FRP STRENGTHENING OF CONCRETE STRUCTURES – DESIGN CONSTRAINTS AND PRACTICAL EFFECTS ON CONSTRUCTION DETAILING

Rob Irwin (1) and Amar Rahman (2)

INTRODUCTION

For eight years New Zealand has been to the forefront in the development and use of FRP materials as a means of strengthening civil engineering structures. However, as of this day, there is no national guideline available which sets down recommendations for the design and detailing of FRP for strengthening of civil engineering structures. The authors have spent 3 years researching the technology and the available guidelines worldwide. This paper sets out proposals for interpretation of these guidelines so that they may be used in the detailing of typical NZ structures.

The paper also illustrates the detailing and use of FRP in the strengthening of West Gate Bridge, Melbourne, the world’s largest example of FRP strengthening of a major structure.

1. GENERIC INFORMATION

The use of fibre reinforced polymers (FRP) as reinforcement for structures is rapidly gaining appeal. This is due to the many advantages these materials afford when compared to conventional steel reinforcement or concrete encasements. Their light weight, high strength-to-weight ratio, ease of handling and application, lack of requirement for heavy lifting and handling equipment and corrosion resistance are some factors that are advantageous in repair, retrofitting and rehabilitation of civil engineering structures.

While no country yet has a national design code, several national guidelines [1-6] offer the state-of-the-art in selection of FRP systems and the design and detailing of structures incorporating FRP reinforcement.

However, there exists a divergence of opinion about certain aspects of the detailing between guidelines. This is to be expected as the use of the relatively new material develops worldwide. Much research is being carried out at institutions around the world and it is expected that design criteria will continue to be enhanced as the results of this research become known in the coming years. This development process is akin to that which occurred in the 60s and 70s in the field of prestressed concrete.

The main areas of detailing where this divergence occurs are presented for discussion in this paper. Recommendations are given as to which guideline should be followed. However, as in all design and detailing carried out by responsible engineers, the final decision as to what criteria is chosen must rest with the designer.

2. TYPES AND PROPERTIES OF FRP USED FOR STRENGTHENING

The main fibre types used are carbon (CFRP), glass (GFRP) and aramid (AFRP). GFRP comes in two types – E-glass and AR glass. E-glass is the most common form used but it has the disadvantage that it is attacked by the alkali in fresh concrete or grout. AR-glass (AR = alkali resistant) is the answer to this.

E-modulus (hence ultimate strain and UTS) is the defining property of all FRPs and dictates the preferred uses for each generic type. Typical properties are given in Figure 1.

<table>
<thead>
<tr>
<th></th>
<th>E-modulus (GPa)</th>
<th>Ultimate Strain (%)</th>
<th>UTS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFRP (laminate)</td>
<td>165 - 215</td>
<td>1.3 – 1.4</td>
<td>2500 - 3000</td>
</tr>
<tr>
<td>CFRP (sheet)</td>
<td>240 - 640</td>
<td>0.4 – 1.6</td>
<td>2650 - 3800</td>
</tr>
<tr>
<td>GFRP (sheet)</td>
<td>65 - 75</td>
<td>4.3 – 4.5</td>
<td>2400</td>
</tr>
<tr>
<td>AFRP (sheet)</td>
<td>120</td>
<td>2.5</td>
<td>2900</td>
</tr>
</tbody>
</table>

Figure 1: Typical FRP Properties (dry fibre)

One of the governing properties used in design is the allowable strain in the FRP at ultimate limit state (ULS). It can be seen from Figure 1 that there is a large range (0.4% – 4.5%) of ultimate strain of the various fibres, depending on the type. Hence selection of the correct material for each application is paramount to the design process. Figure 2 provides a guideline to the selection of the type of material for each structural element and whether the requirement is for enhancement of confinement, flexure, axial load, ductility etc.

The bonding of the fibres to the substrate is affected by immersing the fibres in a matrix of
epoxy resin. The bonding material may be applied to either the surface or within slots cut in the cover concrete. Thus the FRP can be defined as a layer (or layers) of fibre embedded in a matrix of epoxy resin and bonded to (into) the concrete.

<table>
<thead>
<tr>
<th>Element</th>
<th>Application</th>
<th>Glass Fibre Sheet (GFS)</th>
<th>Carbon Fibre Sheet (CFS)</th>
<th>Carbon Fibre Laminate (CFL)</th>
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</thead>
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<tr>
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<td>Uni-directional</td>
<td>Uni-directional</td>
<td>Uni-directional</td>
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<tr>
<td>Fibre arrangement</td>
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<td>Straight</td>
<td>Straight</td>
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<table>
<thead>
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<th>Columns</th>
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<th>Fibre arrangement</th>
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</thead>
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<tr>
<td>Confinement</td>
<td>☉ ☉ ☉ ☉</td>
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<td></td>
</tr>
<tr>
<td>Flexure</td>
<td>☉</td>
<td>☉</td>
<td>☉</td>
</tr>
<tr>
<td>Axial Load</td>
<td>☉ ☉</td>
<td>☉ ☉</td>
<td>☉ ☉</td>
</tr>
<tr>
<td>Ductility</td>
<td>☉</td>
<td>☉</td>
<td>☉</td>
</tr>
<tr>
<td>Durability</td>
<td>☉ ☉</td>
<td>☉</td>
<td>NA</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Beams</th>
<th>Fibre direction</th>
<th>Fibre arrangement</th>
<th>Typical application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure</td>
<td>☉ ☉</td>
<td>☉</td>
<td>☉ ☉</td>
</tr>
<tr>
<td>Shear</td>
<td>☉</td>
<td>☉</td>
<td>☉</td>
</tr>
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</table>

<table>
<thead>
<tr>
<th>Walls</th>
<th>Fibre direction</th>
<th>Fibre arrangement</th>
<th>Typical application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear &amp; flexure</td>
<td>☉ ☉</td>
<td>☉ ☉</td>
<td>☉</td>
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</table>

<table>
<thead>
<tr>
<th>Slabs</th>
<th>Fibre direction</th>
<th>Fibre arrangement</th>
<th>Typical application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexure</td>
<td>☉ ☉</td>
<td>☉</td>
<td>☉</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Durability</th>
<th>Fibre direction</th>
<th>Fibre arrangement</th>
<th>Typical application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spalling</td>
<td>☉ ☉</td>
<td>☉</td>
<td>NA</td>
</tr>
</tbody>
</table>

Possible use ☉ ☉ ☉ ☉ Preferred use ☉ ☉ ☉ Special application

**Figure 2 – Application uses for FRP**

Arrangements of the fibres vary from type to type. Figure 3 sets out the comparison and description of the commonly used materials.

There is no fixed rule as to whether sheet or laminate should be used. Usually economy dictates the choice of one system or the other, but sometimes it is a design choice. Carbon (laminate or sheet) appears to be more economic for use in flexural or shear strengthening. Certainly, carbon has better fatigue properties than glass, so where the strengthening is used to carry often occurring fluctuating live loads, carbon should be chosen. Glass, because of its lower E-modulus, is more suitable for use in confinement of concrete, although it can, in certain circumstances, be used for flexural enhancement. Because of its low modulus, glass is seldom used for shear enhancement.

Laminates can only be applied to plane surfaces, therefore carbon, aramid or glass sheet are used on curved surfaces. Carbon sheet, on the other hand, is difficult to cut and handle in thin strips and therefore laminates are preferred, when narrow bands of FRP reinforcement are required.

Bi-directional glass fabrics are used for increasing the shear strength of masonry walls. Lighter fabrics are used where the substrate strengths are low, such as in old and historic masonry or brick buildings.

<table>
<thead>
<tr>
<th>Composite Type</th>
<th>Fibre direction</th>
<th>Fibre arrangement</th>
<th>Typical application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Fibre Sheet (CFS)</td>
<td>Uni-directional</td>
<td>Straight</td>
<td>Increase in flexural and shear capacity; confinement</td>
</tr>
<tr>
<td>Aramid Fibre Sheet (AFS)</td>
<td>Uni-directional</td>
<td>Straight</td>
<td>Special applications</td>
</tr>
<tr>
<td>Glass Fibre Sheet (GFS)</td>
<td>Bi-directional</td>
<td>Woven</td>
<td>Increase in confinement and ductility</td>
</tr>
<tr>
<td>Carbon Fibre Laminate (CFL)</td>
<td>Uni-directional</td>
<td>Straight (partially pre-tensioned)</td>
<td>Increase in flexural capacity</td>
</tr>
</tbody>
</table>

**Figure 3 – Fibre direction, arrangement and typical uses**

The substrate to which the FRP is to be adhered, must have sufficient strength to transfer the loads from the FRP to the structure. Testing of the tensile strength of the substrate by pull-off tests is imperative. Figure 4 sets out the minimum substrate strengths required for each of the FRP materials to be used efficiently:

<table>
<thead>
<tr>
<th>Product</th>
<th>Minimum Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Fibre Sheet (CFS)</td>
<td>&gt; 1.0</td>
</tr>
<tr>
<td>Aramid Fibre Sheet (AFS)</td>
<td>&gt; 1.0</td>
</tr>
<tr>
<td>Glass Fibre Sheet (GFS)</td>
<td>&gt; 0.2</td>
</tr>
<tr>
<td>Carbon Fibre Laminate (CFL)</td>
<td>&gt; 1.5</td>
</tr>
</tbody>
</table>

**Figure 4 – Minimum substrate strengths for various FRP materials.**

In some instances, the design minimum tensile strength may be increased when multiple in-situ pulloff tests indicate that the substrate strength is substantially higher than the figures given in the table. As an example, the design figure used at West Gate bridge was increased from 1.5 MPa to 1.9 MPa as the pulloff tests repeatedly gave tensile strengths in excess of 3 MPa.

[19] gives further information relating to material properties, application methods and quality control.
3. DESIGN CONSIDERATIONS

3.1 Design for flexural enhancement

The design of externally bonded FRP reinforcement (FRP EBR) for flexural members is based on limit state principles and relies upon the composite action between a reinforced or prestressed concrete element and the EBR. In general, strength, ductility and serviceability requirements must all be investigated. The design procedure is analogous to that for reinforced concrete beam and slab sections, with no axial load. The FRP strengthening materials are treated as additional reinforcement with different material properties. The only difference is the initial strains that are present in the concrete and reinforcement, due to the dead load at the time of applying the FRP.

Current design recommendations generally set acceptable levels of safety against the occurrence of both serviceability limit states (excessive deflections, cracking) and ultimate limit states (failure, stress rupture, fatigue). Possible failure modes and subsequent strains and stresses in each material (concrete, reinforcing steel and FRP) should be assessed at ULS and the avoidance of a brittle concrete failure ensured. In respect of the design of FRP systems for the seismic retrofit of a structure, attention is drawn to recommendations given in section 8.1 of [2].

The design procedure must consist of a verification of both limit states. In some cases, it may be expected that the SLS will govern the design.

For buildings (and other applicable structures), fire should also be included as a limit state as it will influence the properties of both the FRP and the adhesive used to attach it to the concrete.

Accidental loss of support from the FRP due to vandalism, impact etc. should be considered.

The safety concepts at ULS, adopted by most guidelines, are related to the different failure modes that may occur. Brittle failure modes, such as shear and torsion, should be avoided. In addition, and for the same reason, it should be guaranteed that the internal steel is sufficiently yielding at ULS so that the strengthened member will fail in a ductile manner, despite the brittle nature of concrete crushing, FRP rupture or bond failure. Hence the governing failure mode of a flexural member will be either steel yielding/concrete crushing (before FRP rupture or de-bonding) or steel yielding/FRP failure (either FRP rupture of bond failure) before concrete crushing. In all cases, verification that the shear (torsion) capacity of the strengthened member is larger than the acting shear (or torsion) forces is necessary. If needed, flexural strengthening must be combined with shear strengthening.

The design approach to strengthened sections is normally based upon a trial and error approach, which can be easily carried out by means of a simple spreadsheet. The initial type, size and length of the FRP reinforcements are selected at random. Then the flexural safety of the strengthened section is checked by analysing its limit states. If the safety check fails, or if the selected FRP strengthening elements are not economical, a new size or type of element is selected and the process is run again. Usually a few iterations are sufficient to arrive at a safe and economical solution.

Custom designed software exists for the design of FRP strengthening using CFRP laminates. One particularly good programme is available in the public domain on www.frp.at. It is written for either the ACI code (US), the British, French and German codes, as well as Eurocode 2. The properties of the FRP used in this programme are those relating to the products manufactured by the owner of the software.

The following assumptions are considered valid for the concept of design of FRP EBR:

- There is a perfect bond between the FRP and the bonded substrate. This is, in fact, achieved without difficulty in practice, and failure, if it occurs, is always in the substrate.
- Plane section remain plane (Bernoulli’s principle).
- The stress-strain responses for concrete and steel reinforcement follow the idealised curves presented in current codes and standards.
- FRP has a linear elastic response.
- The tensile strength of the concrete is ignored.
- Loads which are in place at the time of application of the FRP cause the element being reinforced to act within its elastic limit.
- The existing conditions have been properly evaluated (this includes steel areas and properties, concrete strength, existing moments and shear forces, steel and concrete strains, etc).

For the ultimate and serviceability limit states, the design loading is obtained by multiplying the characteristic dead and imposed loads by appropriate load factors and strength reduction factors. Designers must incorporate factors from design codes acceptable to the location of the works. Figure 5 sets out load factors, partial factors of safety and material reduction factors for some codes. In addition, it is normal to use strength reduction factors when calculating ultimate strength. Some codes (EC2 and BS 8110,
for example) use separate material strength reduction factors for reinforcing steel and concrete, while others (ACI 318, NZS 3101 and Austroads Bridge Design Code, for example) use global strength reduction factors for these two materials.

<table>
<thead>
<tr>
<th>Code</th>
<th>Load Factors</th>
<th>Material Strength Reduction Factors</th>
<th>Strength Reduction Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Dead loads</td>
<td>Live Loads</td>
<td>Concrete</td>
</tr>
<tr>
<td>BS 8110</td>
<td>1.4</td>
<td>1.6</td>
<td>1.5</td>
</tr>
<tr>
<td>ACI 318</td>
<td>1.4</td>
<td>1.7</td>
<td>-</td>
</tr>
<tr>
<td>NZS 3101 &amp; NZS 4203</td>
<td>1.2</td>
<td>1.6</td>
<td>-</td>
</tr>
<tr>
<td>Euro code 2</td>
<td>1.35</td>
<td>1.5</td>
<td>1.5</td>
</tr>
<tr>
<td>Austr loads</td>
<td>1.2</td>
<td>2.0</td>
<td>-</td>
</tr>
</tbody>
</table>

\[
\gamma_{G} \gamma_{Q} \gamma_{C} \gamma_{S} \gamma_{E} \phi
\]

Figure 5 – Load factors, Material Partial Safety Factors & Strength Reduction Factors of different Design Codes and FRP Guidelines.

The methods of incorporating strength reduction material factors for the FRP varies according to the FRP Design Guideline used.

The **UK Concrete Society TR No. 55** [1] recommends 3 separate factors, which relate to the method of manufacture, the type of FRP material and the degradation of the E-modulus over time.

The **German General Guidelines** [3] presently recommend reduction factors by limiting the allowable strain at ULS to between 0.4 and 0.7 of the ultimate strain and at SLS.

The draft **ACI 440** [2], as well as using a global strength reduction factor recommends a strength reduction factor for the FRP of 0.85 and an additional environmental strength reduction factor.

The **fib Bulletin 14** [4] uses FRP material safety factors and also places limitations on the FRP strain at ULS and SLS.

Presently there are no Codes of Practice that include for the use of FRP as a reinforcement material. The designer therefore must take into account suitable limitations on the use of FRP, either by separate material reduction factors as per the UK and fib Guidelines, additional strength reduction factors used in conjunction with the global reduction factor, as for ACI 440, or a fixed upper limit of allowable strain, as per the German General Guideline and ACI 440.

All guidelines limit the stress/strain in the FRP to avoid de-bonding, which can occur in several mechanisms. In addition, due to the general decrease in ductility of a member strengthened with FRP, care must be taken to ensure ductility is preserved, by ensuring the internal steel will sufficiently yield at failure. This is done by limiting the depth of the compression zone at ULS.

### 3.2 Design values for material properties

As mentioned above, at the time of writing there are no Codes of Practice that set down the requirements for the design and execution of concrete strengthening using FRP. However, there are at least eight national guidelines that have been produced by recognised authorities and these can be accepted as state-of-the-art guidelines for the present. Nevertheless it must be recognised that the use of FRP as a strengthening medium, is a relatively new art and that research is being undertaken in many centres worldwide. The results of this research will undoubtedly cause the recommendations to be varied as experience is gained.

The various FRP Design Recommendations treat the strength reduction of the FRP material in different ways.

**UK Concrete Society TR No. 55 [1]**

TR 55 [1] postulates that the partial safety factors to be applied to the characteristic mechanical properties are a function of the type of fibre and the manufacturing/site application process. Thus

\[
\gamma_{mf} = \gamma_{mf} \times \gamma_{mn}
\]

where \( \gamma_{mf} \) depends on the type of fibre and \( \gamma_{mn} \) depends on the manufacturing and/or site application process. Typical values are given in [19].
The accuracy with which the properties obtained from test samples reflect the overall properties of the material will depend on the method of manufacture, the level of quality control and application. It is imperative that material properties used in design are reproduced in the site application. Beware that the many products available in the marketplace possess different properties (as well as qualities).

**German General Guidelines [3]**

The German General Guidelines approach the material strength reduction factors in a different manner. Each manufacturer must obtain an “Approval” for his particular product. This Approval, will normally limit the strain at ULS, based on testing of the particular product. For most products, the Approval limits the strain at ULS to fixed values with an additional reduction factor for shear in slabs. In addition, the authors recommend that the E-modulus of the FRP be reduced by a factor which takes into account the type of fabric (woven or parallel fibres), the type of material and the method of application [19].

**ACI 440 Draft Guideline [2]**

ACI 440 limits the stress due to creep rupture (a SLS condition) to $0.55 \times f_{ju}$. At ULS, an environmental exposure factor $C_e$ reduces the ULS working strain/stress, the factor depending on the aggressivity of the environment in which the FRP is required to work. Strain at ULS is also limited to prevent debonding and peeling. A material reduction factor $\psi_f$ is also applied to the properties of the FRP (dry fibre properties for sheet and laminate properties for laminate), to take into account variations in the manufacturing processes and the type of FRP material (GFRP, AFRP or CFRP).

**fib Bulletin 14 [4]**

fib Bulletin 14 limits the stresses in the FRP by applying FRP material safety factors. When the design is governed by the SLS or an ULS corresponding with concrete crushing or bond failure, the FRP strain at ultimate is rather limited. In this situation the FRP stress $\sigma_f$ at ULS is considerably lower than the tensile strength, so that the design tensile strength is not governing. To verify this or hence in those cases where the ULS is determined by the FRP tensile failure anyway, reference is made to the design tensile strength, where

$$f_{fd} = f_{pk} \frac{\varepsilon_{fue}}{\gamma_f \varepsilon_{fum}}$$

and $f_{fd}$ is the design value of the FRP tensile strength

$f_{pk}$ is the characteristic value of the FRP tensile strength

$\varepsilon_{fue}$ is the ultimate FRP strain, and

$\varepsilon_{fum}$ is the mean value of the ultimate FRP strain.

The values for the FRP material safety factor $\gamma_f$ are suggested in table 6.6 of [4]. Bulletin 14 [4] points out that these factors are subject to further study, because of the current lack of comprehensive study. The ratio $\varepsilon_{fue} / \varepsilon_{fum}$ normally equals 1.0.

### 3.3 End conditions and development lengths

Members strengthened externally with FRP can fail prematurely as a result of local FRP separation. This can be caused by three different mechanisms: peeling, debonding and cover tension delamination.

Peeling failure may occur at the ends of the FRP where a discontinuity exists as a result of the abrupt termination of the plate. TR 55 [1] reports this phenomenon is normally associated with concentrated shear and normal stresses in the adhesive layer due to the FRP deformation that takes place under load. Peeling failure usually results in ripping of the cover concrete off the adjacent layer of steel reinforcement.

Debonding, unlike peeling, mostly occurs away from the plate end. It occurs if the bonding material is not up to specified strength or has not been properly applied. Debonding failure may also be indicative of inadequate preparation of the concrete substrate. More commonly, however, it is associated with the formation of wide flexural and shear cracks that occur as a result of the yielding of the embedded steel bars. The wide cracks generate high stresses in the FRP across the crack, which can only be dissipated by debonding. The cracks can then propagate towards the plate end, leading to FRP separation failure.

Cover tension delamination results from the normal stresses developed in a bonded FRP laminate. With this type of delamination, the existing reinforcing steel essentially acts as a bond breaker in a horizontal plane, and the reduced area of bulk concrete pulls away from the rest of the beam. The result is the entire cover layer of concrete delaminating from the member.

Peeling or end plate separation failure will be avoided by addressing two criteria:
(i) Limiting the longitudinal shear stress between the FRP and the substrate

(ii) Anchoring the FRP by extending it beyond the point at which it is theoretically no longer required.

The word “theoretically” has produced intense international discussion. See section 6.4.2.1 and 6.4.2.2 of [19] for commentary and recommendation in this regard.

As a word of caution, the authors consider the limitations imposed by TR 55 [1] as deficient in certain aspects and designers should familiarise themselves with the limitations exposed by TR 55 and make the appropriate engineering decision for themselves. The authors recommend that the method developed by Onken & vom Berg [16] [reproduced in [19] [Figure 6] be used to determine end conditions in flexure.

Figure 6 – End condition considerations for FRP used in flexure – Onken and vom Berg [16]

3.4 Design for shear strengthening

Externally bonded FRP sheets can be used to increase the shear strength of reinforced concrete beams and columns. The shear strength of columns can be improved by wrapping with a continuous sheet of FRP to form a complete ring around the member. Shear strengthening of beams however, is likely to be more problematic when the beams are cast monolithically with slabs. Attention needs to be paid to anchoring the FRP at or through the beam/slab junction, ensuring that full anchorage occurs above the neutral axis (i.e., in the compression zone). The FRP should be placed such that the principal fibre orientation, $\beta$, is either 45° or 90° to the longitudinal axis of the member.

Increasing the shear strength can also promote ductile flexural failures.

ACI 440 [2] recommends that beams and columns on moment frames resisting seismic loads, at locations of expected plastic hinges, or at locations where stress reversal and post-yield flexural behaviour is expected, should only be strengthened for shear by completely wrapping the section with strips spaced less than $h/4$ (clear spacing) where $h$ is the depth (width) of the member.

Types of shear wraps

There are three types of shear wraps suitable for increasing the shear strength of rectangular beams or columns (Figure 7).

![Figure 7 – types of wrapping systems for shear reinforcement](image)

Complete wrapping of the FRP around the section is the most efficient, followed by the 3-sided and the 2-sided wrap. In beam applications, especially T-beams where the neutral axis is mostly found in the slab portion of the beam, it is necessary to ensure the FRP is anchored in the compression zone (above the neutral axis). This is achieved by passing the FRP strip through slots cut in the slab and anchoring it on the top of the slab. Alternatively, anchors capable of transmitting the force from the FRP through into and beyond the mild steel reinforcement stirrups, can be used, if proper detailing of the load transfer from FRP to anchor is considered. The 3-sided and 2-sided wraps should be used with absolute caution.

The UK Concrete Society TR 55 [1], German General Guideline [3] and ACI 440 (draft) [2] all treat the shear situation differently. Depending on whether you are in an area where ACI 318 is used (global safety factors), or in Europe (partial material reduction factors), the requirements are quite different. Designers are recommended to study the appropriate code/guidelines, for detailed use.

Current research on shear strengthening with bonded FRP suggests that, as with conventional reinforced concrete, shear failure will occur due to two basic mechanisms, diagonal tension (resisted by shear stirrups) and diagonal compression...
(resisted by inclined concrete compression struts in tie and strut model).

For a detailed summary of the requirements of each of the three guidelines [19] should be consulted. This summary is not exhaustive and readers are advised to consult the appropriate document they are working with.

The authors recommend that designers use a method which makes sure connection of the FRP shear strengthening members follow the internal truss structure (Figure 8). In most cases, this will mean the anchorage of the FRP strips will be located within the compression zone of the concrete.

Spacing of FRP Laminate Strips

As in the case with steel shear reinforcement, the spacing of FRP laminate strips should not be so wide as to allow the full formation of a diagonal crack without intercepting a strip. For this reason, if laminate strips are used, their spacing should not exceed the lesser of \(0.8d\) and \(\frac{w_f + d}{4}\) where \(d\) the effective depth of the beam and \(w_f\) the width of the FRP laminate strips.

3.5 Design for Axial Load Enhancement

Retrofitting to enhance the axial compressive strength of concrete members using FRP material is commonly used. By wrapping a column with an FRP jacket, the shear, moment and axial load capacity, as well as the ductility, are improved. The column is wrapped with the FRP fibres in the hoop direction and this provides significant confinement to the concrete, thus leading to improvement in performance.

GFRP and CFRP are both very effective in enhancing axial performance. Creep of GFRP is not a concern with column wrapping because under normal service loads, the jacket remains virtually stress free.

Both circular and rectangular columns are able to be enhanced with FRP jackets. The most effective situation is the circular or oval jacket, but reasonable enhancement of rectangular columns is achievable, although less than that of square or circular columns.

The original theory and experimental work was carried out by Priestley [7] in 1988 and this has been followed by much research by others. The basis of the theory used widely today comes from the research work carried out by Wang Yung-Chih from 1996 – 2000, at the School of Engineering, University of Canterbury, New Zealand [8].

Figure 9 shows a cross section of a reinforced concrete rectangular column that is confined by an FRP jacket. The concentric compressive load carried by a short reinforced concrete column is the combination of the compressive loads carried by the concrete and the longitudinal reinforcing bars respectively.

Wang [8] assumes that the ultimate limit state in a concentrically loaded column is associated with 1% axial strain. With Poisson’s ratio conservatively assumed to equal \(\nu = 0.5\), at this strain level, the transverse strain at 1% axial strain is equal to 0.5%.

Wang [8] also postulates that the reinforcing steel behaves as an elasto-plastic material. The nominal compressive strength carried by the concrete, results from the stresses in three distinct regions shown in Figure 9. He further postulates that at 1% axial strain the unconfined concrete has reached its peak strength and has degraded to a residual strength to \(0.3f'_c\).

Figure 8 - Connection of the FRP shear strengthening to the internal truss structure

Figure 9 – Dual Confinement Effect on a rectangular column with FRP jacket and internal steel hoops.
For design purposes it is necessary to reduce the nominal concentric strength to account for variations in the materials properties, scatter in the design equation, bending of the columns, nature and consequences of failure and reduction in load carrying capacity under long-term loads. This is done by strength reduction factors and material reduction factors.

The compressive load carried by the concrete results from the loads sustained by three distinct regions, viz, the unconfined concrete region, the effective area confined by the FRP jacket and the effective area of the concrete confined by both the FRP jacket and the steel stirrups. Hence the entire uni-axial stress-strain relationship for a concentrically loaded column wrapped with an FRP jacket can be obtained if the constitutive stress-strain realtionships for each of the regions and for the reinforcing steel are known. The determination of the compressive strength of the confined concrete and the evaluation of the lateral confining pressure due to the elastic jacket and internal reinforcing stirrups is then able to be calculated [8].

3.6 Conclusions on design aspects

It is not possible, in the space allocated for a paper such as this, to provide a total picture of the state-of-the-art of this technology. However, suffice it to say that the use of FRP materials will greatly increase in the coming years. They have served their apprentiship and have proven to be economical and beneficial substitutes for the alternative methods of strengthening. The fact that we continue to upgrade our structures to increase ductility, load carrying capacity and seismic resistance will dictate that these materials will continue to be a strong participant in such activities.

New Zealand has lead the way for over 30 years in concrete technology – is it not time now for NZ to decide it must produce its own set of guidelines for use in this country? This is something the industry should carefully consider.

4. A SPECTACULAR EXAMPLE – WEST GATE BRIDGE, MELBOURNE

The remainder of this paper describes what is considered to be the world’s largest application of FRP reinforcement in the strength enhancement of a prestressed concrete bridge. Designed in the 1960s, construction of the West Gate Bridge in Melbourne, Australia was completed in 1978. Due to a more than seven-fold increase in daily vehicle usage since its opening, the owner, state road authority VicRoads, decided to increase the number of lanes over a 670 m length of one of the concrete approach viaducts. This was achieved by utilizing a service lane without additional construction works on the superstructure. In addition, an increase in design loads and changes in design philosophy in the most recent relevant Australian guidelines [24] compared to the 1960s code [25], prompted VicRoads to commission a study for the strength enhancement of West Gate Bridge to meet these new requirements. A Design and Construct approach was chosen and the successful bid team provided an innovative solution which incorporated external post-tensioning located within the box cells, together with CFRP sheets and laminates. Structural analyses, design and detailing of the strength enhancement system was carried out by URS Australia Pty Ltd who engaged Bow Ingenieure of Braunschweig, Germany as specialist designers in the FRP field, while construction was carried out by Abigroup/Savcor Joint Venture. Savcor is an associate company of Contech. In addition to supply of the FRP reinforcement, BBR Systems Ltd, Zurich provided the initial FRP alternative concept and during the construction, technical support.

Designed in the 1960s in accordance to the loading requirements in effect at the time [25], West Gate Bridge was finally opened for traffic in 1978. The current daily usage is approximately 150'000 vehicles representing a seven-fold increase in traffic volume since West Gate Bridge was first opened.

Figure 10 – Overview of West Gate Bridge – centre section is a cable stayed steel box girder.

The bridge consists of continuous curved approach viaducts, constructed from precast concrete segments post-tensioned together, and a steel stay-cable bridge with a multi-cell deck cross-section (Figure 10). Each of the two pylons supports six cables in a single-plane fan arrangement. The stay-cable bridge main span, side spans and approaches span a total of 848 m. The curved approach viaducts have lengths of 670m and 871m for the west and east viaducts respectively.

The approach viaducts comprise a central three-cell box girder element flanked by precast cantilever frame elements (Figures 11 & 13). A
precast deck slab element spans the cantilever units, with a 76 mm in-situ topping cast onto the precast deck slabs completing the deck cross-section (Figure 11). The complete deck is 35.62m in width with a depth of 3.9m and spans approximately 67m between piers. The centre-to-centre distance between consecutive cantilever segments is approximately 3.7m.

Figure 11 – segmental construction of approach viaducts

The bridge was designed to carry eight lanes of traffic, four lanes in either direction in addition to two emergency (service) lanes. In order to accommodate increased traffic during peak hours, VicRoads decided to increase the number of lanes over the eastbound segment of the western approach viaduct. This is achieved by utilizing an existing service lane over the 670m length of the viaduct thus obviating the need for superstructure expansion (Figure 12).

Figure 12 – Accommodation of increased traffic by the addition of an additional lane (by utilisation of service lane on LHS.

4.1 Strengthening Requirements

The increased traffic loading requirements, due to the change in usage, was one of the factors which prompted VicRoads to invite tenders for Design and Construct proposals to accommodate this increase. In addition to this, the Australian bridge design loading requirements currently in force [24] allow for traffic loads higher than those considered acceptable at the time West Gate Bridge was designed [25]. It is also to be noted that the former guidelines are based on a limit state format while the latter applied a working stress design philosophy. Structural analyses conducted in 1999 by Hardcastle and Richards [26], now URS Australia, identified several areas of the bridge which had insufficient capacity:

- Global hog of the box girder over the piers at SLS
- Combined shear and torsion near the piers at ULS.
- Local sag moments in the deck slab at ULS.
- Local bending capacity in the cantilever frame at ULS.

The assessment looked at a range of loading conditions with associated deficiencies and estimated costs for repair.

4.2 The Strengthening Scheme

The concept developed and offered by the tender team proposed enhancement of the box girder
flexural capacity by means of conventional externally post-tensioned tendons, located within the twin cell box girder. Other areas of concern were addressed by means of externally bonded reinforcement in the form of carbon fibre reinforced polymer (CFRP) using both unidirectional sheets and laminates. VicRoads chose the solution using FRP reinforcement instead of bonded steel plates for reasons of substantial economy. Additional material costs of FRP over steel were negated by practical aspects, as no heavy lifting, cutting or welding equipment would have been required as is the case with steel, and labour hours would have been significantly less. In addition, no disruption to traffic was necessary throughout the strengthening operations and there are no problems associated with corrosion protection.

The distribution of the FRP reinforcement was as follows:

4.2.1 Torsional strengthening of the box girder required the use of CFRP laminates distributed around the external circumference (i.e. external webs and soffit slab) of the box girder elements near the piers where the torsional demands were large. In other regions (i.e. adjacent two segments), only the soffit was strengthened due to lower torsional demands and adequate internal steel reinforcement in the web and top slab. Continuous shear flow reinforcement was achieved by slotting and bonding the laminates into the underside of the box girder top deck slab. At the lower corner of the box girder, the web and soffit slab laminates were spliced by means of CFRP sheets wrapped around the bottom corner of the box girder (Figure 14). This area required special detailing to ensure that continuity of the reinforcement was achieved from laminate through sheet through laminate.

4.2.2 Flexural capacity enhancement of the precast deck slab elements spanning between the cantilever frames was provided by (CFRP) laminates. In the negative moment regions, i.e. above the piers, the laminates were bonded into slots cut into the deck. Positive moment capacity was enhanced by means of laminates glued to the soffit of the slabs in the span direction (Figure 15).

4.2.3 The flexural capacity of the cantilever frame elements was increased by means of steel plates glued and bolted to the compression strut and CFRP laminates glued to that portion of the deck slab soffit which acts with the frame as a composite element (Figure 16).

Details of the design philosophy can be found in [20]-[23]. The assumptions applied in the design of reinforced concrete elements using conventional materials, i.e. plane sections remain plane, strain compatibility, equilibrium of forces acting at a cross section, constitutive (stress/strain) behaviour, are also applicable when strengthening with FRP reinforcement. The potential for sudden brittle failure is alleviated by relying on safety factors which take the linear stress/strain behaviour of the FRP reinforcement into consideration.

In brief, the design was based on TR 55 [1] and certain aspects of the German General Guidline [3] in areas where TR 55 was found to be deficient. These were combined with the loading conditions required by Austroads Bridge Design Manual [24]. Key features of the design were:

- **Limiting strains:** In most situations designs were controlled by upper limits of FRP strain at the ULS as follows:
  - *Flexure:* to avoid FRP separation at failure due to debonding, the FRP strain was limited to 0.8% at ULS for uniform moments and 0.6% if combined shear forces and bending moments were present.
  - *Shear and torsion:* The FRP was designed to achieve the required resisting forces at ULS at a strain not exceeding 0.4%.

- **Anchorage:** It was considered that TR 55 was deficient in this area (as are other guidelines) and the German General Guideline approach was adopted.

- **Design for flexure:** TR 55 was used with the exception that a parabolic-rectangular compression block was adopted in accordance with the 1990 CEB Model Code. Australian Bridge Design Code materials factors were applied to internal concrete and steel reinforcement resisting forces.

The maximum allowable spacing of laminates on the slabs was according to the German Guideline. They were the lesser of:
- 0.2 x span
- 5 x slab thickness
- 0.4 x cantilever length

![Figure 14 – Detail of box girder torsional reinforcement with CFRP](image)
- **Design for Shear and Torsion:** This was based on TR55 however, as TR 55 has no guideline specific to torsion a FRP strain limit of 0.4% (0.004) as for shear was applied in the same way for torsion.

![Figure 15 – Strengthening details of cantilever and precast deck segments](image)

### 4.3 Installation of the FRP

Installation of the FRP reinforcement was carried out in accordance with the detailing prescribed by the JV designers, URS and the QA requirements of BBR Systems. In order to guarantee optimal utilization of the unique strength characteristics of FRP reinforcement, proper application procedures were outlined and followed. To this end, a comprehensive Quality Assurance document was drafted by Abigroup/Savcor with the assistance of BBR Systems, outlining not only the proper installation procedure but also a quality control programme that guaranteed the designated performance levels. It is to be noted that traffic flow was not disrupted during the FRP installation and the bridge was operational to its full capacity. The mechanical properties of the FRP materials are given in Figure 16.

The high quality of the concrete surfaces of the precast elements at West Gate Bridge, i.e. lack of uneven surfaces, low porosity, etc even after more than twenty years since its construction, obviated the need for extensive surface treatment and/or preparation. Regular pull-off testing of the concrete substrate ensures that a tensile bond strength exceeding 1.5 MPa was achievable at all surfaces where the FRP reinforcement was bonded.

Cement laitance at the concrete surface was removed by grit blasting and the resulting surface vacuumed with an industrial vacuum cleaner. The lack of surface irregularities meant that the prescribed levelling mortar was not required. Environmental conditions, such as dew point, ambient temperature and relative humidity, were regularly tested prior to application to ensure lack of moisture at the FRP-concrete interface.

![Figure 16 – Properties of the BBR FRP products used at West Gate Bridge.](image)

<table>
<thead>
<tr>
<th>Product</th>
<th>Description</th>
<th>Dry Fibre E-modulus (GPa)</th>
<th>Ultimate Strain (%)</th>
<th>Ultimate Tensile Strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CFL 165</td>
<td>1.4mm thick pultruded laminates with widths of 20-120mm</td>
<td>165</td>
<td>1.4</td>
<td>2'500-3'000</td>
</tr>
<tr>
<td>CFS 240</td>
<td>Unidirectional sheets, 300mm wide with 300 gm/m² carbon fibre in warp (0°) direction</td>
<td>240</td>
<td>1.55</td>
<td>3'800</td>
</tr>
</tbody>
</table>

After proper surface preparation the BBR CFL 165 laminate was glued to the surface by means of the BBR 150 epoxy adhesive, a thixotropic two-component thermosetting resin. The adhesive was applied with a spatula directly onto the laminate with more epoxy applied in the middle than at the edges. The CFL laminates were then pushed on to the concrete surface with finger pressure until the adhesive was discharged from the edges of the laminate. The thixotropic consistency of the adhesive ensured that no further pressure was required. The excess adhesive was then wiped off.

Installation of the BBR CFS 240 sheets was achieved by first applying a coat of BBR 125 saturant, a low viscosity two-component

![Figure 17 – View of large working platform used to access the underside of the structure](image)

![Figure 18 – Travelling working platform in place](image)
thermosetting epoxy resin, to the surface. The sheet was then pressed and rolled onto the surface and additional amounts of saturant were applied to the sheet until the fibres were completely saturated.

**Figure 19 – application of the CFRP laminates to the underside of the box girder.**

After curing of the epoxies, further tests were conducted on the FRP to ensure proper installation procedures had been followed. Hollow, drummy areas, identified by tapping the surface of the FRP, are indicative of insufficient saturant, which could result in potential debonding and can be treated by epoxy injection. Pull-off tests, the frequency of which decreased with increased confidence in the installation procedure, were also conducted to ensure proper bonding of the FRP reinforcement to the substrate.

**Figure 20 – Provision of additional tensile capacity by inserting CFRP into slots cut through the asphalt into the deck**

4.4 Conclusion to West Gate Bridge Section

The increasing appeal of FRP composites for use in the rehabilitation, retrofit and/or strengthening of civil engineering structures can be attributed to the many advantages of this type of material over conventional steel reinforcement. Durability, corrosion resistance, low weight, high strength and ease of installation are some of the factors which favour the use of FRP reinforcement over bonded steel plates. What can be considered to be the world's largest application of FRP reinforcement in the strengthening of a reinforced concrete bridge is now successfully completed. Approximately 40 km of CFRP sheets and laminates have been used to upgrade the strength capacity of this economically important bridge. The advanced composite material was used in flexure, shear and torsion applications.

The Design and Construct contract was competitively won with an innovative solution to conventional steel plate technology proving to be more economical and quicker to execute. The pooled skills of the contractor, designer and technical advisors during the concept and tender stage provided the winning innovative concept. This illustrates the advantages of D & C projects of this type.

Whilst design methods for FRP are still in their formative years, the contractor AbiGroup-Savcor JV and its designers, URS Australia assisted by bow ingenieure, Germany, aided by technical assistance in FRP matters from BBR Systems, have pioneered the large scale use of FRP for major bridge rehabilitation. The knowledge derived during the design and construct exercise will enable future major projects to proceed with confidence.

FRP for use as a strengthening medium has an extremely bright future in the rehabilitation of concrete structures of all descriptions.

5. REFERENCES


[25] National Association of Australian State Road Authorities 1965; Highway Bridge Design Specifications; NAASRA.

[26] Hardcastle & Richards, 2000; Report on West Gate Bridge Concrete Approach Spans, Vehicular Loading Assessment.

26 August 2002