CORROSION-FREE CONCRETE STRUCTURES BY THE USE OF FRP (FIBRE REINFORCED POLYMERS)

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ABSTRACT

Fibre reinforced polymers may be suggested as a solution to one of the major durability problems with reinforced concrete; the corrosion of the internal steel reinforcement. The durability of fibre reinforced polymers, FRP, e.g. towards chlorides, has the potential of saving extensive maintenance costs and avoiding resulting inconveniences for the public. This paper presents a pilot study with the use of glass fibre reinforced polymers, GFRP, as internal reinforcement for concrete structures. In the comparison between major international design guidelines, similar design procedures for ultimate limit state, ULS, but different design procedures for serviceability limit state, SLS, are used. Four T-beams, of which two were reinforced with GFRP rebars and the other two with steel rebars, were tested to failure in 4-point bending. Optical strain measuring equipment was used for the measurement of crack propagation. A comparison between predictions of failure loads, deflections and crack widths based on the design guidelines and the experimental results has been performed. For both GFRP reinforced beams different from predicted failure modes occurred, but at loads almost at the predicted level or higher. Extensive deflections and crack propagation gave clear warnings before failure. The behavior of the GFRP reinforced beams in SLS was reliable predicted by the design guidelines for FRP with the same accuracy as for the reinforced concrete beams. The SLS predictions of crack widths and deflections showed increasing overestimation for increasing reinforcement ratios. SLS criteria have shown to be governing in the design for the GFRP reinforced members.

Keywords: Concrete, FRP, Design, Testing, Corrosion

1. INTRODUCTION

Developments in the construction industry are strongly linked to developments in the material science field. This has been a driving factor when it comes to improved production technology and profitability. A common factor for all materials used, individually or in systems, is that they should be durable over time. The durability and longevity of reinforced concrete structures today is in focus, where in many cases it can be noted that the design life higher than 100 years, is not achieved and that maintenance needs are greater than assumed during the construction. In particular, the corrosion of steel reinforcement cause by carbonation and/or chlorides is a serious problem for many structures worldwide leading to many structural failures. This challenge have prompted numerous research programs that address the following problems: material degradation models [1], repair and upgrading methods [2], and maintaining the desired levels of reliability [3] – just to mention a few. An alternative to steel reinforcement that has gained attention the last two decades is the use of non-metallic, non-corrosive reinforcement, i.e. fiber-reinforced polymers (FRP). The FRP is a composite material consists of continuous fibers protected by a resin matrix. Commercially available fibers are glass, carbon, aramid and basalt fibers. Due to their excellent properties, i.e. non-corrosive properties, magnetic neutrality, lightweight, and high tensile strength the FRPs constitute a viable method for reinforcing concrete structures and achieve corrosion-free structures. A major reason why the materials have not come to greater use in our Nordic countries is the lack of knowledge at all stages in the construction process, both for clients, designers and contractors. You simply do not know the potential and if no one prescribes these materials, the contractors will not use them even if you can understand the long-term benefits [4]. Unlike steel, which exhibits yielding and a plastic behavior, the FRP composites behaves linearly up to failure. This means that serviceability rather than the ultimate limit state are often the decisive aspect in the design of such structures [5]. Specifically, the GFRP (GlassFRP) bars shows lower modulus of elasticity compared with steel and a different bond behavior, meaning large crack widths and deflections. Empirical equations for can be found in international guidelines to predict maximum crack width and deflections [6-9]. Such equations has shown to provide large scatter (both conservative but also un-conservative) against the experimental tests [10].

2. GENERAL DESIGN OF FRP

A difference between steel reinforcement and FRP reinforcement, and especially GFRP reinforcement, is that as mentioned earlier it has a significantly lower E-module. This is of less importance for uncracked concrete elements, but is of greater importance once cracking has occurred. This means that for the corresponding amount of reinforcement, the cracks become larger. This usually does not matter in terms of resistance, but the deflections increase. An FRP reinforced concrete element loaded in bending has similar failure modes as for steel reinforcement; compressive failure in the concrete, tensile failure in the FRP reinforcement and concurrent crushing of the concrete and tensile breakage in the FRP reinforcement. It should be emphasized that for a steel reinforced concrete element yielding in the tensile reinforcement is a ductile failure mode. However, should tensile breakages occur in the FRP reinforcement, this is brittle. Depending on the design, however, cracking and large deformations can be obtained before failure. The failure in the ultimate limit state is usually design for compressive failure in the concrete. It can also be mentioned that as long as the concrete member is in stage II, the concrete element returns to its original position when unloading – it behaves linear elastic. This does not happen for a steel reinforced concrete element if you start to achieve yielding - then a remaining deformation is obtained, even if the concrete member is in stage II.

3. LABORATORY TEST AND MOMENT CAPACITY

The tests presented in this paper were conducted at Denmark Technical University in 2007 under the supervision of Professor Täljsten, [11] and this research will now be continued at Luleå University of Technology, see section 4 in this paper. In total 4 beams were tested in four point bending. The distance between the supports was 4.0 m and the loading was applied 700 mm from the center of the beam. During loading, strain in tensile steel and FRP reinforcement as well strain on the top of the concrete was measured section A. Relative deflection was measured by LVDT between the supports and the mid-section of the beam, see also Figure 1. In Table 1 the reinforcement and type of reinforcement is shown together with calculated moment capacity and type of failure. The glass fibre bars used was Combar with an Elastic modulus of 61 MPa for ϕ 16 mm and 49 MPa for ϕ 32 mm. Corresponding strains at failure were 7.0 respectively 3.5 ‰. This was verified by tests in the laboratory [11].

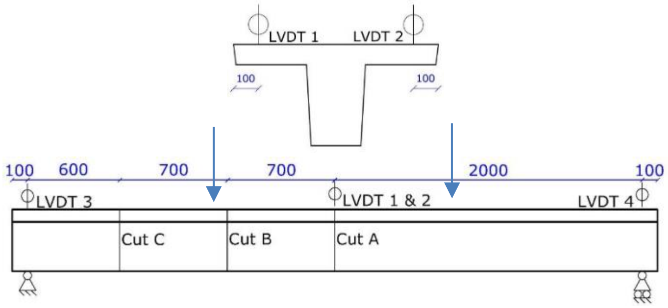

Figure 1 – Placement of sensors during loading, [11]

Table 1 – Rebars in beams [11]

Beam	Description	Rebar	Expected failure mode [-]	Moment capacity [kNm]
BS5O16	Beam Steel rebars 5 x ϕ 16	Steel	Yield steel	206
BS6O20	Beam Steel rebars 6 x ϕ 20	Steel	Yield steel	347
BF3O16	Beam FRP rebars 3 x ϕ 16	GFRP	Rupt. GFRP	260 - 265
BF3O32	Beam FRP rebars 3 x ϕ 32	GFRP	Conc. crushing	461 - 579

3. RESULTS

The mid point deflection-load diagrams for all four beams are shown in Figure 2. For BF3O16 the diagram only shows up to approximate a load of 200kNm and a deflection of 93mm. BF3O16 failed in flexural failure with concrete crushing at a load of 295kNm and a deflection of 139mm. The stiffness remained the same until failure, which is indicated with the elongated red line. We can also notice that BF3O16 had a substantial deflection compared to BS6O20 even though the two beams had almost the same load capacity. At the failure load of 295kNm BF3O16 had a deflection of 139mm which was almost 6 times the failure deflection of 24mm for BS6O20 (which additionally was for an 40kNm increased load). The two beams originally designed to have the same strength, BF3O16 and BS5O16 showed significant difference in deflection. BS5O16 failed at a load of 207kNm with a deflection of 26mm. The deflection at the same load for BF3O16 was 94mm, which was 3.6 times more than BS5O16. In Table 1, a comparison between actual and expected failure mode and load is presented. The expected and obtained moment capacity conform quite well.

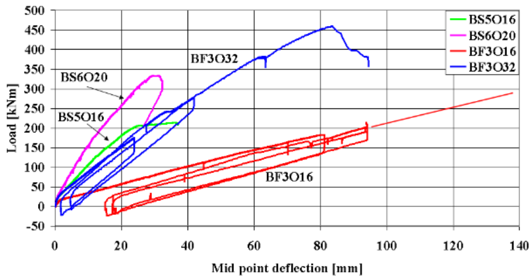

Figure 2 – Load-deflection diagrams for the tested beams [11]

Table 2 – Actual and expected failure mode and failure load. *[11]*

Beam	Actual failure mode	Actual failure load, [kNm]
BS5O16	Flexural failure – yielding of tensile reinforcement	207
BS6O20	Flexural / shear failure – yielding of tensile reinforcement	333
BF3O16	Flexural failure – Concrete crushing	295
BF3O32	Flange separation	459

4. CONCLUSIONS AND FUTURE WORK

Even though the tests performed [11] were successful from the design point of view there are additional research needed. Currently more advanced technology has been developed and it is by fibre optic sensing today possible to follow the strain distribution at every millimeter along the rebars up to failure. This could give a very good understanding of the behavior in the SLS and the ULS stages. It will also then be possible to study the behavior of the anchorage. In addition future tests should also have FRP stirrups which was not the case in the tests by [11].

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