BEHAVIOUR OF CONCRETE SLABS REINFORCED
BY GFRP

by

Dr. Hany Abdalla, P.Eng.
Lecturer, Cairo University, Cairo, Egypt

Dr. Mamdouh El-Badry, P.Eng.
Associate Professor, Concordia University, Montreal, Canada

and

Dr. Sami Rizkalla, P.Eng.
President of the Canadian Network of Centres of Excellence
on Intelligent Sensing for Innovative Structures
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1. INTRODUCTION

Parking structures and bridge decks are prime examples of structures subjected to severe environmental conditions, leading to the deterioration of the main structural concrete components due to corrosion of steel, and consequently, a reduced serviceability. Significant fluctuation of the temperature and use of salt for deicing also accelerates the corrosion process of the steel reinforcements. In Canada, it is estimated that the cost of repair of parking structures is in the range of four to six billion dollars\(^{(1)}\). The estimated repair cost for existing highway bridges in U.S. is over 50 billion dollars, and one to three trillion dollars for all concrete structures\(^{(2)}\). Excessive corrosion problems also exist in Arabian Gulf countries\(^{(3)}\). The exterior of reinforced concrete structures in these countries are subjected to an extremely aggressive environment due to high temperatures and humidities. Presence of shrinkage and flexural cracks allows intrusion of the salt-laden condensation, contaminated rain water, oxygen and carbon dioxide gases. This mixture of chemicals and moisture eventually penetrates the level of steel reinforcement and accelerates their corrosion rates. These include cathodic protection systems\(^{(4)}\) and use of galvanized or epoxy coated rebars\(^{(5)}\). Long term efficiency of these systems is still uncertain\(^{(6)}\).

Engineers are currently exploring the use of fibre reinforced plastics (FRP) as a promising solution for the corrosion problem. FRP is non-corrosive, magnetically neutral and has high strength-to-weight ratio. Although there is tremendous diversity of the available Glass Fibre Reinforced Plastic (GFRP) bars, no general information, nor clear specifications specifically address their behaviour as reinforcements for concrete structures.

In this paper the ultimate behaviour of concrete slabs reinforced by GFRP reinforcements is presented and compared to that of slabs reinforced with conventional steel. A rational analysis is introduced based on strain compatibility approach to describe the flexural behaviour using the actual constitutive characteristics of the concrete and FRP reinforcements. In addition, in the light of the experimental results, an attempt is made to assess the contribution of the shear resistance components to the ultimate shear resistance of the FRP reinforced concrete sections. The applicability of the various methods available for prediction of the shear capacity of
reinforced concrete members is examined. Based on the experimental results, modifications are introduced to these methods to account for the nature of crack pattern and propagation in FRP reinforced concrete members.

This paper presents a phase of an extensive program to study the flexural behaviour of concrete slabs reinforced by GFRP reinforcements. These are also used as reinforcements for beams and rock anchorages.

2. RESEARCH PROGRAM

Details of the eight prototype one-way concrete slabs, reinforced by three different reinforcement materials, tested in this program are given in Table 1. Five specimens were reinforced by GFRP bars, two specimens were reinforced by conventional steel rebars and one specimen was reinforced by CFRP rods. The three slabs reinforced by CFRP and steel reinforcements are used as control specimens to compare the behaviour of the slabs reinforced by GFRP bars.

The length and width of all the slabs were 3500 mm (11.5 ft) and 1000 mm (3.28 ft), respectively, with a clear span of 3000 mm (10 ft), which were kept constant throughout the study. The thickness of slabs were 150 mm (6 in.) according to the requirements of the Canadian Design Code, CAN3-A23.3-M94, and the program was expanded to include 200 mm (8 in.) thick slabs. A concrete cover of 38 mm (1.5 in.) was used for the longitudinal reinforcements. Initially, the slabs were designed to achieve the classical three modes of failure, including: rupture of the reinforcements; simultaneous rupture of the reinforcements and crushing of the concrete and; crushing of the concrete while the reinforcement remains elastic. This was accomplished by using reinforcement ratios less, equal and more than the balanced reinforcement ratio, \( P_b \), for the slabs, respectively. After completion of testing the 150 mm (6 in.) thick slabs, the program was revised for reasons related to the observed modes of failure, as will be discussed later.

Schematic and view of the test set-up used are shown in Figures 1(a) and 1(b), respectively. A 1000 kN (225 kips) closed-loop MTS actuator was used to apply the load through a spreader beam system as shown in Figure 1(b). The slabs were simply supported at each end along the entire width. The slabs were instrumented to measure the applied load, midspan deflections, strains in the extreme compression fibres of the concrete, strains in the reinforcements, strains in the concrete at the level of reinforcements, crack widths within the constant moment zone and bond slippage at both ends of the specimens. The slabs were initially loaded at a slow rate of 0.6 mm/min. (3/128 in./min.) up to initiation of cracks. After cracking, the rate of applied load was increased to 1.2 mm/min. (364 in./min.) up to failure.

Although GFRP bars possess the lowest tensile strength in comparison to other available
FRP reinforcements, they have the advantage of being the least expensive\(^1\), along with their non-corrosive, magnetically neutral and high strength to weight ratio\(^7\) characteristics. Therefore, GFRP reinforcements are an excellent candidate for reinforced concrete structures subjected to aggressive environmental conditions and for those sensitive to magnetic fields. Figure 2 shows the material characteristics of the three types of reinforcement used in this program, including GFRP, Carbon Fibre Reinforced Plastic (CFRP) and steel. Table 2 provides their fundamental mechanical properties.

The GFRP bars are manufactured by pultration of E-glass continuous fibres and thermosetting polyester resin. To enhance the bond characteristics, the surface is wrapped by helically glass fibre strands and covered by a mixture of a known grain size of sand and polyester resin\(^7\). Figure 3 shows the particular GFRP bars used in the investigation. The CFRP reinforcement used in this test program is an 8 mm (5/16 in.) diameter CFRP rod produced by Mitsubishi Kasei, Japan. The CFRP rods are fabricated using continuous coal tar pitch-based continuous fibre and epoxy resin\(^9\). The measured average compressive strengths of the concrete used for the slabs ranged from 60.0 MPa (8600 psi) to 66.3 MPa (9600 psi) at the time of testing, with a maximum aggregate size of 14 mm (1/2 in.).

3.0 FLEXURAL BEHAVIOUR

During the experimental program, the concrete slabs reinforced by GFRP or CFRP carbon fibre reinforced plastic, Leadline, behaved linearly up to cracking. After cracking, they behaved linearly also but with reduced stiffness, as shown in Figures 4(a) and 4(b). This is attributed to the linear characteristics of the FRP rebars. In Figure 4(a), the concrete compression strain for concrete slab reinforced with Leadline is compared to that for concrete slab reinforced with conventional steel. The reinforcement ratios for both slabs were very close and less than the balanced reinforcement ratios. It can be seen from Figure 4(a) that the ultimate load for the concrete slab reinforced by Leadline is higher than that reinforced by steel. This is due to the high ultimate tensile strength of Leadline, 1970 MPa (285 ksi), compared to the yield stress of steel, 435 MPa (63 ksi).

Prior to the steel yielding, the concrete compression strain was larger in slab LL-200-C reinforced with Leadline than in slab S-200-T reinforced with steel, for the same load. This is due to the lower elastic modulus of the Leadline compared to steel. After the steel has yielded, the concrete strain was higher in the slab reinforced with steel than in the slab reinforced with Leadline. It can be seen also from Figure 4(a) that for under-reinforced sections, the ratio between the ultimate load to the cracking load for slab reinforced with FRP is much higher than that for slab reinforced with steel. This is due to the fact that the Leadline does not yield and the reinforcement stress is always increasing with the increase of strain up to failure. For FRP over-reinforced section, Figure 4(b), where the failure was expected to occur by crushing of concrete, the ultimate load for slab reinforced with steel was higher than that for slab reinforced by GFRP
for the same reinforcement ratio. This was due to the premature shear failure that took place in this slab.

Slabs I-150-A and I-150-B failed due to rupture of the reinforcements. Slab I-150-C was over-reinforced and expected to fail by crushing of the concrete before rupture of the reinforcements. However, the failure was due to shear of the GFRP bars at a major crack located outside the constant moment zone, as shown in Figure 5. Behaviour of slab S-150-T, reinforced by conventional steel, was typical by a reduction in the stiffness after the initiation of the first crack, yielding, reflected by the measured large deflections, and failure due to the crushing of the concrete in the compression zone at a location within the constant moment zone of the slab.

Based on the behaviour described above, the experimental program was revised to test only two 200 mm (8 in.) thick slabs reinforced by GFRP bars, slabs I-200-A and I-200-C. These slabs were designed to fail by the two extreme modes of failure, namely; rupture of the GFRP reinforcements and crushing of the concrete, respectively. Along with a control slab reinforced by steel reinforcements, a fourth 200 mm (8 in.) slab reinforced by CFRP, was included in the program. Slab I-200-A failed by rupture of the GFRP reinforcements due to flexure in the maximum moment zone. Slab I-200-C failed by shear of the GFRP bars at a major diagonal crack located outside the constant moment zone, as shown in Figure 6. The steel reinforced slab, S-200-T, failed, as predicted, by crushing of the concrete in the compression zone. The slab reinforced by CFRP reinforcements, LL-200-C, displayed slipping of the CFRP reinforcements at one end of the slab at a load level 27.8% higher than the nominal strength of the slab. The flexural bond strength of the Leadline reinforcement at failure is estimated to be 4.6 MPa (667 psi). This value is similar to the values obtained from experimental tests conducted at University of Manitoba(10), and less than the flexural bond strength of steel reinforcements. The ultimate load was significantly higher than the other two slabs, due to the relatively high tensile strength of the CFRP rods in comparison to the GFRP and steel reinforcements, as given in Table 2. The measured tensile stress of the CFRP rods at failure was approximately 3000 MPa (435 ksi), which is well above the guaranteed ultimate tensile strength value of 1970 MPa (285 ksi), given in Table 2, and used in the original design of the slab. These values are in agreement with measured values obtained in an independent study conducted at the University of Manitoba(11). It should be noted that prior to failure, all the FRP reinforced concrete slabs gave an ample warning effect through large deflections and extensive cracking.

3.1 Strain distribution

The strain distribution at cracking and at ultimate load stages for slab I-150-C reinforced with Isorod is shown in Figure 7. The slab section is over-reinforced with $\rho$, reinforcement ratio, of 0.955%. In the same figure, the strain distribution for slab S-150-T having almost the same reinforcement ratio, $\rho=0.962\%$, is shown for comparison purposes. It can be seen from Figure
7 that the strains and hence the curvatures are generally higher in slab I-150-C reinforced with Isorod than in slab S-150-T reinforced with steel.

Immediately after cracking when the tensile stress is mostly carried by the reinforcement, concrete strain at the reinforcement level for the slab reinforced with Isorod was about 7 times that of the slab reinforced with steel. This is due to the relatively low elastic modulus of the Isorod. This resulted in smaller depth for the neutral axis in slab I-150-C than in slab S-150-T. Increasing the load above the cracking load resulted in slight upward shifting of the neutral axis in the slab reinforced with Isorod. The increase in the applied load was resisted internally by increase in the reinforcement and concrete strains not by increase in the moment lever arm. At ultimate load, the neutral axis depths of the two slabs, reinforced with Isorod and steel, were almost the same after allowing the steel in slab S-150-T to yield. This resulted in almost the same curvatures for the two slabs at the collapse load. It has to be mentioned that for slabs reinforced with conventional steel, the failure is considered to occur when the reinforcing steel yields since there is no much increase in the section capacity beyond the yield load stage and due to the excessive curvature and deflection taking place after yielding. Therefore, according to Figure 7, the curvature at ultimate of slab I-150-C reinforced with Isorod is more than 3 times the curvature at the yield stage of slab S-150-T reinforced with steel.

The strain distribution at cracking and at ultimate load stages for under-reinforced slab LL-200-C is compared to that of slab S-200-T reinforced with steel in Figure 8. It can be seen from Figure 8 that the curvature at ultimate load for slab LL-200-C was more than 6 times than the curvature at yield load for slab S-200-T. As the load increases after cracking the neutral axis moves upward causing reduction in the compression zone for slabs reinforced with steel while it remains almost stationary for slabs reinforced with Leadline. At yielding of the steel bars, the increase of the internal moment resistance due to the increase of the applied load could be achieved only by increasing the internal lever arm length and consequently the decrease of the compression zone depth. For the slab reinforced with Leadline, the increase of the internal moment resistance is achieved by an increase of the tensile resultant force due to increase in Leadline stress and therefore the neutral axis will remain almost stationary up to failure.

In Figure 9, comparison between strain distribution of FRP and steel reinforced concrete slabs is shown at the same applied load. Comparing the results shows that at the same load, the strains and hence the curvature are higher in the FRP reinforced concrete slabs than in steel reinforced concrete slabs. Also the depth of the compression zone is always smaller in FRP reinforced concrete slabs than in steel reinforced concrete slabs at the same load for the same reinforcement ratio.
3.2 Flexural capacity of FRP reinforced concrete slabs

According to the Canadian Code\(^{(8)}\) and the ACI Code\(^{(12)}\), the actual parabolic stress distribution of concrete in compression is approximated by an equivalent rectangular stress block shown in Figure 10. The flexural capacity of a singly-reinforced section will be governed by: (i) rupture of the FRP tension reinforcement in which case, the section is referred to as an under-reinforced section; (ii) crushing of the concrete where the section is referred to as an over-reinforced section and (iii) simultaneous failure of both the concrete and the tension reinforcement, in which case the section is referred to as a balanced section. This case is also referred to as a balanced strain condition.

Applying the equilibrium and compatibility conditions, the reinforcement ratio \( \rho_b \) in the balanced strain condition can be obtained using the equivalent rectangular stress block, Figure 10, as:

\[
\rho_b = \frac{0.85 \beta f'c}{\frac{\varepsilon_{uc}}{\varepsilon_{uc} + \varepsilon_{fu}}} \tag{1}
\]

where \( \beta = \) ratio of depth of equivalent rectangular stress block to depth of the neutral axis, which can be taken, in SI units, as:

\[
\beta = 0.85 - 0.05 \left( \frac{f'c - 27.6}{6.7} \right) \geq 0.65 \tag{2}
\]

Substituting for \( \varepsilon_{uc} = 0.003 \) in Equation (1) and multiplying through by \( E_F \). Equation (1) can be rewritten in the form:

\[
\rho_b = \frac{0.85 \beta f'c}{\sigma_{fu}} \left( \frac{0.003 E_F}{\sigma_{fu}} \right) \tag{3}
\]

The balanced reinforcement ratios for the Isorod and Leadline FRP reinforcements and for conventional steel reinforcement is shown in Figure 11 for different concrete strengths. These ratios are based on the properties of the FRP reinforcement given in Table 2.

It can be seen from Figure 11 that the balanced reinforcement ratios for FRP reinforced concrete sections are much lower than those for steel reinforced concrete sections. This is due to the higher tensile strength and the lower modulus of elasticity of the FRP reinforcements relative to the conventional steel. For practical ratios of FRP reinforcements and in order to
control deflection and cracking, most of the FRP reinforced concrete sections will be over-reinforced. It has to be noted that whether the FRP reinforced concrete section is under-reinforced or over-reinforced, the flexural failure will be a brittle failure. This is due to the fact that the FRP reinforcements do not yield which results in a brittle failure even for under-reinforced sections.

Satisfying the equilibrium conditions, the ultimate moment capacity of a FRP under-reinforced concrete section can be obtained by Equation (4):

\[ M_u = A_F f_{Fu} \left( d - \frac{a'}{2} \right) \]  

where

\[ a' = \frac{A_F \sigma_{Fu}}{0.85 \ b f'c} \]  

For a FRP over-reinforced concrete section, the stress in the tension reinforcement will be less than \( \sigma_{Fu} \) when the concrete failure occurs. For the case of concrete compression failure, the depth of the neutral axis, \( C \), measured from the concrete extreme compression fibre, must be obtained based upon the condition that the concrete strain is reaching \( \varepsilon_{cu} \) while the strain in the FRP reinforcement is less than \( \varepsilon_{Fu} \). From the equilibrium of forces shown in Figure 10:

\[ 0.85 \ f'c \ b \ \beta \ C = A_F \ \sigma_F \]  

From the compatibility of strains when the concrete compression strain reaches \( \varepsilon_{cu} \):

\[ \varepsilon_F = \varepsilon_{cu} \ \frac{d - C}{C} \]  

Substituting Equation (7) in Equation (6) leads to an expression for the depth of the neutral axis, \( C \), as follows:

\[ C = d \cdot \frac{(\sqrt{1 + 4 \ \beta \ K} - 1)}{2 \ \beta \ K} \]  

where
\[ K = \frac{0.85 \, f'_c}{\rho \, E' \, e_{cu}} \]  

Hence the ultimate moment capacity of a FRP over-reinforced concrete section can be estimated by:

\[ M_u = 0.85 \, f'_c \, a' \, b \, (d - \frac{a'}{2}) \]  

where

\[ a' = d \cdot \frac{\left(\sqrt{1 + 4 \, \beta \, K} - 1\right)}{2 \, K} \]  

Equations 4, 5, 10 and 11 were used to estimate the ultimate flexural capacity of the FRP reinforced concrete slabs described in this research. The analytical results based on the material characteristics obtained from tests are compared to those from the experiments in Table 3. It can be seen from Table 3, that for the under-reinforced slabs I-200-A, I-150-A and I-150-B, the ratio of the experimental ultimate loads to the predicted values are 72%, 94% and 80%, respectively.

The measured concrete strains at the level of the GFRP reinforcements and the strain gauges attached to the GFRP bars were lower than the expected ultimate strain values for all under-reinforced slabs. This resulted in lower ultimate moment load resistance of these slabs. Typical load-strain at the level of reinforcements, measured by six DEMEC point stations and two electrical strain gauges attached to each GFRP bar within the constant moment zone, is shown in Figure 12. The results indicate that rupture of the GFRP reinforcements occurred, at strain levels lower than the ultimate strain levels achieved in tension tests. This behaviour suggests that the sudden impact caused by the transfer of tensile forces from the concrete to the GFRP bars at the time of cracking, created a localized rupture of the fibres, impeding the GFRP reinforcements to achieve their ultimate tensile strengths. Using the measured ultimate loads and strain compatibility, the percentage of the ultimate tensile stresses of the GFRP bars to the nominal values, \( \sigma_{Exp}/\sigma_{Fu} \), for slabs I-200-A, I-150-A and I-150-B, were 76.8%, 95.4% and 81.0%, respectively. The load deflection behaviour for the three aforementioned slabs are presented in Figure 13 in ascending order with respect to their bar diameters and percentage of reinforcements. All three slabs exhibit fairly linear behaviour after the initiation of cracking, as evident by the load deflection envelope. As the bar diameter increased from 9.5 mm (3/8 in.) to 12.7 mm (½ in.), and consequently the reinforcement ratio, the immediate drop of the load
resistance, $\Delta P$, due to the initiation of cracks significantly decreased, as shown in Figures 13(a) and 13(b). This resulted in a reduction in the impact effect of the load transfixed from the concrete in tension to the GFRP and consequently reduces the possibility of localized fracture of the fibres. This was evident by the increase of the ratio of the measured stress of GFRP at failure to the tensile strength, $\sigma_{F_{\text{eff}}}/\sigma_{F_{u}}$, from 76.8% to 95.4%. It was also noticed that increasing the bar diameter from 12.7 mm ($\frac{1}{2}$ in.) to 15.9 mm (5/8 in.), reduced the ratio $\sigma_{F_{\text{eff}}}/\sigma_{F_{u}}$ from 95.4% to 81.0%, as given in Figures 13(b) and 13(c). This behaviour could be attributed to the fact that GFRP bars are made of thousands of layers of glass fibre and the interlaminate shear lag could lead to a possible unstressed central region of the GFRP bar. These characteristics lead to a reduction of the ultimate tensile strength of larger diameter GFRP bars in comparison to small diameters using the same type and percentage by volume of fibres. This phenomenon is reported by others\textsuperscript{(13)} for GFRP reinforcements.

These findings lead to the development of the following proposed equation to evaluate the general strength of FRP reinforcements, $T$, in terms of the cross-sectional area, $A_{F}$, and ultimate tensile strength, $\sigma_{F_{u}}$, of specific sizes of GFRP bars.

$$T = \beta_{1} \beta_{2} A_{F} \sigma_{F_{u}} \quad [\text{kN}] \quad (12)$$

The factor $\beta_{1}$ accounts for the reductions in the strengths due to the increase of the diameter in comparison to the specific size of bar manufactured using the same type of fibres and percentage of fibres by volume. The factor $\beta_{2}$ is related to the percentage of FRP reinforcements ratio used in a given concrete member. Evaluation of $\beta_{1}$ could be determined using pure tension tests of various diameters of FRP bars manufactured using the same type of fibres and percentage of fibres by volume. Evaluation of $\beta_{2}$ requires performing flexural tests of concrete members using the same diameter of FRP bars and various percentages of reinforcement.

4. Shear analysis of FRP reinforced concrete slabs

During the experimental program, two of the test slabs failed on shear of the GFRP bars at a major crack within the flexural-shear location. The unexpected shear failure in the GFRP slabs I-150-C and I-200-C occurred well below the ultimate shear capacity according to the Canadian Code\textsuperscript{(8)} and the ACI Code\textsuperscript{(12)}. Close up of the shear rupture of the GFRP bars at the cracks is shown in Figure 14. The measured crack widths, in both slabs prior to failure was approximately 15 mm (9/16 in.), and extended vertically along the entire depth of the slab, providing a compression zone depth of approximately 15 mm (9/16 in.). The shear failure of concrete slabs reinforced with fibre reinforced plastics was also reported by others\textsuperscript{(14),(15)}.

The shear behaviour of reinforced concrete members is generally more complicated than
the flexural behaviour. Failure in shear takes place under combined stresses resulting from applied shear force and bending moment. The design procedures for shear depend, to a large extent, on empirical formulas based on extensive tests and simplifying assumptions rather than on a theoretical analysis. This is due to the large number of parameters that affect the shear capacity such as combined load effects, presence of reinforcement, presence of cracks, the non-linearity and non-homogeneity of the material.

According to the Canadian Design Code\(^8\), the shear strength of members without shear reinforcement is given in SI units by:

**CSA CAN3-A23.3-M94 [SI]:**

\[
V_c = 0.2 \lambda \phi_c \sqrt{f'_c} b_w d \quad [N]
\]  

(13)

The ultimate shear capacities based on Equation (13) were estimated for the test-slabs, using unity for the factors \(\lambda\) and \(\phi\), and compared to the experimental results in Table 3. It can be seen from Table 4 that for slabs I-150-C and I-200-C, failed in shear, the ultimate shear capacity based on Equation (13) is 4 times and 3 times, respectively, of the experimental ultimate shear capacity. In addition, the shear capacity according to Equation (13) does not depend on the type or ratio of the longitudinal reinforcement.

The ACI Code\(^{12}\) provides a more detailed formula to estimate the shear capacity of members without shear reinforcement. According to this method, which takes into consideration, the ratio of the longitudinal reinforcement, the shear strength can be estimated in SI units by:

**ACI Clause 11.3.2.1 [SI]:**

\[
V_c = \frac{1}{7} \left( \sqrt{f'_c} + 120 \rho \left( \frac{V_u d}{M_u} \right) \right) b_w d \leq 0.3 \sqrt{f'_c} b_w d
\]

(14)

where: \[\frac{V_u d}{M_u} = d / a\]

Results from Equation (14) are compared to those from Equation (13) in Table 3. Shear capacities predicted by Equation (14), although smaller than those from Equation (13), still well above the experimental ultimate shear loads for slabs I-150-C and I-200-C.

The measured ultimate shear capacities of slabs I-150-C and I-200-C were also compared
to the Japanese Society of Civil Engineers (JSCE) Design Code 1984\(^{(16)}\), Equation (15), the recommendation by Machida\(^{(17)}\) for the JSCE Design Code 1996, Equation (16), and the Comité Euro-International du Béton and Fédération Internationale de la Précontrainte (CEB-FIP) Design Code\(^{(18)}\), Equation (17). The results are presented in Figures 15 and 16 for the 150 mm (6 in.) and 200 mm (8 in.) slabs, respectively.

**JSCE 1984 [SI]:**

\[
V_c = 0.94 \sqrt[3]{f''_c} (1 - \beta_d + \beta_p + \beta_n) b_w d \quad [kg] \tag{15}
\]

where:

\[
\begin{align*}
\beta_d &= \sqrt{100/d} - 1 \geq 0.0 \quad (d \text{ in [cm]}) \\
\beta_p &= \sqrt{100/p} - 1 \leq 0.73 \\
\beta_n &= \frac{M_o}{M_d} \leq 1.0 \\
f''_c \text{ in [kg/cm}^2\]
\]

where \(M_o\) = decompression moment and \(M_d\) = applied moment at the section.

**Machida (JSCE 1996) [SI]:**

\[
V_c = 0.9 \beta_d \beta_p \beta_n \sqrt[3]{f''_c} b_w d \quad [kg] \tag{16}
\]

where:

\[
\begin{align*}
\beta_d &= \sqrt{100/d} \leq 1.5 \quad (d \text{ in [cm]}) \\
\beta_p &= \sqrt{100/p} \leq 1.5 \\
\beta_n &= 1 + \frac{M_o}{M_d} \leq 2.0 \\
f''_c \text{ in [kg/cm}^2\]
\]

**CEB-FIP [SI]:**

\[
V_c = \tau_{Rd} k (1 + 50 \rho) b_w d \quad [N] \tag{17}
\]

where:

\[
\begin{align*}
\tau_{Rd} &= 0.00842 (f''_c - 50) + 5 \quad (\text{for } f''_c \geq 50 \text{ MPa}) \\
k &= 1.6 - d \geq 1.0 \quad (d \text{ in [m]}) \\
\rho &= 0.02
\end{align*}
\]
It can be seen from Figures 15 and 16 that the current building codes significantly overestimate the shear capacities of slabs I-150-C and I-200-C.

4.1 Shear resistance components

In case of concrete members reinforced with conventional steel bars, the applied shear stress is resisted by the following components:\cite{19}: (I) shear strength of the uncracked concrete, $V_{cz}$; (ii) aggregate interlock, or interface shear transfer, force $V_a$, tangential along a diagonal crack and similar to a frictional force due to irregular interlocking of the aggregates along the rough concrete surfaces on each side of the crack; (iii) dowel action force, $V_d$, developed in the longitudinal tension reinforcement functioning as a dowel between the two segments of the beam at the crack.

Prior to flexural cracking, the applied shear is resisted almost entirely by the uncracked concrete. At flexural cracking, there is a redistribution of internal stresses and some measure of interface shear and dowel action develop. After an inclined crack has formed, the ability of a concrete member to carry additional shear depends on whether or not the portion of shear formerly carried by uncracked concrete can be redistributed across the inclined crack. For rectangular concrete sections without shear reinforcement, it is reported\cite{19,20} that after an inclined crack has formed, the portion of the shear transferred by the various mechanisms is as follows: 15 to 25\% by dowel action; 20 to 40\% by uncracked concrete compression zone; and 33 to 50\% by aggregate interlock or interface shear transfer.

The nature of the crack pattern, propagation and height in FRP reinforced concrete members was found to be different from that in steel reinforced concrete members. In the experimental program described in this research, the immediate crack width in slabs reinforced with FRP bars were much larger than those in slabs reinforced with conventional steel. The crack width of FRP reinforced concrete members was found\cite{21} to be larger than that of steel reinforced concrete members. The crack width in members reinforced with FRPs was evaluated as $(E_s / E_f)$ times that in members reinforced with conventional steel. Assuming that the aggregate interlock force is inversely proportional to the crack width, this force can be estimated in members reinforced with FRPs as $(E_f / E_s)$ times that in members reinforced by steel.

Tests were carried out in Japan\cite{22} to determine the force carried by dowel action in FRP reinforced concrete sections. It was found that the ratio of the dowel force carried by fibre reinforced plastic rebars to that carried by conventional steel rebars is equal to $(E_f / E_s)^{1/3}$. In addition, based on the experimental study described in this research, it was found that the ratio of the depth of neutral axis at failure for FRP reinforced concrete members to the depth of the neutral axis at steel yielding for steel reinforced concrete members is also $(E_f / E_s)^{1/3}$. Hence it can be assumed the shear force carried by the uncracked concrete in compression zone in FRP
reinforced concrete slabs is \((E_f/E_s)^{1/3}\) times that in steel reinforced concrete sections.

Applying the above ratios to the different components contributing to the shear capacity of the concrete members, the shear strength of GFRP reinforced concrete sections is expected to be 28 to 50% of the shear strength of steel reinforced concrete sections. The shear strengths of slabs I-150-C and I-200-C, failed in shear, were 35% and 47%, respectively, of the shear strength predicted by Equation (14) given by the ACI Code\(^{(12)}\) for members reinforced with conventional steel.

Many Investigations\(^{(14,15,23)}\) have been carried out on flexural capacities of FRP reinforced concrete members. Comparatively little attention has been directed toward the shear capacity of such members. Tottori and Wakui\(^{(22)}\) conducted an experimental program to evaluate the shear capacity of FRP reinforced concrete beams. They pointed out that the shear capacity of such beams is smaller than that estimated for beams reinforced with conventional steel. Based on their experimental results, they found that the shear capacity equation used for conventional steel, used in their research, has to be multiplied by a coefficient of \((E_f/E_s)^{1/3}\) to calculate the shear capacity of FRP reinforced concrete beams. Hence, the shear capacity of FRP reinforced concrete beams without shear reinforcement can be estimated according to Equation (14) in N-mm units as:

\[
V_c = 200 \times 10^{-3} \left( 100 \rho f'_c \frac{E_f}{E_s} \right)^{1/3} d^{-1/4} [0.75 + \frac{1.4}{(a/d)}] b d
\]  

(18)

where \(a' = \) shear span. The shear capacities predicted by Equation (14) are compared to the ultimate shear capacities of the test slabs in Table 3. It can be seen from Table 3 that the predicted shear capacities of slabs I-150-C and I-200-C are, respectively, 80% and 20% above the experimental shear failure load.

4.2 Modification method 1:

The first modification method, labelled Modification 1 on Figures 15 and 16, considers the ratio \(E_f/E_s\) as a multiplication to the entire shear capacity based on Equations (13) to (17). Modification 1 is represented by Equations (19) to (23):

**CSA CAN-A23.3-M94 (Modified 1) [SI]:**

\[
V_c = 0.2 \left( \frac{E_F}{E_s} \right) \sqrt{f'c} b_w d \quad [N]
\]  

(19)
ACI Clause 11.3.2.1 (Modified 1) [SI]:

\[ V_c = \frac{1}{7} \left( \sqrt{f'_c} \cdot 120 \cdot \frac{V_y \cdot d}{M_u} \right) \left( \frac{E_F}{E_s} \right) b_w \cdot d \leq 0.3 \sqrt{f'_c} \left( \frac{E_F}{E_s} \right) b_w \cdot d \] (20)

JSCE 1984 (Modified 1) [SI]:

\[ V_c = 0.94 \sqrt{f'_c} (1 + \beta d) \beta_p \beta_n \left( \frac{E_F}{E_s} \right) b_w \cdot d \quad [kg] \] (21)

Machida (JSCE 1996) (Modified 1) [SI]:

\[ V_c = 0.9 \beta_d \beta_p \beta_n \sqrt{f'_c} \left( \frac{E_F}{E_s} \right) b_w \cdot d \quad [kg] \] (22)

CEB-FIP (Modified 1) [SI]:

\[ V_c = \tau_{rd} k (1.50 \rho) \left( \frac{E_F}{E_s} \right) b_w \cdot d \quad [N] \] (23)

4.3 Modification method 2:

The second modification method, labelled Modification 2 on Figures 15 and 16 and based on Machida’s approach, considers only the reduction in shear capacity due to the effect of elastic modulus on the reinforcement ratio. Modification 2 is represented by Equations (24) to (28). It should be noted that for the equations that do not include the reinforcement ratio parameter, the proposed modification 1 is identical to Modification 2.

CSA CAN-A23.3-M94 (Modified 2) [SI]:

\[ V_c = 0.2 \left( \frac{E_F}{E_s} \right) \sqrt{f'_c} b_w \cdot d \quad [N] \] (24)
ACI Clause 11.3.2.1 (Modified 2) [SI]:

\[ V_c = \frac{1}{7} \left( \sqrt{f'_c} \cdot 120 \rho \left( \frac{V_u}{M_u} \right) \left( \frac{E_P}{E_s} \right) \right) b_w d \leq 0.3 \sqrt{f'_c} b_w d \]  

(25)

JSCE 1984 (Modified 2) [SI]:

\[ V_c = 0.94 \sqrt{f'_c} (1-\beta_d \beta_p \beta_n) b_w d \]  

[kg]  

(26)

where: \( \beta_p = \sqrt{100 \rho \left( \frac{E_P}{E_s} \right)} - 1 \leq 0.73 \)

Machida (JSCE 1996) (Modified 2) [SI]

\[ V_c = 0.9 \beta_d \beta_p \beta_n \sqrt{f'_c} b_w d \]  

[kg]  

(27)

where: \( \beta_p = \sqrt{100 \rho \left( \frac{E_P}{E_s} \right)} \leq 1.5 \)

CEB-FIP (Modified 2) [SI]:

\[ V_c = \tau_{Rd} k \left( 1 + 50 \rho \left( \frac{E_P}{E_s} \right) \right) b_w d \]  

[N]  

(28)

The measured ultimate shear loads for slabs I-150-C and I-200-C are compared to the predicted values based on the two modifications in Figures 15 and 16, respectively. The comparison indicate that modification 1 is conservative for the two test slabs for all design codes. Machida’s\(^{17}\) approach, used in modification 2, was found to overestimate the shear capacity, as shown in Figures 15 and 16, thus, remaining as an unsafe prediction.

4.4 Proposed method for estimating the shear capacity of FRP Reinforced concrete slabs

Based on an experimental study, Zsutty\(^{24}\) introduced an expression for estimating the shear capacity of simply supported concrete beams reinforced with conventional steel. The expression given by Equation (29), in SI units, includes the effect of the shear span and the reinforcement ratio.
Applying the same concept proposed by Tottori and Wakui\(^{22}\), the shear capacity predicted by Equation (30) for members reinforced with conventional steel can be multiplied by \((E_r/E_s)^{1/3}\) to predict the ultimate shear capacity of FRP reinforced concrete members. Therefore Equation (29) can be rewritten in the following form:

\[
V_c = 2 \left( f'_{c} \cdot \rho \cdot \frac{d}{a} \right)^{1/3} b \cdot d
\]  

(30)

The shear capacities of the test-slabs were estimated according to Equation (30) and compared to the experimental results in Table 4. It can be seen from Table 4 that Equation (30) provides a lower estimate for the shear capacity than Equations (13), (14) and (18). Also it can be seen from Figures 15 and 16 that the ratio \(V_{exp}/V_c\), based on Equation (30), is 0.87 and 1.10 for slabs I-150-C and I-200-C, respectively. Consequently, until further research is conducted on the shear strength of FRP reinforced concrete slabs, Equation (30) can be used to estimate the shear capacity of such slabs.

5. SUMMARY AND CONCLUSIONS

Eight one-way reinforced concrete slabs, with clear span of 3000 mm (10 ft), were tested under static loading conditions. Five slabs were reinforced with GFRP bars, two slabs were reinforced with conventional steel rebars and one slab was reinforced with CFRP rods. Based on the test results, analytical methods are proposed to estimate the ultimate flexural and shear capacities of such slabs. According to the results from this investigation the following conclusions can be made:

1) Behaviour of the FRP reinforced concrete specimens was bilinearly elastic until failure. Stiffness of the slabs reinforced by GFRP reinforcements is significantly reduced after initiation of cracks in comparison to the slab reinforced by steel rebars. The slabs showed adequate warning prior to failure through large and deep cracks, accompanied by large deformations for slabs reinforced by GFRP bars.

2) Strains, curvatures and hence deflections are generally higher in FRP reinforced concrete slabs than in steel reinforced concrete slabs. Therefore, serviceability of such slabs, particularly deflection and cracking, has to be checked.
3) For the under-reinforced slabs, designed to fail by rupture of the reinforcements, the GFRP bars did not reach the ultimate strain based on pure tension tests of the bars. This behaviour could be attributed to the localized failure of the fibres at the crack, due to the sudden transfer of tensile forces from the concrete to the GFRP bars at the crack. Increasing the diameter of the GFRP bars could also magnify the effect of the interlaminar shear lag phenomenon and consequently reduce ultimate tensile stress of the GFRP bar. These two factors lead to the development of Equation (12) proposed to estimate the capacity of GFRP bars for flexural behaviour.

4) Due to the reduced compressive stress block size and due to the nature of cracking of FRP reinforced concrete slabs, shear strength of such slabs is 28-50% of the shear strength of slabs reinforced with conventional steel.

5) The shear capacity provided by the current code equations significantly overestimate the shear capacity of the slabs reinforced by GFRP bars. A conservative estimate for the shear capacity can be obtained by modifying current code equations for shear by the ratio of the elastic moduli of GFRP and steel, $E_r/E_s$.

6) Bond strength of Leadline rebars is slightly less than that of steel. Therefore design of concrete slabs reinforced with Leadline should include the bond capacity of the rebars.

7) Until further research is available, a design equation is proposed to estimate the shear capacity of FRP reinforced concrete members. The proposed equation takes into account the ratio and type of reinforcement.

8) Further research should be pursued prior to allowing the use of GFRP as reinforcement for reinforced concrete slabs.

6. ACKNOWLEDGMENTS

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7. NOTATION

\(a\) = shear span;
\(a'\) = depth of equivalent rectangular stress block;
\(A_c\) = cross-sectional area of FRP reinforcement;
\(A_s\) = cross-sectional area of steel reinforcement;
\(b_w\) = width of slabs (b);
\(C\) = depth of neutral axis;
\(d\) = depth to centre of reinforcements, measured from the extreme compression fibre of concrete ([cm] for Japanese equations, [mm] for the rest);
\(d_b\) = diameter of reinforcement;
\(E_c\) = elastic modulus of concrete;
\(E_F\) = elastic modulus of FRP reinforcement in tension;
\(E_s\) = elastic modulus of steel reinforcement in tension;
\(f_c'\) = compressive strength of concrete cylinders;
\(f_s\) = stress in steel reinforcement;
\(g\) = acceleration due to gravity (9.81 m/s\(^2\));
\(M_u\) = ultimate moment capacity;
\(P_{cr}\) = cracking load;
\(\Delta P\) = step wise drop in load after initiation of first crack;
\(P_{ult}\) = ultimate load;
\(V_c\) = concrete contribution of the shear capacity of a reinforced concrete member;
\(V_u\) = ultimate shear strength;
\(\beta\) = ratio of depth of equivalent rectangular stress block to depth of neutral axis;
\(\beta_1\) = material factor for the diameter effect on FRP bars;
\(\beta_2\) = material factor for the general behaviour of a specific FRP bar type in concrete;
\(\beta_d\) = shear factor for d in JSCE Design Code\(^{(16)}\);
\(\beta_p\) = shear factor for \(\rho\) in JSCE Design Code\(^{(16)}\);
\(\beta_{a}\) = shear factor for \(\rho\) in JSCE Design Code\(^{(16)}\);
\(\varepsilon_{cu}\) = strain in concrete at ultimate compression stress (0.003);
\(\varepsilon_F\) = tensile strain in FRP reinforcements;
\(\varepsilon_{fu}\) = ultimate tensile strain in FRP reinforcements;
\(\varepsilon_s\) = strain in reinforcing steel;
\(\phi_c\) = concrete material factor (0.60);
\(k\) = shear factor for d in CEB-FIP Design Code\(^{(18)}\);
\(\lambda\) = concrete type factor (1.0 for normal);
\(\rho\) = reinforcement ratio (\(A_c\) or \(A_s\) / \(bd\));
\(\rho_{b}\) = balanced reinforcement ratio;
\(\sigma_F\) = tensile stress in FRP reinforcements;
\(\sigma_{Fexp}\) = tensile stress achieved in FRP reinforcement at a given stage; and
\(\tau_{Rd}\) = shear factor for concrete strengths in CEB-FIP Design Code\(^{(18)}\).
8. CONVERSION FACTORS

25.4 mm = 1 in.
645.1 mm² = 1 in.²
4.448 kN = 1 kip
6.895 MPa = 1 ksi
0.981 MPa = 1 kg/m²

9. REFERENCES


16. Japan Society of Civil Engineers (JSCE), "Concrete Library," International No. 4, Tokyo, Japan, December, 1984, 329 pages.


20. Taylor, M.P.J., "The Fundamental Behavior of Reinforced Concrete Beams in Bending and Shear," Shear in Reinforced Concrete, Vol. 1, SP-42, Detroit, American Concrete Institute, 1974, pp. 43-77.


Table 1- Details of the Test Specimens

<table>
<thead>
<tr>
<th>Thickness mm (in.)</th>
<th>Reinforcement</th>
<th>Mark No.</th>
<th>$\rho^*$</th>
<th>$\rho/\rho_b$</th>
<th>Predicted Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>150 (5.9)</td>
<td>Steel</td>
<td>S-150-T</td>
<td>0.962</td>
<td>0.23</td>
<td>Yield</td>
</tr>
<tr>
<td></td>
<td>ISOROD</td>
<td>I-150-A</td>
<td>0.487</td>
<td>0.66</td>
<td>Rupture of Rebars</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I-150-B</td>
<td>0.764</td>
<td>0.99</td>
<td>Balanced</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I-150-C</td>
<td>0.955</td>
<td>1.25</td>
<td>Crushing of Concrete</td>
</tr>
<tr>
<td>200 (7.9)</td>
<td>Steel</td>
<td>S-200-T</td>
<td>0.390</td>
<td>0.09</td>
<td>Yield</td>
</tr>
<tr>
<td></td>
<td>ISOROD</td>
<td>I-200-A</td>
<td>0.230</td>
<td>0.31</td>
<td>Rupture of Rebars</td>
</tr>
<tr>
<td></td>
<td></td>
<td>I-200-C</td>
<td>0.774</td>
<td>1.01</td>
<td>Crushing of Concrete</td>
</tr>
<tr>
<td></td>
<td>Leadline</td>
<td>LL-200-C</td>
<td>0.303</td>
<td>0.87</td>
<td>Crushing of Concrete</td>
</tr>
</tbody>
</table>

$\rho = \frac{\text{area of reinforcement}}{b \cdot d}$

Table 2- Tensile Properties of the Reinforcements

<table>
<thead>
<tr>
<th>Reinforcement Material</th>
<th>Ultimate Stress MPa (ksi)</th>
<th>Ultimate Strain [%]</th>
<th>Elastic Modulus GPa (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Average Value</td>
<td>$\sigma^*$</td>
<td>Average Value</td>
</tr>
<tr>
<td>GFRP-ISOROD$^{(7)}$</td>
<td>692 (100)</td>
<td>19.4 (2.8)</td>
<td>1.767</td>
</tr>
<tr>
<td>CFRP-Leadline$^{(9)}$</td>
<td>1970 (285)</td>
<td>1.3</td>
<td>147 (21300)</td>
</tr>
<tr>
<td>Steel-rebar</td>
<td>435$^*$ (62.9)</td>
<td>4.08 (0.61)</td>
<td>0.246$^*$</td>
</tr>
</tbody>
</table>

$^*$standard deviation.
$^1$yield stress.
$^2$yield strain.
Table 3- Flexural Capacities of the Test-Slabs

<table>
<thead>
<tr>
<th>Slab</th>
<th>$P_{cr}$ kN (kips)</th>
<th>$\Delta P/P_{cr}$ %</th>
<th>Predicted Ultimate Load $P'_{ult}$ kN (kips)</th>
<th>Experimental Ultimate Load $P_{ult}$ kN (kips)</th>
<th>$P_{ult}/P'_{ult}$</th>
<th>Predicted Mode of Failure</th>
<th>Observed Mode of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-150-T</td>
<td>40.6 (9.1)</td>
<td>8.1</td>
<td>99.3* (22.3)</td>
<td>109.6* (24.6)</td>
<td>1.10</td>
<td>Yield of Rebars</td>
<td>Yield of Rebars</td>
</tr>
<tr>
<td>I-150-A</td>
<td>24.1 (5.4)</td>
<td>49.4</td>
<td>64.8 (14.6)</td>
<td>60.6 (13.6)</td>
<td>0.94</td>
<td>Rupture of Rebars</td>
<td>Rupture of Rebars</td>
</tr>
<tr>
<td>I-150-B</td>
<td>32.9 (7.4)</td>
<td>49.5</td>
<td>101.0 (22.7)</td>
<td>80.4 (18.0)</td>
<td>0.80</td>
<td>Balanced</td>
<td>Rupture of Rebars</td>
</tr>
<tr>
<td>I-150-C</td>
<td>27.0 (6.1)</td>
<td>36.7</td>
<td>103.9 (23.4)</td>
<td>74.6 (16.7)</td>
<td>NA*</td>
<td>Crushing of Concrete</td>
<td>Shear</td>
</tr>
<tr>
<td>S-200-T</td>
<td>79.0 (17.7)</td>
<td>9.1</td>
<td>101.1* (22.7)</td>
<td>101.6* (22.8)</td>
<td>1.00</td>
<td>Yield of Rebars</td>
<td>Yield of Rebars</td>
</tr>
<tr>
<td>I-200-A</td>
<td>43.9 (9.9)</td>
<td>49.9</td>
<td>65.3 (14.7)</td>
<td>47.1 (10.6)</td>
<td>0.72</td>
<td>Rupture of Rebars</td>
<td>Rupture of Rebars</td>
</tr>
<tr>
<td>I-200-C</td>
<td>44.0 (9.9)</td>
<td>34.3</td>
<td>231.3 (52.0)</td>
<td>158.1 (35.5)</td>
<td>NA*</td>
<td>Crushing of Concrete</td>
<td>Shear</td>
</tr>
<tr>
<td>LL-200-C</td>
<td>51.0 (11.44)</td>
<td>19.4</td>
<td>271.0 (60.9)</td>
<td>259.1 (58.1)</td>
<td>0.96</td>
<td>Crushing of Concrete</td>
<td>Bond</td>
</tr>
</tbody>
</table>

* denotes not applicable
Table 4- Shear Capacities of the Test-Slabs

<table>
<thead>
<tr>
<th>Slab</th>
<th>Mode of Failure</th>
<th>$V_{\text{ex}}$ (kN (kips))</th>
<th>$V_{\text{eq}}$ (Equation (13)) (kN (kips))</th>
<th>$V_{\text{eq}}$ (Equation (14)) (kN (kips))</th>
<th>$V_{\text{eq}}$ (Equation (18)) (kN (kips))</th>
<th>$V_{\text{eq}}$ (Equation (30)) (kN (kips))</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-150-T</td>
<td>Yield of Rebars</td>
<td>60.1 (13.5)</td>
<td>169.0</td>
<td>116.7</td>
<td>NA*</td>
<td>NA*</td>
</tr>
<tr>
<td>I-150-A</td>
<td>Rupture of Rebars</td>
<td>35.6 (8.0)</td>
<td>168.7</td>
<td>121.5</td>
<td>62.2 (14.0)</td>
<td>40.2 (9.0)</td>
</tr>
<tr>
<td>I-150-B</td>
<td>Rupture of Rebars</td>
<td>45.5 (10.2)</td>
<td>169.0</td>
<td>122.4</td>
<td>72.2 (16.2)</td>
<td>46.4 (10.4)</td>
</tr>
<tr>
<td>I-150-C</td>
<td>Shear</td>
<td>42.6 (9.6)</td>
<td>169.0</td>
<td>122.5</td>
<td>77.7 (17.5)</td>
<td>49.0 (11.0)</td>
</tr>
<tr>
<td>S-200-T</td>
<td>Yield of Rebars</td>
<td>57.8 (13.0)</td>
<td>242.6</td>
<td>175.2</td>
<td>NA*</td>
<td>NA*</td>
</tr>
<tr>
<td>I-200-A</td>
<td>Rupture of Rebars</td>
<td>30.6 (6.9)</td>
<td>251.0</td>
<td>180.5</td>
<td>70.9 (15.9)</td>
<td>52.9 (11.9)</td>
</tr>
<tr>
<td>I-200-C</td>
<td>Shear</td>
<td>86.1 (19.4)</td>
<td>250.0</td>
<td>182.0</td>
<td>104.8 (23.6)</td>
<td>78.5 (17.6)</td>
</tr>
<tr>
<td>LL-200-C</td>
<td>Bond</td>
<td>136.6 (30.7)</td>
<td>257.0</td>
<td>185.4</td>
<td>119.4 (26.8)</td>
<td>90.3 (20.3)</td>
</tr>
</tbody>
</table>

* denotes not applicable
LVDT to measure concrete strain

concrete slab 3.5 m long

dial gauge to monitor the slip in the rods

two LVDTs to measure slab deflection

Figure 1(a) : Schematic view of test set-up

Figure 1(b) : Test set-up
Figure 2: Material characteristics of the GFRP, CFRP and Steel reinforcements
Figure 3: Photo of a tension test performed on Isorod
Figure 4(a): Concrete compression strains for slabs reinforced with Leadline and steel rebars.

Figure 4(b): Concrete compression strains for slabs reinforced with Isorod and steel rebars.
Figure 5(a) : Failure mode of slab I-150-C

Figure 5(b) : Close-up of failure crack at left end of slab I-150-C
Figure 6(a) : Failure mode of slab I-200-C

Figure 6(b) : Close-up of shear failure of slab I-200-C
Figure 7: Strain distribution of over-reinforced sections
Figure 8: Strain distribution of under-reinforced sections
Figure 9: Strain distribution of steel and GFRP reinforced concrete slabs
Figure 10: Equivalent rectangular stress distribution
Figure 11: Balanced reinforcement ratio for sections reinforced by Isorod, Leadline and Steel bars

Figure 12: Measured strain of the concrete at the level of reinforcement for slab I-150-A
Figure 13 (a): Load-deflection curve of slab I-200-A

Figure 13 (b): Load-deflection curve of slab I-150-A

Figure 13 (c): Load-deflection curve of slab I-150-B
Figure 14: Close-up of shear rupture of GFRP bar at the crack of slab I-150-C
Figure 15: Comparison of modified and unmodified V_c equations for slab I-150-C

Figure 16: Comparison of modified and unmodified V_c equations for slab I-200-C