductility factors can be calculated as:

Curvature ductility μ_ϕ :

$$\mu_\phi = \phi_u \big/ \phi_y \tag{2}$$

where (ϕ_u) is curvature of the element at ultimate state and (ϕ_y) is curvature of the element at yield limit.

Rotation ductility μ_θ :

$$\mu_\theta = \theta_u / \theta_y \qquad (3)$$

where (θ_u) is the ultimate plastic hinge rotation and (θ_y) is the yield plastic hinge rotation.

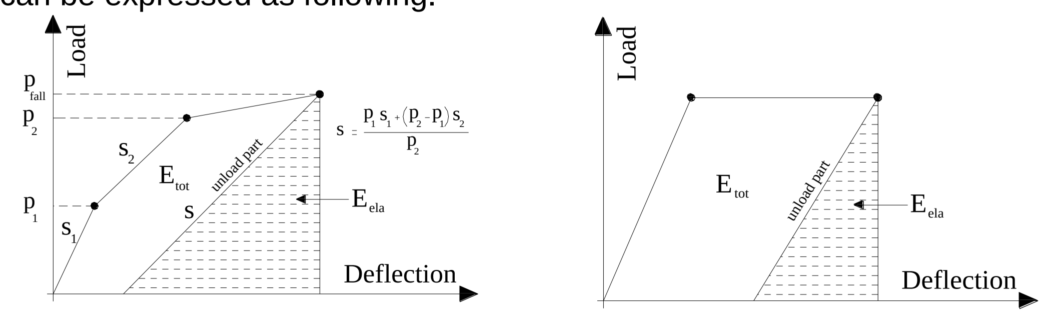
Displacement ductility μ_Δ :

$$\mu_\Delta = \Delta_u / \Delta_y \qquad (4)$$

where (Δ_u) is the displacement of a structural element or of a whole structure at the ultimate state and (Δ_y) is the displacement at yield limit.

Ashour (2000) mentioned that members with a displacement ductility in the range of 3 to 5 has adequate ductility and can be considered for structural members subjected to large displacement, such as sudden forces caused by earthquake.

All these conventional methods for ductility are not suitable for beams with FRP as FRP does not have a yield point (Patrick 2003). Researchers have proposed some new equations to quantify the ductility of concrete beams reinforced by FRP and by steel so that a comparison may be made between them. These modified ductility or deformability factors can be expressed as following.



Graph to determine ductility index μ_{en} for beam with elastic reinforcement according to Naaman and Jeong (1995)

Graph to determine ductility index μ_{en} for beam with elasto-plastic reinforcement according to Naaman and Jeong (1995)

Fig. 8 Total and elastic energy under load-deflection curve (Naaman and Jeong 1995)

Naaman and Jeong (1995) proposed a ductility index μ_{en} as:

$$\mu_{en} = 0.5\left(\frac{E_{tot}}{E_{ela}} + 1\right) \qquad (5)$$

where (E_{tot}) is the total energy under the load-deflection curve up to failure load (Fig. 8) and (E_{ela}) is the elastic energy computed as the area of the triangle formed at failure load by unloading the beam (Fig. 8). The development of this equation was based on the assumption that the concrete beams has fully elasto-plastic behaviour and is equally applicable for beams with FRP. ACI-440 (2001) defined the ductility index as the ratio of energy absorption (area under the moment-curvature curve) at ultimate strength of the section to the energy absorption at service level (ACI-440, 2001).

Abdelrahman, et. al. (1995) presented a deformability factor μ as:

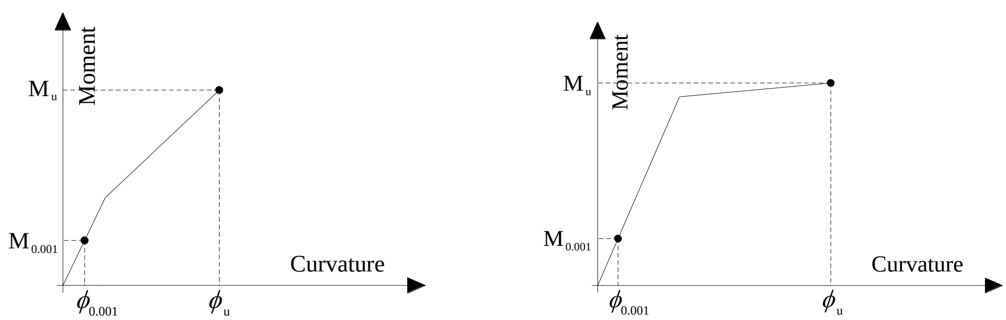
$$\mu = \Delta_u / \Delta_l \qquad (6)$$

where (Δ_u) is the ultimate deflection and (Δ_l) is the equivalent deflection at uncracked section.

Mufti, et. al. (1996) and Jaeger, et. al. (1997) proposed an overall performance factor μ_M as:

$$\mu_M = (M_u \ \phi_u / M_{0.001} \ \phi_{0.001}) \qquad (7)$$

where (M_u) is the ultimate moment, (ϕ_u) is the curvature at ultimate state, ($\phi_{0.001}$) is the curvature at concrete strain of 0.001 at the outer most compression fibre and ($M_{0.001}$) is the moment at a concrete strain of 0.001 at the outer most compression fibre (Fig. 9).



Graph to determine overall performance factor μ_M for beam with elastic reinforcement according to Mufti, et. al. (1996)

Graph to determine overall performance factor μ_M for beam with elasto-plastic reinforcement according to Mufti, et. al. (1996)

Fig. 9 Moment-curvature diagram for concrete beams in elastic and elasto-plastic behaviour

This model was developed for concrete beams with a rectangular cross-section and was based on a particular type of failure mode, namely, crushing of the concrete. The researchers claimed that under service load conditions, the concrete strain at the top compression fibre is about 0.001 for reinforced concrete beams.

Canadian Highway Bridge Design (CAN/CSA) (2000) Code has included provisions for fibre-reinforced structures where FRP is used as reinforcement and the Code includes a section "*Design for Deformability*". The code mentioned that for concrete beams reinforced with FRP bars or grids, the overall performance factor ($_M$) must be at least 4.0 for rectangular sections and 6.0 for T–sections (Bakht, et. al. 2000).

7. DUCTILITY AND DEFORMABILITY INDICES

Depending on the conventional, modified ductility and deformability indices that mentioned in previous section as well as the experimental results, the comparison between the ductility and deformability indices for the flexural cracked tested group beams are summarized in Table 4.

Table 4 Ductility and deformability indices for the flexural tested group beams

Group Beam number	GB1	GB2	GB3	GB4	GB5	GB6
Type of Reinforcement	GFR	GFR	GFR	SRFT	SRFT	SRFT
Reinforcement ratio [%]	0.50	0.75	1.00	0.50	0.75	1.00
Conventional Ductility						
Displacement ductility, μ_Δ	NA	NA	NA	3.39	3.08	1.92
Rotation ductility, μ_θ	NA	NA	NA	2.91	2.23	1.58
Curvature ductility, μ_ϕ	NA	NA	NA	5.15	3.34	2.47
Modified Ductility						
Ductility index, μ_{en}	1.540	1.321	1.261	3.680	6.810	14.330
Deformability factor, μ	16.11	18.23	27.45	93.83	61.22	44.37
Performance factor, μ_M	7.501	6.798	5.901	7.179	6.021	5.744

NA: Not Applicable

For the conventional ductility (Table 4), the ductility of steel reinforced beams with low reinforcement ratios is less than that with high reinforcement ratio, while the conventional ductility equations are not applicable for GF reinforced beams.

As a comparison between the different modified ductility and deformability indices mentioned in Table 4, the deformability factor (μ) and the performance factor (μ_M) show logical and acceptable trend as conventional ductility for steel reinforced beams, while the ductility index (μ_{en}) and the performance factor (μ_M) show similar trend for the ductility of GF reinforced beams. The performance factor (μ_M) plays as indicator for the ductile behaviour when it compared to the limit value that were mentioned in Canadian Highway Bridge Design (CAN/CSA) Code (2000), which must be at least 4.0 for rectangular sections, therefore, the performance factor (μ_M) shows that all the tested GF reinforced beams behave as ductile beams, although they contain brittle materials such as GFR and concrete. This may be due to the high deformation capacity of these beams before their failure, which present enough warranty before failure.

8. CONCLUSION

The conventional ductility for steel reinforced concrete beams showed that when the reinforcement ratio increased the ductility of the beams decreased specially in the case of curvature ductility.

By comparing the three modified ductility equations for steel reinforced concrete beams, they showed various trends. In the case of ductility index, the ductility increased by increasing the steel reinforcement ratio, which is not meet the trend of conventional ductility. However, the deformability factor and the performance factor for steel reinforced beams are closed to conventional ductility values especially in the case of deformability factor. Thus, the deformability factor may be used as a comparison index for both steel reinforced beams and GF reinforced beams.

The GF reinforced beams reach their peak moment capacity just before failing. However, it should be stated that the GF reinforced beams are capable of exhibiting deformation characteristics comparable to that of steel reinforced beams before failure, the only difference being that they are unable to sustain peak capacities for long before failing. This can be attributed to the elastic ideal plastic behaviour of steel and a purely elastic behaviour of GFR rods.

In General, the GFR concrete beams showed ductile behaviour less than that of steel reinforced concrete beams. The ductile behaviour of GF reinforced beams enhanced by increasing the GF reinforcement ratio as the deformability factor showed. This may be referring to the increasing in the deformation capacity before failure for the beams that reinforced with high GF reinforcement ratio.

The high deformation of GF reinforced beams can be attributed to the rods being capable of fairly large strains before reaching an ultimate strength of 1000 MPa that will not occurred, where the GFR designed strength is less than 360 MPa (Table 1). It can be stated that at high reinforcement ratio, the difference between the maximum deflections values prior to failure for GF reinforced beams and steel reinforced beams will be more than that at low reinforcement ratio. The deformability factor shows that, the tested GF reinforced beams behave as ductile beams, although they contain brittle materials such as GFR and concrete. This may be due to the high deformation capacity of these beams before their failure, which present enough warranty before failure.

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