COMPARISON BETWEEN FRP REBAR, FRP GRID AND STEEL REBAR REINFORCED CONCRETE BEAMS

Conference Paper · June 2003

DOI: 10.1142/9789812704863_0102

7 authors, including:

Lawrence C Bank
City College of New York

Doug Gremel
Owens Corning

design equation yields a 'safe' design. This brings in the discussion basic issues regarding the safety level obtainable with a calibration procedure.

For the sake of discussion, through a set of Monte Carlo simulations of FRP-flexural-strengthening designs, a distribution of the partial safety coefficient for the FRP strength was obtained and the value $\gamma_f = 1.33$ corresponding to a 50% fractile was selected. This implies that 50% of the strengthening designs with FRP that use this value would have a lower reliability than the target one. As always, the problem is to decide *how safe is enough*[6], but discussion on this debated issue would require to introduce economic considerations. This clearly goes beyond the scope of this paper and shall be the object of further studies.

Moreover, given that the same 'material' partial safety factor should be used for FRP regardless of the particular strengthening measure to design, the scope of subsequent works shall be to extend the methodology to include other strengthening measures, such as in shear, for confinement, for anchorage zones, etc., where the 'internal' γ_f and 'external' partial safety factors $\gamma_{R,i}$ are simultaneously calibrated.

REFERENCES

1. Eurocode1: ENV 1991-1, Basis of design and actions on structures, 1996.
2. Madsen, H.O., Krenk, S., and Lind, N.C., Methods of structural safety, Prentice Hall Inc., Englewood Cliffs, New Jersey, 1986.
3. Melchers, R.E., Structural reliability analysis and prediction, 2nd Ed., Wiley, Chichester, England, 1999.
4. Melchers, R.E., "Assessment of existing structures – Approaches and research needs", J. of Structural Engineering, ASCE, 127(4), 2001.
5. Monti, G., Santini, S., "Reliability-based calibration of partial safety coefficients for FRP", J. of Composites for Construction, ASCE, 6(3), 2002.
6. Rackwitz, R., "Optimization – the basis of code-making and reliability verification", Structural Safety, 22, 27-60, 2000.

FRPRCS-6, Singapore, 8–10 July 2003
Edited by Kiang Hwee Tan

COMPARISON BETWEEN FRP REBAR, FRP GRID AND STEEL REBAR REINFORCED CONCRETE BEAMS

M. OZEL, L.C. BANK, D. ARORA AND O. GONENC

Department of Civil Engineering, University of Wisconsin – Madison
1415 Engineering Drive, Madison, WI 53706, USA

D. GREMEL

Hughes Brothers, Inc.
Seward, NE 68434, USA

B. NELSON AND D. McMONIGAL

Strongwell
Chatfield, MN 55923, USA

A series of tests were conducted on concrete beams that were designed to have the same load carrying capacity and have the same geometry but different types of reinforcement. All beams were 101 in (2565 mm) long and had a rectangular cross-section measuring 8 in by 12 in (203 mm by 305 mm). All beams were designed using current American Concrete Institute (ACI) codes or design guides to develop a nominal moment capacity of 750 kip-in (85 kN-m) corresponding to a ultimate failure load of 50 kips (222.5 kN). Concrete design strength was 5000 psi (34.5 MPa). The beams were tested in four-point bending with 1/3 point loads. Three different types of reinforcement systems were used: (a) commercially manufactured GFRP main bars, GFRP U-shaped stirrups and GFRP top bars; (b) commercially manufactured pultruded GFRP grids consisting of I-shaped main bars, flat vertical and top bars, and round interlocking transverse cross-bars; (c) commercially produced grade 60 deformed steel reinforcing bars and closed-loop stirrups. Beams were tested to failure in displacement control. The methods used to design the different reinforcement systems are reviewed. The results of the tests are compared with the code predicted capacities. The failure modes and load-deflection responses of the beams are discussed. The behavior of the three types of beams are compared in terms of failure mode, ductility and serviceability. The results should enable designers to make a one-to-one comparison between FRP reinforced beams and steel beams, designed for the same nominal load carrying capacity.

INTRODUCTION

A total of seven beams reinforced with steel, GFRP rebars or GFRP grid were manufactured and tested. The beams were designed according to the current design codes: ACI 318-99[1] for the steel reinforced beams and ACI 440.1R-01[2] for the GFRP rebar reinforced beams. The design moment and total load capacity for the beams were 750 kip-in and 50 kips, respectively. In this paper, design and fabrication as well as comparisons of these beams in terms of reinforcement ratio, axial stiffness and failure modes is presented.

EXPERIMENTAL STUDY

Initially, one GFRP grid reinforced (SD3-90), two identical GFRP rebar reinforced (FRP 1 and FRP 2) and two identical conventional steel rebar reinforced (Steel 1 and Steel 2) beams were designed and fabricated. When the beams were tested, it was observed that the steel rebar reinforced beams carried approximately 20 kips more load than they were actually designed for. Therefore, two additional steel rebar reinforced beams (Steel 3 and Steel 4) with a lower reinforcement ratio were designed, fabricated and tested.

All the beams had an 8 in by 12 in cross-section. They were 101 in long. SD3-90 was reinforced with four 2 in high T-bars for longitudinal reinforcement and three 1.5 in by 0.2 in rectangular strips for vertical reinforcement placed 6 in on-center along the length of the beam. Three-part proprietary connecting bars called crossbars with a combined diameter of 0.5 in were used as the transverse components of the grids[3,4,5]. FRP 1 and FRP 2 were reinforced with three #7 glass fiber reinforced plastic (GFRP) rebar and #3 open stirrups spaced at 4 in on-center along the length of the beam. Steel 1 and Steel 2 were reinforced with three #6 rebars, whereas, Steel 3 and Steel 4 had three #5 rebars. The steel reinforced beams had #3 stirrups at 5 in on-center along the length of the beams. All the beams had #3 top bars to tie the reinforcing cages together and to provide stability. The properties of the FRP reinforcements used in the design calculations are summarized in Table 1.

Design

The beams were designed for a nominal moment capacity of 750 kip-in. The concrete design strength, f_c' was 5000 psi. SD3-90 was designed as part of a prior set of experiments on FRP grid reinforced beams. The details

Table 1. Properties of GFRP grid and GFRP rebar

Bar Size	*Bar Diameter (in)*	A_f (in^2)	f_{fu} (ksi)	E_f (ksi)
2" T	N/A	0.542	85.8	4771
# 3	0.375	0.131	110	5920
# 7	0.875	0.593	85	5920

1 ksi = 6.9 MPa, 1 in^2 = 645 mm^2, 1 in = 25.4 mm

of the design are reported elsewhere[4,5]. The FRP reinforcement ratio, ρ_f, and balanced reinforcement ratio, ρ_{fb}, for FRP1 and FRP 2 were calculated using equations ACI440.1R-01[2]. The balanced reinforcement ratio, ρ_f, for the steel beams was calculated using ACI 318-99[1]. The nominal moment capacities for the FRP beams were calculated using ACI440.1R-01. The details of the flexural design are provided in Table 2.

Table 2. Flexural Design

Beam Name	A_f (in^2)	b (in)	d (in)	ρ_f	ρ_{fb}	M_n (kip-in)	P_{max} (kip)
FRP1/FRP2	1.78	8	10	0.022	0.007	692	46
Beam Name	A_s (in^2)	B (in)	d (in)	ρ_s	ρ_b	M_n (kip-in)	P_{max} (kip)
Steel 1/Steel 2	1.32	8	10	0.017	0.034	699	46
Steel 3/Steel 4	0.93	8	10	0.012	0.034	512	34

1 kip = 4.45 kN, 1 kip-in = 0.113 kN-m

The nominal shear capacity of the FRP beams was determined from,

$$V_n = V_{c,f} + V_f \tag{1}$$

where, V_n = nominal shear capacity, $V_{c,f}$ = shear capacity of the concrete when FRP bars are used, and V_f = shear capacity provided by the FRP stirrups.. The shear capacity provided by concrete, $V_{c,f}$, was obtained as 2.3 kips from,

$$V_{c,f} = \frac{E_f}{E_s} V_c \tag{2}$$

The strength of the bent portion of the stirrups, f_{fb}, was calculated using the following equation from ACI440.1R-01,

$$f_{fb} = \left(0.05\frac{r_b}{d_b} + 0.3\right) f_{fu} \leq f_{fu} \tag{3}$$

where, r_b is the radius of the bend (in) and d_b is diameter of reinforcing bar (in). The inside radius of the bent portion of the stirrup was assumed to be $r_b = 3d_b$ where d_b is the diameter of the stirrups. In reality, $r_b = 5.7d_b$, the design was conservative and f_{fb} was calculated as 49.5 ksi. The manufacturer's data stated that the strength of the bent portion of the stirrups was generally 38% of the ultimate tensile strength of the unbent bars, that is $f_{fb} = 41.8$ ksi which was lower than the calculated value. When the strain in the stirrups was limited to 0.002, the tensile strength of the FRP stirrups for shear design, f_{fv}, was calculated as 11.8 ksi, which was approximately 10% of the actual tensile strength of the #3 bars. This value was felt to be extremely conservative and the tensile strength of the FRP stirrups for shear design, f_{fv}, was taken as 41.8 ksi, according to the manufacturer's recommendation. The shear resistance provided by the stirrups was calculated as $V_f = 27.4$ kips and the shear capacity of the cross section was calculated as $V_n = 29.7$ kips. This was greater than the nominal design requirement of V = 25 kips corresponding to the desired load-carrying capacity of P = 50 kips.

Fabrication, Casting and Test Method

The beams were cast using ready mix concrete. The measured 28 day compressive strength of the concrete, f_c', was 5880 psi. The beams were tested in a 1000 kip servo-hydraulic testing machine in displacement control at a rate of 0.03 in/min. They were tested under four point bending on simply supported spans of 90 inches. The moment span was 30 in. in all tests. Fig.1 shows beam SD3-90 during testing and Fig. 2 shows the same beam at failure. Load and the crosshead deflection data were recorded continuously using a HP 3852 data acquisition unit and Labview© software.

TEST RESULTS

The maximum total load, P_{max}, carried by each beam and the deflection, δ_{max}, corresponding to the maximum load are presented in Table 3. Load versus crosshead deflection graphs are shown in Figs. 3 and 4. In Fig. 3, load deflection graphs of all FRP rebar and steel reinforced beams are provided.

In Fig. 4, the load deflection graph of SD3-90 is compared with one FRP and one steel beam.

Fig. 1. Test setup and typical crack pattern for SD3-90

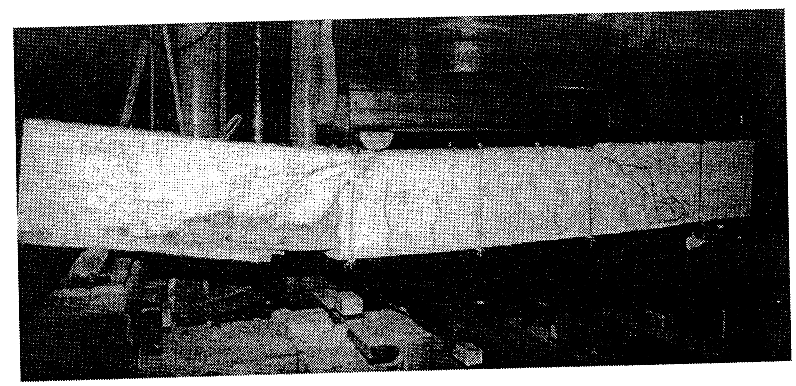


Fig. 2. Failure mode, SD3-90

Failure Modes

All the beams except FRP 1 and SD3-90 failed in flexure. SD3-90 failed in shear/compression followed by splitting failure of the longitudinal reinforcement. FRP 1 also failed in shear/compression, whereas FRP 2 failed in flexure/compression. All steel reinforced beams failed in flexure/tension. The ultimate failure of the steel reinforced beams occurred after the tensile reinforcement yielded.

Table 3. Experimental test results

Beam Name	P_{max} (kips)	δ_{max} (@P_{max}) (in)	Beam Failure Mode
FRP 1	47.5	1.59	shear/compression
FRP 2	57.1	1.53	flexure/compression
Steel 1	71.7	1.74	flexure/tension
Steel 2	66.3	1.71	flexure/tension
Steel 3	49.1	3.02	flexure/tension
Steel 4	49.7	3.02	flexure/tension
SD3-90	49.6	1.57	shear/compression

1 kip = 4.45 kN; 1 in = 25.4 mm

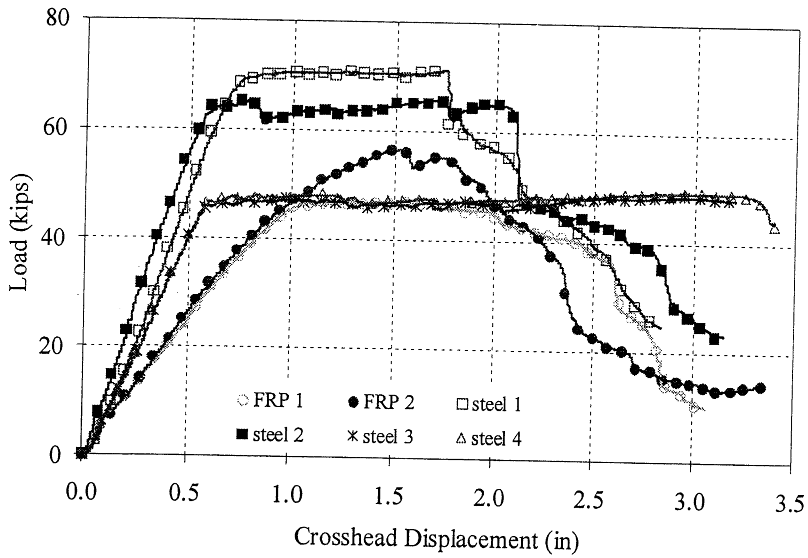


Fig. 3. Load deflection plots of steel and FRP rebar reinforced beams

Fig. 5 shows FRP 1 at failure. The failure of this beam was much like the failure mode observed in most of the FRP grid reinforced beams[5]. Uniformly distributed shear cracking was observed, followed by flexural shear cracking in the shear spans. The load deflection behavior was almost linear until the major shear crack that caused the failure of the beam reached the load point and the concrete in the vicinity of the load point started crushing. The beam held the maximum load while deflecting an additional 0.7 in. The catastrophic drop in the load was due to the failure of the stirrup

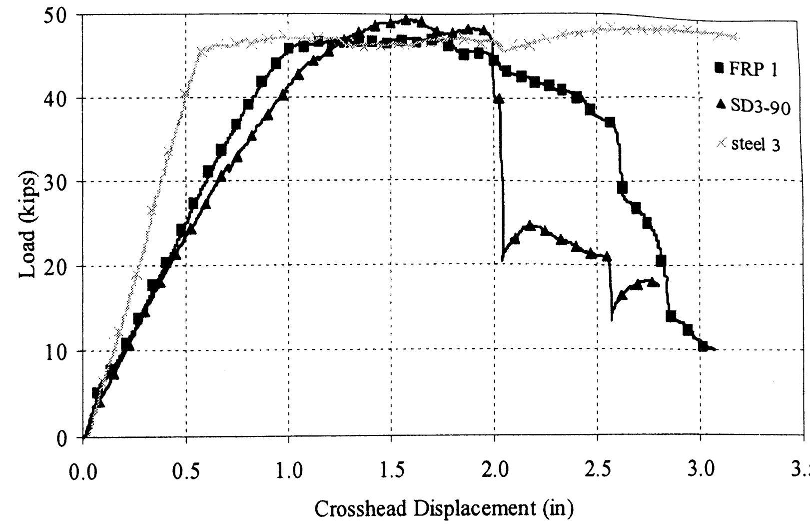


Fig. 4. Load deflection plots of FRP 1, SD3-90 and Steel 3 compared

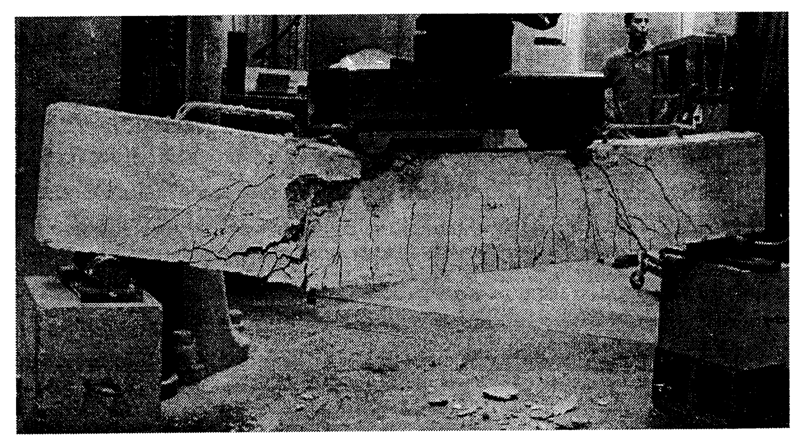


Fig. 5 Failure of FRP 1 in shear/compression

at the lower bend. This failure mode was undesirable and unexpected since the beam was designed to ultimately fail in flexure/compression.

Fig. 6 shows FRP 2 at failure. This beam failed in flexural compression. Major flexural and shear cracking was observed prior to the failure of the beam. When the maximum load was attained, the concrete under both of the load points started crushing. These local cracks lead to horizontal cracks that propagated towards the midspan that joined together in the middle resulting in compressive crushing in the entire moment span.

Fig. 6 Failure of FRP 2 in flexure/compression

The ultimate strength of FRP 1 was 47.5 kips, whereas the ultimate strength of FRP 2 was 57.1 kips. The 10 kip difference between the beam capacities was due to the premature failure of FRP 1 in shear due to the failure of the stirrup at the bend. All the steel reinforced beams failed in the same manner. The steel reinforced beams displayed elastic-plastic load displacement behavior. The tensile reinforcement started yielding at about 65 kips for Steel 1 and Steel 2 and at about 45 kips for Steel 3 and Steel 4. The beams ultimately failed due to secondary concrete compression failure after yielding.

DISCUSSION AND CONCLUSIONS

FRP 1 did not fail in flexure as expected. FRP 2, which was identical to FRP 1, failed in this mode. As mentioned earlier, the strength of the stirrups at the bends was determined according to the manufacturers data. The strain in the stirrups was not limited to 0.002 as recommended by ACI 440.1R-01. However, the strength of the stirrups at the bends is unpredictable, as they are greatly influenced by the manufacturing process. As observed during the testing of FRP 1 and FRP 2, the stirrups did fail in FRP 1 causing the beam to fail in flexure, whereas they were intact throughout the testing of FRP 2 and this beam failed in flexure/compression.

These results show that it is reasonable to limit the strain in the stirrups in the design to 0.002 even though it appears to be a conservative value.

In Table 4 the FRP reinforced and steel reinforced beams are compared in terms of area and axial stiffness (AE) of the reinforcement, and load capacity at a deflection of $l/240$ (service conditions.)

Table 4. Experimental Test Results

Beam Name	A_s, A_f (in^2)	*AE* (*kip*)	P @ $\delta_{l/240}$ (*kip*)	M_{exp} (*kip-in*)
FRP 1	1.780	10538	19.7	726
FRP 2	1.780	10538	20.5	857
Steel 1	1.320	38280	36.5	1076
Steel 2	1.320	38320	44.8	995
Steel 3	0.930	26970	31.1	737
Steel 4	0.930	26970	31.6	746
SD3-90	2.216	10572	18.2	744

1 kip = 4.45 kN, 1 in^2 = 645 mm^2, 1 kip-in = 0.113 kN-m

The FRP grid reinforced beam and FRP rebar reinforced beams are comparable in terms of axial stiffness, and the load the beams can carry under service conditions. FRP 1 and FRP 2 had an axial stiffness of 10538 kips, whereas, SD3-90 had an axial stiffness of 10572 kips. These beams had an average ultimate load carrying capacity of approximately 50 kips. Steel 3 and Steel 4 also had the same ultimate capacities as these beams. The average load carried by the FRP reinforced beams (SD3-90, FRP 1 and FRP 2) under permissible deflections was approximately 60% of the steel reinforced beams (Steel 3 and Steel 4). On the other hand the average axial stiffness of the FRP reinforced beams was only 40% of steel reinforced beams.

ACKNOWLEDGMENTS

Support for this work was provided by the US National Science Foundation under grant no. CMS 9896074. The donation of FRP materials by Strongwell (manufacturers of the FRP grid), and Hughes Brothers (manufacturers of the FRP rebar) is acknowledged. William Lang and John

Dreger of the Wisconsin Structures and Materials Testing Laboratory are thanked for their assistance and support.

REFERENCES

1. ACI Committee 318, *Building Code Requirements for Structural Concrete (318-99) and Commentary (318R-99)*, American Concrete Institute, Farmington Hills, Michigan, 1999.
2. ACI Committee 440, *Guide for the Design and Specification of Concrete Structures Reinforced with FRP Bars*, American Concrete Institute, ACI 440.1R-01, Farmington Hills, Michigan, 2001.
3. Bank, L. C. and Ozel, M. "Shear Failure of Concrete Beams Reinforced with 3-D FRP Grids," *Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures, Fourth International Symposium, SP-188,* C. W. Dolan, S. H. Rizkalla and A. Nanni, eds., American Concrete Institute, 1999, pp. 145-156.
4. Ozel, M., "Behavior of Concrete Beams Reinforced with 3-D Fiber Reinforced Plastic Grids," *PhD Dissertation*, University of Wisconsin, Madison, 2002, 180 pp.
5. Ozel, M. and Bank, L.C., "Behavior of Concrete Beams Reinforced with 3-D Composite Grids", *American Society for Composites, 16th Annual Technical Conference*, Virginia, September 9-12, 2001, CD-ROM.

FRPRCS-6, Singapore, 8–10 July 2003
Edited by Kiang Hwee Tan

CONCRETE BEAMS STRENGTHENED WITH PRE-STRESSED NEAR SURFACE MOUNTED REINFORCEMENT

H. NORDIN

Dept of Civil Engineering, Luleå University of Technology, 971 87 Luleå, SWEDEN

B. TÄLJSTEN

Dept of Civil Engineering, Luleå University of Technology, 971 87 Luleå, SWEDEN
Skanska Teknik AB, 169 83 Solna, SWEDEN

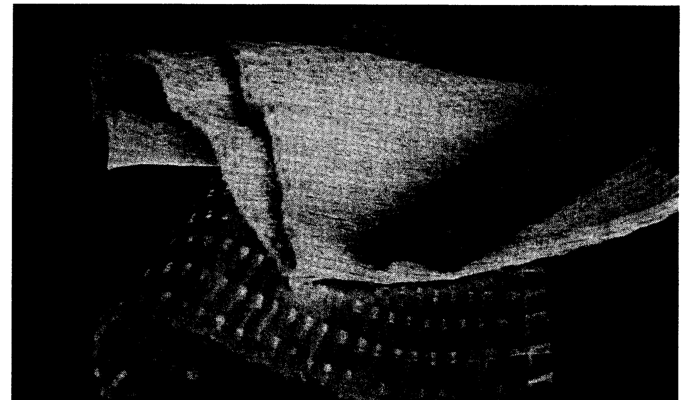
The need for concrete repair and rehabilitation is well known and many countries in the world are researching in this field. New technical solutions and methods that can effectively bring forth more economical ways of upgrading structures are most welcome. In recent years the use of CFRP plate bonding has shown to be such a method. This technology implies that a thin carbon fibre laminate or fabric is bonded to the surface of the structure and then acts as an outer reinforcement layer. Most of the applications world-wide have been with no pre-stressing of the laminates or sheets, even though there have been investigations presented where CFRP laminates have been pre-stressed before they have been bonded to the concrete surface. However, the risk of damage by for example vehicle impact can be disastrous to such a structure. At Luleå University of Technology, research is taking place in the field of CFRP – plate bonding. Recently, extensive research has been undertaken to investigate the possibilities of using CFRP laminates as Near Surface Mounted Reinforcement (NSMR). This method may be defined as a method where FRP rods are bonded in slots in a concrete cover. By pre-stressing the reinforcement a compressive force is transferred into the concrete structure at the same time as they are protected in the slot. This paper presents laboratory tests for concrete beams strengthened with pre-stressed near surface mounted CFRP bars.

INTRODUCTION

It is no doubt that there is a great potential for, and considerable economic advantages in, FRP strengthening. However, if the technique is to be used effectively, a sound understanding of both the short-term and long-term behaviour of the bonding system is required. It also requires reliable information concerning the adhesion to concrete and composite. The execution of the bonding work is also of great importance in order to

Edited by Kiang Hwee TAN
National University of Singapore, Singapore

Singapore 8–10 July, 2003



Fibre-Reinforced Polymer Reinforcement for Concrete Structures

VOLUME 2

Proceedings of the Sixth International Symposium on FRP Reinforcement for Concrete Structures (FRPRCS-6)

FRPRCS
2003 Singapore

World Scientific