Ductility of sections prestressed with FRP

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ABSTRACT

The main problem in the use of FRP materials for prestressing beams is their linear elastic behaviour; this results in a brittle structural failure. By enhancing the plastic capacity of concrete and by using it, we can increase the ductility of our structures. An analytical study is presented that shows that the same values of energy dissipation can be achieved using steel and FRP prestressed concrete sections, provided we use over-reinforced beams with confined concrete.

INTRODUCTION

Interest in the use of Fibre Reinforced Plastics (FRP) as a reinforcement material started in the early 1980's due to corrosion problems with steel, particularly in hot and wet or saline environments. Small bridge sections and full bridges have been built on a trial basis in Canada, Japan and Germany (Grace et al, 1998) as well as many other places.

The main advantages in the use of FRP prestressing reinforcement are high resistance to corrosion, high strength-to-weight ratio, good fatigue resistance and low relaxation. The main disadvantages are high cost in comparison to steel; lack of design codes, brittle behaviour resulting in reduced structural ductility and lack of understanding of the behaviour of FRP reinforced continuous structures.

FRP reinforcements exhibit elastic behaviour up to failure, without the typical yield plateau of steel (See Figure 1). Therefore, the ductility of FRP prestressed structures has been questioned.

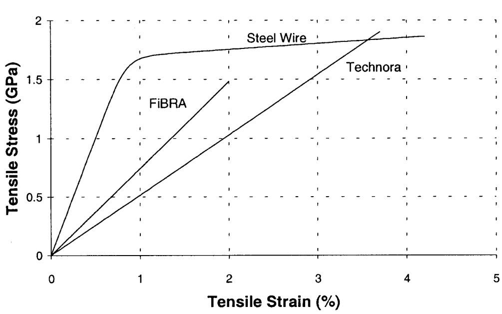

Figure 1 - Stress-Strain Curves for steel, braided aramid rods and pultruded aramid rods.

FRPRCS-5, Thomas Telford, London, 2001.

This study focuses on the determination of the ductility capacity of steel and FRP prestressed concrete beams. The use of AFRP spirally-confined concrete and steel fibre reinforced concrete will be studied in order to enhance the ductility of FRP prestressed concrete beams.

The ductility of reinforced concrete members is a basic requirement of various design approaches and is fundamental to traditional approaches to reinforced concrete design with steel. According to the CEB bulletin 242 (1998), the plastic deformation capacity of reinforced members is indispensable for:

- Warning before failure of statically determinate and indeterminate structures by large deflections;
- Allowing the use of linear elastic analysis *without* moment redistribution, based on the stiffness of the uncracked section, which implicitly assumes a certain rotational capacity in plastic areas;
- Allowing the use of linear analysis *with* moment redistribution, which requires rotation capacity in the plastic areas to allow for the assumed degree of redistribution;
- Allowing the use of elasto-plastic analysis, which is based on the assumption of indefinite plasticity of the member;
- Permitting equilibrium methods which are valid only if compatibility of displacements can be achieved by plastic deformation (e.g. truss models, strut and tie models);
- Giving resistance against imposed deformations (e.g. due to temperature, support settlement, shrinkage, creep);
- Providing an ability to withstand unforeseen local impact and accidental loading without collapse (robustness);
- Permitting redistribution of internal forces in statically indeterminate structures under fire attack;
- Dissipating energy under cyclic (e.g. seismic) loading;

The most conventional way of defining ductility is the ability of a material, section, structural element, or structural system to sustain inelastic deformation prior to collapse, without significant loss in resistance. In the past, ductility measures have been expressed in the form of a ratio called the ductility index or ductility factor. This index is commonly based on stresses and is the ratio between the curvature, rotation or deflection at the ultimate stage and the same quantity at the yielding of the reinforcement. Since the FRP tendons do not yield, the conventional definition of ductility index cannot be used (Naaman & Jeong, 1995).

To overcome the difficulties caused by stress-based definitions, several authors have expressed the ductility capacity in terms of rotation capacity and hence deformability. However, due to the small Young's Modulus and large strain capacity of FRP prestressing tendons, it is possible to achieve the same values of deflection in beams prestressed with steel and FRP. Lees (1997, 1999) reported that using partially bonded tendons, large rotation capacity and high ultimate load capacity could be achieved. Abdelrahman and Rizkalla (1997) tested partially prestressed beams reporting that, provided the failure is controlled by crushing of the concrete in the compression zone, the deflection of beams prestressed by CFRP is equivalent to the deflection of beams prestressed by steel.

However, large deflections prior to failure do not necessarily imply good ductility. When a beam prestressed with FRP fails by rupture of the tendons horizontal splitting cracks appear. These fractures are the result of the release of a large quantity of elastic energy stored in the tendon. The release of such energy at failure could be devastating to the structure and its users. The elastic energy accumulated in the FRP tendon beams can be two to three times larger than in steel tendon beams and the inelastic energy consumed prior to failure can be several times smaller. Naaman and Jeong (1995) proposed a new definition of ductility expressed by the ratio of the elastic and total energies stored in a beam. This definition is also applicable to steel reinforcement and hence provides a common basis for comparison. By applying this index, they concluded that beams prestressed with FRP tendons have substantially lower ductility than beams prestressed with steel tendons.

How can ductility be enhanced?
Macchi (CEB-FIP 1998) pointed out that structural concrete has its beginning in the lucky coincidence of the following facts:

- Complementary properties of steel and concrete
- Intuition of an engineer in exploiting these properties
- Successful application of these intuitive techniques in design and execution

For FRPs to be successful, it is necessary to identify the complementarity of these materials with concrete, rather than to try to make them look like replacement steel. Since FRPs have different properties from steel, it might be necessary to find concrete with different properties. The lack of plasticity of FRP tendons could be supplemented by increasing the plasticity of concrete.

Thus, ductility of FRP prestressed structures might be achieved by forcing the structure to fail by compressive crushing of the concrete. In this way the plasticity of concrete can be used to make up for the lack of plasticity of the tendons. One approach to the enhancement of the limited ductility of concrete is to use concrete confined with aramid FRP spirals (Leung (2000)), which does not significantly improve the concrete strength but the strain capacity achieved is two to three times higher (see Figure 2). By using these spirals to confine the concrete in the compression flange of FRP prestressed beams, it should be possible to considerably improve the ductility index.

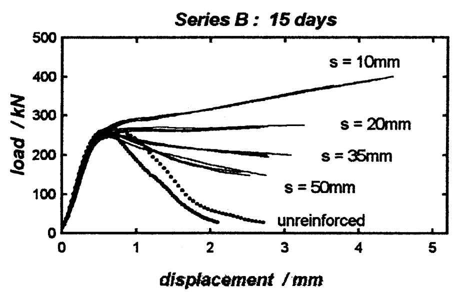

Figure 2 - Experimental Results of Concrete Confined with an AFRP spirals (HY Leung)

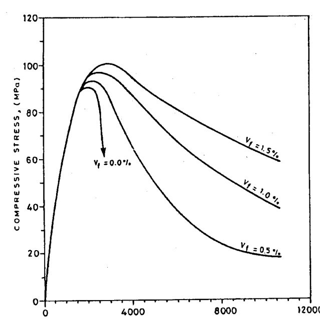

Figure 3 - Effect of Hooked-Steel Fibre Content on Compressive Stress-Strain Curves

Alternatively, adding steel fibres to the mix can enhance the strain capacity of concrete. In Figure 3 the stress-strain relationships of concrete with different percentages of hooked-steel fibres, obtained by Wafa & Ashour (1992), are presented. Although the addition of 1.5% of steel fibres results in a 4.6% increase in compressive strength, the concrete behaves in a much more ductile mode with a higher post-cracking strain capacity.

DUCTILITY ANALYSIS OF A PRESTRESSED CONCRETE SECTION

The ideal method of determining the ductility capacity of a method of construction would be based on the total to elastic energy in the section immediately before failure, and would be measured in tests. But this would require that the specimen be unloaded immediately before failure, which is the only way to determine the stored elastic energy, and is both expensive and difficult to carry out since the moment of failure is not easy to predict. There are also size effects associated with the length of the failure zone which would make comparison difficult. A better approach, at least for comparing alternative forms of construction, is to carry out a computer analysis, in which unloading from any point on the load deflection curve is possible.

A computer program was thus developed that determines the total and elastic/inelastic energy of a reinforced or prestressed section. The total energy stored in a section is defined as the area below the moment-curvature diagram. The elastic energy stored in a section is defined as the area below the unloading curve from that point, while the inelastic energy is the remainder (Fig. 4). The energy ratio is defined as the ratio between the inelastic and total energy. Results will be presented below for unloading from the maximum moment and from a point in the post-peak region when the moment capacity has dropped to half its peak value.

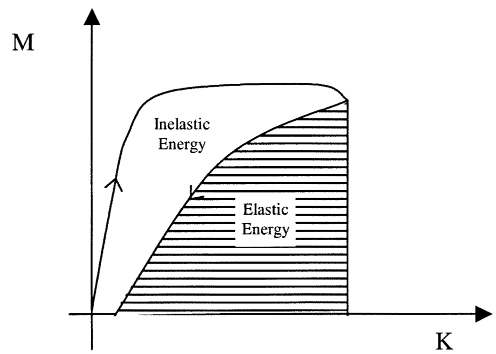

Figure 4 – Elastic and Inelastic Energies

The program determines the moment that must be applied to give a specified curvature, κ; a Newton-Raphson iterative scheme is used to find the bottom strain, ε_b, that achieves longitudinal equilibrium to a chosen accuracy. The section is divided into 50 horizontal strips and the appropriate stress-strain curve is used to determine the stress in each strip (Fig. 5). Numerical integration is then used to determine the bending moment and the axial force.

When a section is being unloaded (and in certain circumstances, when the strain at a section is decreasing while the beam is being loaded), unloading stress-strain relationships are used. For steel, FRP or concrete this is assumed to be linear with a slope equal to the initial Young's modulus. The program keeps a record of the maximum strain that any element reached.

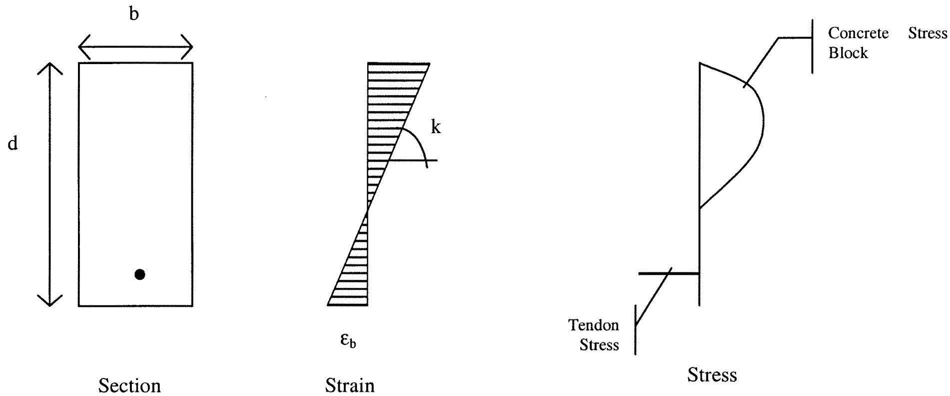

Figure 5 - Strain and Stress Distributions

In order to compare different design philosophies, the program has been used to prestressed concrete sections with the same cracking bending moment (and hence similar design working loads) but with different section depths and amounts of prestressing. This allows the comparison of the full range from under-reinforced to over-reinforced sections. The section depth/width ratio has been fixed at 2.

Material Properties

The program has been used to study prestressed concrete with several combinations of different materials.

Two types of concrete have been used. The first was ordinary concrete with a stress-strain relationship according to the CEB-FIP Model Code 90 with a mean compressive strength, f_{cu}, of 48.9 MPa. The second type was AFRP spirally-confined concrete with a stress-strain curve taken from the experimental results of Leung (2000) for single-spiral concentric compression of cylinders (Figure 2); this concrete had a mean compressive strength of 38.9 MPa and a spiral pitch of 20 mm. The behaviour of the concrete before the peak is similar whether it is confined or not so the Model Code 90 curve was taken before the peak. Subsequently, the confined concrete was considered to behave linearly with an ultimate failure stress of 35.9 and a strain of 0.015.

Two tendon materials were considered, steel and braided aramid rod (FiBRA). The steel stress-strain diagram was considered linear up to yielding and then with an infinite perfectly plastic plateau. For the FiBRA, the material was assumed to be linear elastic to failure (See Figure 1). The elastic properties of both are presented in Table 1.

Material	Young's Modulus (GPa)	Yield Stress (MPa)	Yield Strain (%)	Ultimate Strength (MPa)	Ultimate strain (%)
Steel	200	1800	0.9	1800	∞
FiBRA	68.6	-	-	1480	2.0

Table 1 Tendon Properties

RESULTS

Figure 6 shows the typical response of a conventional under-reinforced steel tendon beam. The solid line shows a moment-curvature response that is as expected; an initial slight hogging curvature at no load due to the prestress followed by a stiff response before the concrete cracks, a loss of stiffness due to cracking, a relatively flat response due to steel yield before a gradual loss of moment capacity at high curvatures caused by crushing of the concrete. The two dashed lines show the unloading response from the peak moment (which is at a not very well-defined position on the yield plateau) and a similar unloading curve from the ultimate moment (half the peak moment). In this type of section most of the plastic capacity of the concrete and steel is used so a large part of the total energy stored in the section is plastic.

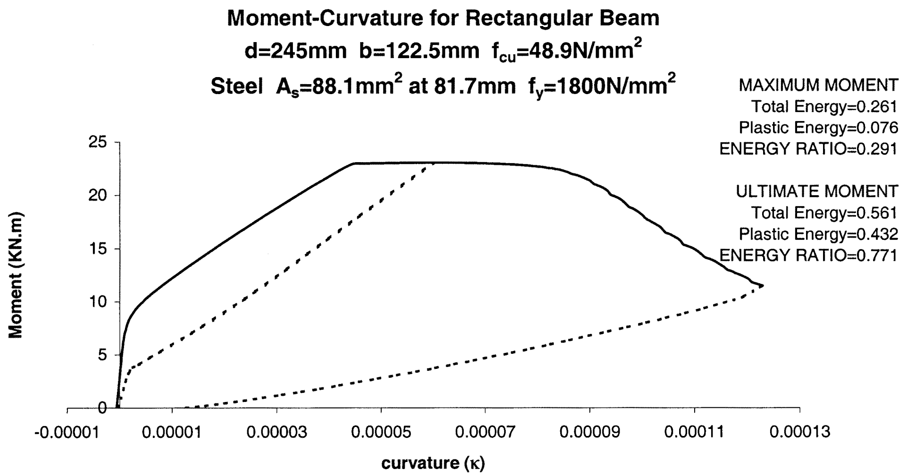

Figure 6 - Moment-Curvature Relationships for Under-Reinforced Steel Tendon Beam with Ordinary Concrete

When the prestressing tendon is with FRP, there is no yielding of the reinforcement, and in an under-reinforced beam the tendon reaches its capacity before the concrete so failure occurs by snapping of the reinforcement, without any descending branch in the moment-curvature relationship. Only a small part of the plastic capacity of the concrete is used so the plastic energy ratio is small (Figure 7).

In over-reinforced prestressed beams with ordinary concrete, the behaviour is not very different for steel and FRP tendons. In both cases, the failure is by crushing of the concrete and both steel and FRP behave linear elastically up to failure. The main difference is the lower Young's modulus of FRP resulting in a slightly higher curvature. In both cases, most of the concrete plastic capacity is used. However, ordinary concrete has a limited plastic capacity (Figure 8).

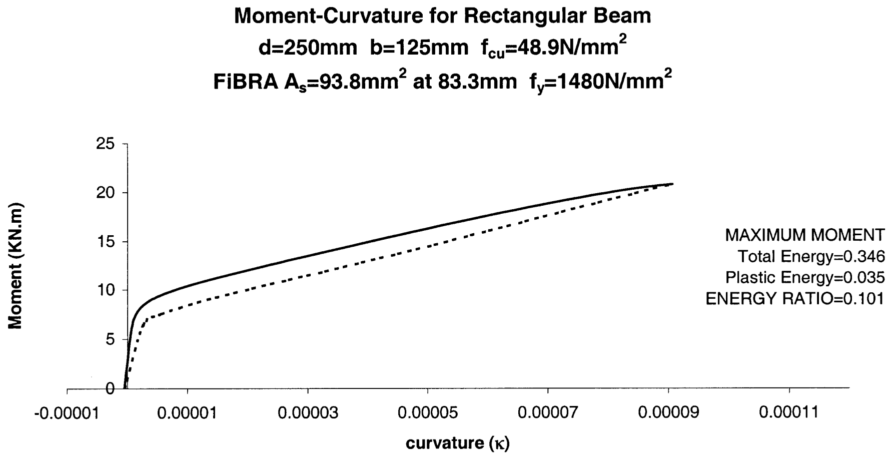

Figure 7 - Moment-Curvature Relationships for Under-Reinforced FiBRA Tendon Beam with Ordinary Concrete

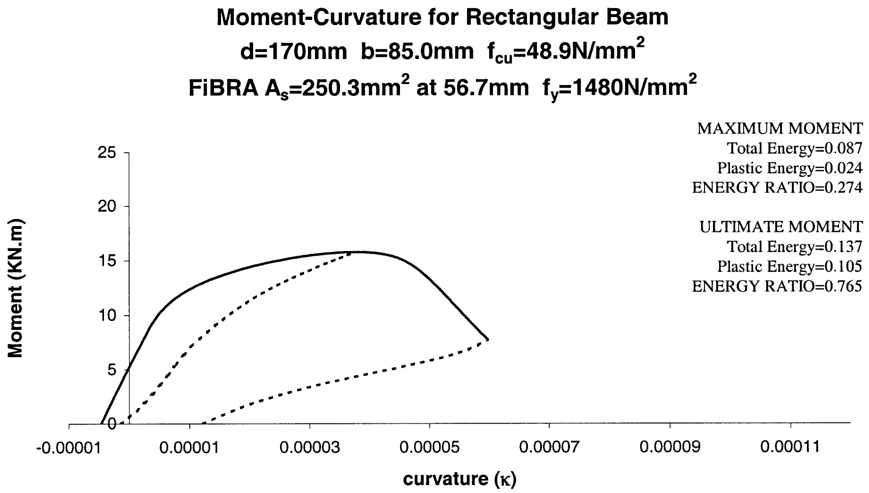

Figure 8 - Moment-Curvature Relationships for Over-Reinforced FiBRA Tendon Beam with Ordinary Concrete

For over-reinforced sections with AFRP spirally-confined concrete (Figure 9) most of the plastic capacity of the concrete is used, and since the concrete has a much higher plastic capacity, most of the total energy stored in the section is plastic. These curves are similar to the under-reinforced steel prestressed concrete section.

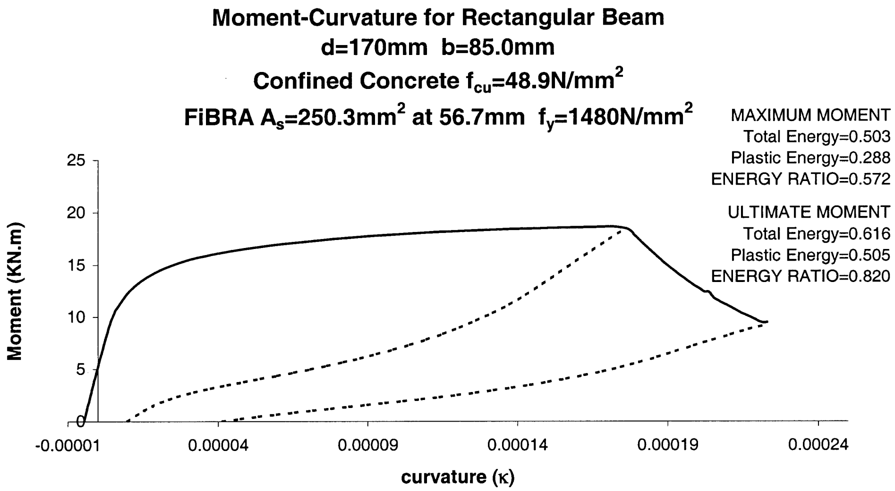

Figure 9 - Moment-Curvature Relationships for Over-Reinforced FiBRA Tendon Beam with AFRP Spirally Confined Concrete

Figure 10 shows the energy dissipation, expressed as the ratio between the plastic and the total energies, for different mechanical reinforcement ratios. There are two sets of lines, the lower lines represents the energy ratio when unloading from the maximum moment in the moment-curvature curve (Max). The upper lines represent the energy ratio when unloading from final moment - half the maximum moment in the post-peak region (Post). Each set has 3 lines corresponding to different material properties; steel tendons with unconfined concrete (StUc), FiBRA with unconfined concrete (FiUc) and FiBRA with Confined concrete (FiCc).

For the two lines corresponding for steel, the lowest values on each curve correspond to a balanced section; the concrete and steel reach their peak capacities at the same time, so they behave elastically, and therefore the energy dissipation is a minimum. On the left of the balanced section, for under-reinforced sections, the energy is mainly dissipated in the steel. On the right of the balanced section, for over-reinforced sections, the energy is only dissipated in the concrete. It is worth noting that many designers regard balanced sections as being the "best" use of the materials, but in ductility terms they are the worst.

For beams with FRP reinforcement, at the lower values of the mechanical reinforcement ratio, failure occurs by snapping of the reinforcement so there is no post peak behaviour. For the confined concrete, since the concrete has a much higher strain capacity, it is necessary to have much larger mechanical reinforcement ratio for the tendon not to snap.

CONCLUSIONS

Figure 10 clearly shows that the energy absorption of sections with FRP reinforcement matches that of balanced sections with steel, provided that the sections are over-reinforced, and that by the use of spirally confined concrete the energy absorption can match that of the best under-reinforced sections with steel.

However, these curves also show that the design philosophy which allows these materials to work efficiently with concrete is significantly different from the philosophy that is conventionally used for sections with steel tendons.

These results have been produced by assuming that the cracking load is significant, rather than the maximum load, and a number of rather arbitrary assumptions have been made to reduce the problem to a simple form. Nevertheless, it is believed that the methods presented here form a rational means of comparing sections of different types, and they also show clearly the changes in principle that need to be applied when using different materials.

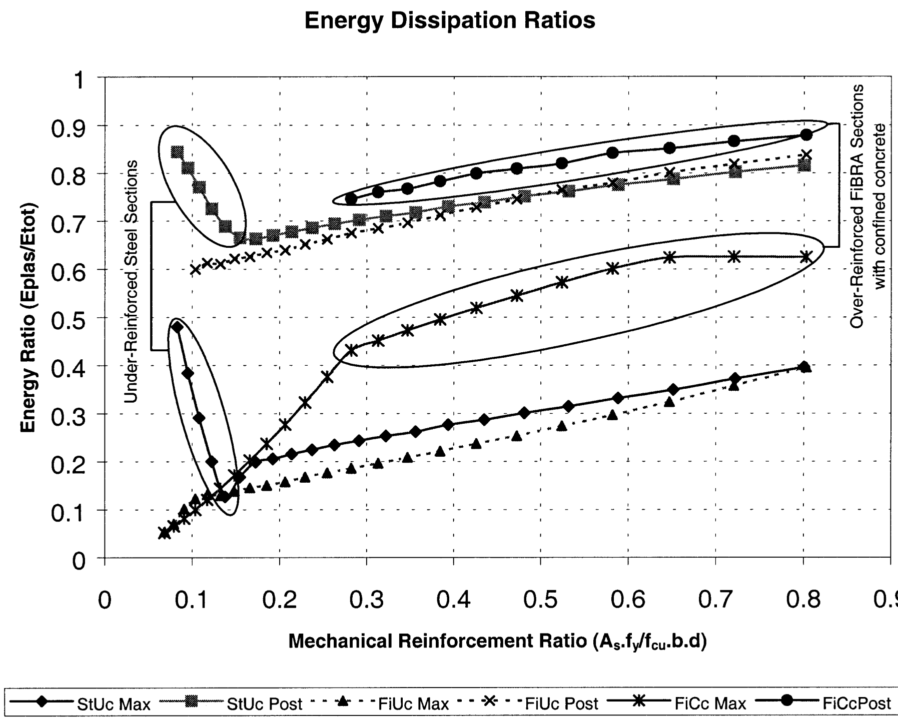

Figure 10 - Variation of Energy Dissipation Ratios with the Mechanical Reinforcement Ratios and Material Properties

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NOMENCLATURE

A_s – Area of tensile reinforcement (mm^2)
b – Section width (mm)
d – Section depth (mm)
ε_b – Bottom strain
f_{cu} – Mean compressive strength of concrete (Mpa)
f_y – Ultimate tensile strength of Steel or FiBRA (Mpa)
K - Curvature
M – Bending Moment (KN.m)
s – Spiral pitch (mm)
v_f – Fibre volume (%)