PRECAST GFRP
REINFORCED LIGHTWEIGHT CONCRETE BRIDGE DECK PANELS

Chris P. Pantelides, PhD, SE, Civil & Environmental Department, Salt Lake City, UT
Ruifen Liu, PhD candidate, Civil & Environmental Department, Salt Lake City, UT
Lawrence D. Reaveley, PhD, PE, Civil & Environmental Department, Salt Lake City, UT

ABSTRACT

Lightweight concrete results in bridge deck panels that are easier to lift, and its use reduces the bridge deck and substructure weight. There is a paucity of experimental data regarding the use of lightweight concrete reinforced with GFRP bars. Twenty panels constructed using lightweight and normal weight concrete reinforced with GFRP bars for flexure without any shear reinforcement were tested to failure. The variables investigated were concrete compressive strength, deck span, panel thickness and width, and reinforcement ratio. Lightweight concrete panels performed similar to normal weight concrete panels, although they experienced larger deflections under the same loading level and had a lower ultimate load capacity than normal weight concrete panels. An extended database of 97 tests including normal and lightweight concrete restricted to members reinforced with GFRP bars for flexure without any shear reinforcement was compiled. The shear strength of lightweight concrete panels in the database was compared to that of normal weight concrete beams and panels. Analysis of the data has resulted in a proposed reduction factor for sand-lightweight concrete panels reinforced with GFRP bars for possible use in the current ACI 440.1R-06 guidelines. The ultimate load capacity of lightweight concrete panels reinforced with GFRP bars can be predicted with sufficient accuracy, therefore lightweight concrete reinforced with GFRP bars can be used in construction.

Keywords: bridge deck, GFRP bar, lightweight concrete, normal weight concrete, structural behavior, ultimate shear load

INTRODUCTION

Expansive corrosion of steel reinforcing bars in concrete members leads to excessive cracking, spalling, reduced strength, and loss of structural integrity. The corrosion of steel reinforcing bars is a major problem which requires the rehabilitation or reconstruction of concrete members[1]. Bridge deck slabs are one of the bridge components most vulnerable to deterioration because of direct exposure to environment, deicing chemicals, and ever-increasing traffic loads[2]. Glass Fiber Reinforced Polymer (GFRP) bars are noncorrosive, high strength, and lightweight, and are becoming cost-competitive for structures that are vulnerable to corrosion, especially bridge decks and parking garages.

Extensive research has been done on the flexural and shear performance of GFRP reinforced concrete beams or decks. In references 2-9, the authors have investigated the flexural performance of normal weight concrete beams or slabs reinforced with GFRP bars. Even though GFRP bars have different material properties than steel bars, the strain compatibility prediction of flexural capacity is effective. The shear capacity of GFRP reinforced members has also been investigated; in references 10-22, the authors performed research on the shear capacity of normal weight concrete beams or slabs reinforced with GFRP bars without transverse shear reinforcement; all specimens in these studies failed in one-way shear. Design provisions and guidelines have been developed regarding the performance and design of GFRP reinforced concrete structures, such as the Japan Society of Civil Engineers Design Provisions (JSCE[23]), the Canadian Design Provisions (CAN/CSA-S806-02[24]), the American Concrete Institute Guidelines (ACI 440.1R-06[25]), and the American Association of State Highway and Transportation Officials (AASHTO) Bridge Design Guide Specifications for GFRP Reinforced Concrete Decks and Traffic Railings[26].

However, all studies regarding GFRP reinforced concrete members have used normal weight concrete. There is no experimental data regarding GFRP reinforced members cast with lightweight concrete. The JSCE[23] and the ACI 440.1R[25] guidelines do not provide guidance for lightweight concrete reinforced with GFRP bars. The AASHTO GFRP Reinforced Deck Specifications[26] does not allow the use of lightweight concrete for decks reinforced with GFRP bars because of lack of research. The Canadian guidelines CAN/CSA S806[24] consider the effect of concrete density on tensile strength through a modification factor.

The use of lightweight concrete combined with GFRP as reinforcement could benefit the structure, especially when Accelerated Bridge Construction (ABC) is used as the construction method, which reduces on-site construction time and traffic disruption. The noncorrosion properties of GFRP bars could extend the deck life and reduce the life-cycle time cost of decks. The reduced weight of GFRP bars compared to steel bars makes them easier to handle during construction. The reduced weight of decks constructed with lightweight concrete implies that they could be lifted with smaller cranes and could reduce the transportation requirements such as Self Propelled Modular Transporters (SPMTs). In addition, the reduction in weight is beneficial in the design of the superstructures, substructures and foundations since the weight of the deck is the main dead load resisted by the girders, substructures and foundations. Reduction of weight of concrete and reinforcement is also beneficial when seismic forces are considered.

This paper presents the test results of twenty panels reinforced with GFRP bars, twelve of which were cast using lightweight concrete and eight using normal weight concrete. The variables studied in this research include concrete compressive strength, reinforcement ratio, deck thickness, deck span, and panel width. The performance of lightweight concrete as well as normal weight concrete precast panels reinforced with GFRP bars was evaluated, including the panel structural behavior, failure modes, service deflection and ultimate load capacity. Available test data for normal weight concrete reinforced with GFRP bars in flexure were collected and compared to the tests in the present research. A reduction factor is proposed for the ultimate shear capacity of lightweight concrete members reinforced with GFRP bars for the shear prediction equations in the ACI 440.1R[25] recommendations.

MATERIALS AND INSTRUMENTATION

All concrete panels were reinforced with #5 GFRP bars. The guaranteed tensile strength of the specific lot of GFRP bars used was 103,700 psi; the modulus of elasticity was 6,280,000 psi. The compressive strength of normal weight concrete (NWC) at the time of testing ranged from 8,500 psi to 12,600 psi; the compressive strength of lightweight concrete (LWC) ranged from 8,100 psi to 10,900 psi. The lightweight concrete used in this research is sand-lightweight concrete. The coarse hard rock aggregate for NWC had a diameter of ¾ in.; the expanded shale aggregate for LWC had a diameter of ½ in.

Electrical resistance strain gauges were adhesively bonded to GFRP bars to measure strains in the longitudinal and transverse directions. Additional electrical resistance strain gauges were bonded to the top surface of the panels to measure strain in the concrete. Linear variable Differential Transducers (LVDTs) were attached to the bottom of the panels to measure deflections at midspan and quarter span.

SPECIMEN DETAILS

Twenty concrete panels reinforced with GFRP bars were constructed and tested, including twelve lightweight concrete and eight normal weight concrete panels. The NWC panels were designed using the ACI 440.1R flexural design method; the specimens were designed with concrete crushing as the preferred mode of failure. Three of the panels were built using a reinforcement spacing twice that of the flexurally designed panels. All LWC panels were reinforced in an identical manner to NWC panels for comparison.

Four series of panels were built according to their dimensions and reinforcement. Series A and B panels were 2 ft wide, whereas Series C and D panels were 6 ft wide. Tables 1-4 show relevant dimensions and the reinforcement ratio for all panels. The first letter and following number in the specimen number is the batch number; NW=normal weight; LW=lightweight;

the fifth letter (E) when used stands for the case of reduced reinforcement ratio. For Series A, B and C panels, the longitudinal direction was reinforced with #5 @ 4'', while the transverse direction was reinforced with #5 @ 6''. For Series D panels, both longitudinal and transverse direction was reinforced with #5 @ 8''.

Table 1. Series A panels: 2 ft wide x 9 ¼ in. thick with 8 ft span

Specimen Number	f_c' (psi)	ρ_f (%)	ρ_b (%)	Initial crack width (in.)	V_{exp} (kips)	P_{max} (kips)	$\Delta_{ult.}$ (in.)
#1 B1NW	10,370	0.94	0.95	0.002	30.6	59.4	2.09
#2 B2NW	12,650	0.94	1.16	0.002	30.3	58.6	1.77
#3 B2NW	8,760	0.94	0.80	0.002	27.6	53.4	1.70
#4 B1LW	9,090	0.94	0.83	0.007	24.3	48.7	1.52
#5 B1LW	10,930	0.94	1.00	0.002	23.1	44.3	1.06
#6 B2LW	8,700	0.94	0.80	0.005	23.2	44.5	1.61
#7 B1LW*	8,730	0.94	0.91	0.002	27.5	53.2	0.69

*span was 6.66 ft (2.03 m)

Table 2. Series B panels: 2 ft wide x 10 ¾ in. thick with 9 ½ ft span

Specimen Number	f_c' (psi)	ρ_f (%)	ρ_b (%)	Initial crack width (in.)	V_{exp} (kips)	P_{max} (kips)	$\Delta_{ult.}$ (in.)
#8 B1NW	11,420	0.79	1.05	0.016	24.0	45.5	2.73
#9 B2NW	8,840	0.79	0.81	0.002	28.0	53.3	1.81
#10 B1LW	9,080	0.79	0.83	0.004	22.4	42.3	1.79
#11 B2LW	8700	0.79	0.80	0.003	23.7	44.8	1.88

Table 3. Series C panels: 6 ft wide x 9 ¼ in. thick with 8 ft span

Specimen Number	f_c' (psi)	ρ_f (%)	ρ_b (%)	Initial crack width (in.)	V_{exp} (kips)	P_{max} (kips)	$\Delta_{ult.}$ (in.)
#12 B1NW	12,130	0.96	1.11	0.007	87.6	169.8	3.31
#13 B2NW	8,510	0.96	0.78	0.002	72.7	140.1	2.64
#14 B1LW	9,080	0.96	0.83	0.003	61.8	118.1	1.83
#15 B1LW	9,080	0.96	0.83	0.007	65.2	125.0	3.32
#16 B2LW	8,250	0.96	0.76	0.002	67.2	129.0	2.63
#17 B2LW	8,060	0.96	0.74	0.005	68.0	130.6	1.65

Table 4. Series D panels: 6 ft wide x 9 ¼ in. thick with 8 ft span

Specimen Number	f_c' (psi)	ρ_f (%)	ρ_b (%)	Initial crack width (in.)	V_{exp} (kips)	P_{max} (kips)	$\Delta_{ult.}$ (in.)
#18 B1NWE	12,130	0.54	1.11	0.005	62.1	118.8	2.96
#19 B1LWE	9,080	0.54	0.83	0.009	56.0	106.6	2.35
#20 B2LWE	8,060	0.54	0.74	0.005	49.7	94.0	1.99

TEST SETUP AND PROCEDURE

Panels were simply supported on two reinforced concrete beams, and tested in three-point loading, as shown in Fig. 1. The load was applied using a hydraulic actuator through a 10 in. x 20 in. x 1 in. steel bearing plate which was loaded to simulate the area of a double tire truck load on a bridge deck (AASHTO[27]). The load was applied as a series of half-sine downward cycles of increasing amplitude without stress reversals. The load application was displacement controlled, with a slow constant loading rate of 0.2 in./min. The loading procedure used for the actuator displacement is shown in Fig. 2, where downward displacement is positive. The loading scheme was intended to simulate the repeated truck loading on the panels of a precast concrete bridge deck.

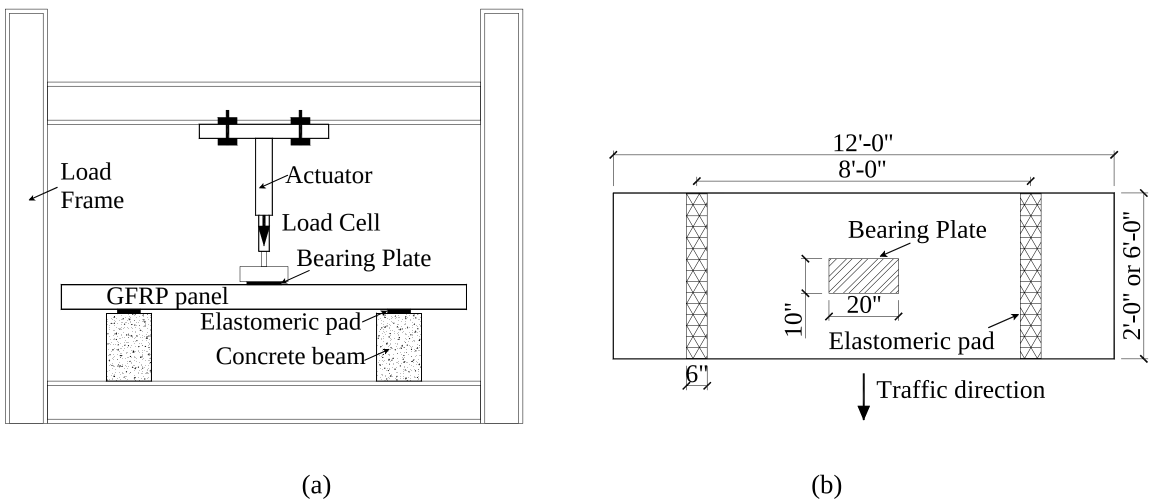

Fig. 1 Test setup: (a) elevation; (b) plan

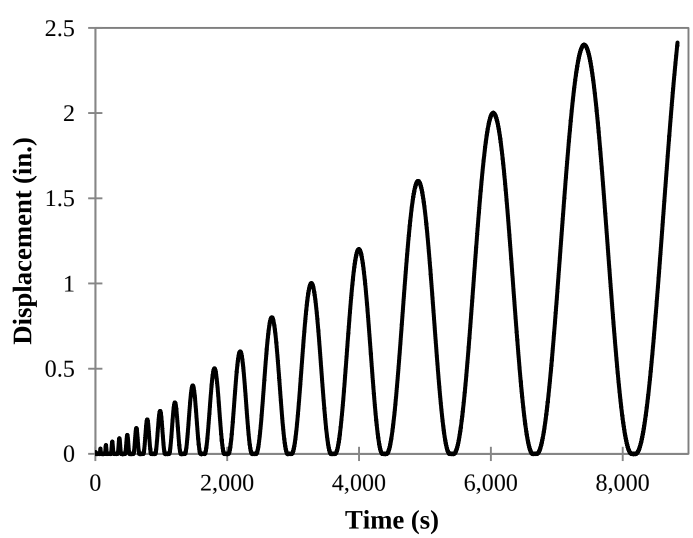

Fig. 2 Loading procedure for actuator displacement

EXPRIMENTAL RESULTS AND DISCUSSION

LOAD DEFLECTION DIAGRAM

All the tested panels failed in one-way shear. This was expected since as shown in Figure 1 the panels were supported on two concrete beams, simulating two girders in a bridge. Flexural cracks were developed at lower load levels, and diagonal cracks were developed at higher load levels near failure of the panels. The diagonal tension failure mode is shown in Fig. 3 for 2 ft wide panels and 6 ft wide panels. Even though the reinforcement ratio of some of the panels is lower than the balanced reinforcement ratio because of the higher concrete compressive strength than the designed concrete compressive strength, the panels failed in concrete crushing failure mode. All panels failed with the same failure mode regardless of concrete type, panel dimensions, and amount of reinforcement. The GFRP bars in the bottom mat did not fracture in any of the tests even though they experienced significant deformation. The measured maximum load ($P_{\max}$) and ultimate deflection (Δ_{ult}) of each panel is provided in Tables 1-4.

(a) (c)

(b) (d)

Fig. 3 Diagonal tension failure of 2 ft and 6 ft wide panels: (a) 2 ft wide NWC GFRP panel; (b) 2 ft wide LWC GFRP panel (c) 6 ft wide NWC GFRP panel; (d) 6 ft wide LWC GFRP panel

Load deflection envelopes were developed by connecting points of the maximum load for each cycle. Figures 4-6 show that the load deflection envelopes are generally bilinear. The first line segment is up to the point where the sections become cracked; in this segment, the panels have a higher stiffness. The second line segment is from the point where the sections crack until failure of the panels. After the section becomes cracked, both NWC and LWC panels had a much smaller stiffness, approximately 13% to 35% of the initial stiffness. In addition, after the section became fully cracked, the stiffness of the panels with reduced reinforcement ratio (Series D) was 60% of the stiffness of the Series C panels with the higher reinforcement ratio, for both NWC and LWC panels, respectively.

As discussed earlier, Series B panels had a larger span than Series A panels. A comparison of the stiffness before and after the section cracked was carried out: Series B panels had 80% the stiffness of Series A panels before the section was cracked; after the section was cracked, Series B panels had 83% the stiffness of Series A panels. Even though Series D panels had about half the reinforcement ratio of Series C panels, Series D panels had 94% the stiffness of Series C panels before the section was cracked; after the section cracked, Series D panels only had 60% of the stiffness of Series C panels. Comparing the stiffness of panels which had the same reinforcement but different width, Series A panels had 50% the stiffness of Series C panels before the section was cracked; after the section was cracked, Series A panels had a stiffness 39% the stiffness of Series C panels. For panels with wider width, the stiffness is larger both before and after the section cracked. However, this may be caused by the loading procedure, in which only a double tire area was loaded and the panels with larger width could distribute the load from the loaded area to the unloaded areas.

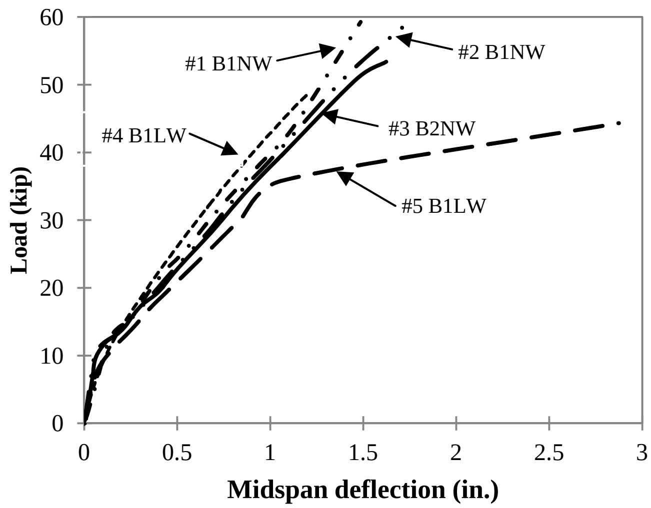

Fig. 4 Load-deflection diagrams for 2 ft wide Series A panels

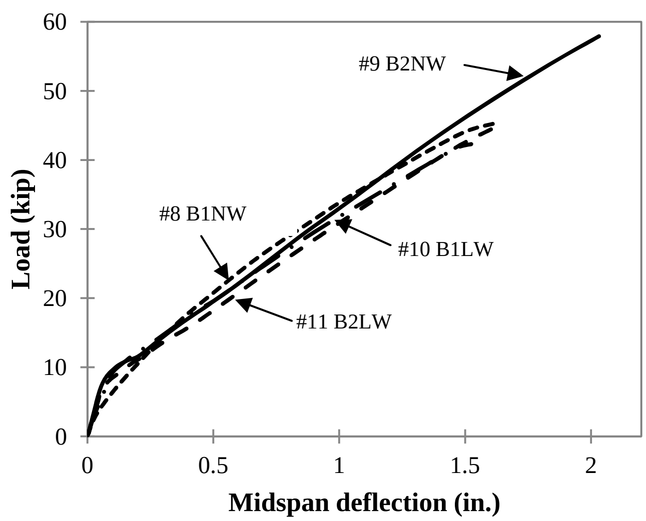

Fig. 5 Load-deflection diagrams for 2 ft wide Series B panels

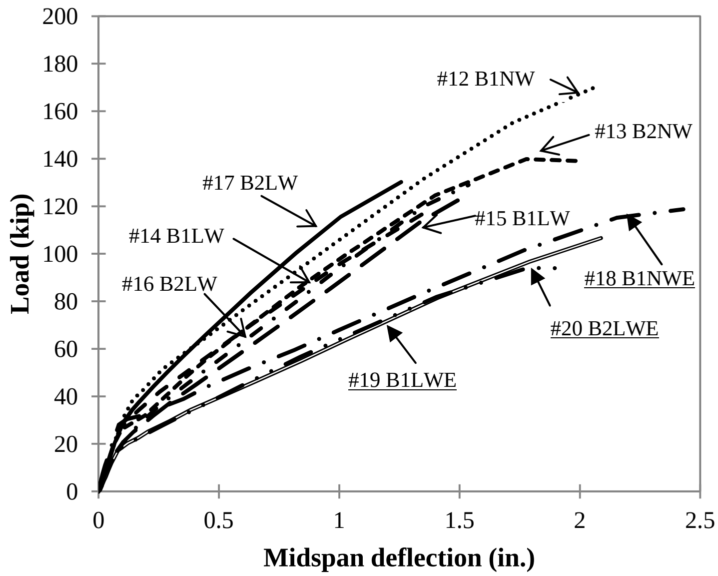

Fig. 6 Load-deflection diagrams for 6 ft wide Series C and Series D panels (underlined)

SERVICEABILITY REQUIREMENTS AND ULTIMATE DEFLECTION

The tested GFRP reinforced precast concrete panels were checked for deflection at the service moment according to the AASHTO LRFD Bridge Design Specifications[27]; all panels satisfied the deflection requirement. The AASHTO Specifications use an HL-93 live load for the service and ultimate load design of bridge decks. According to AASHTO, the HL-93 live load consists of either a design truck or tandem, combined with a design lane load. In the present case, the service moments were calculated based on the following assumptions: for the 2 ft wide specimens only one set of wheels from a truck could be placed on the panel with a load equal to 16 kips; for the 6 ft wide specimens, two sets of wheels from the tandem could be placed on the panel with a total load equal to 25 kips; the design lane load is a uniform load of 640 lbs per linear foot of load lane. GFRP bars have a higher tensile strength than steel bars but a much smaller modulus of elasticity; typically, service load deflection controls the design of GFRP reinforced members. Table 5 shows the deflection of the panels at service and ultimate load. The table shows that under service load, the predicted deflection is much smaller than the experimental deflection. The ratio of experimental deflection to predicted deflection for service load ranged from 2.8 to 7.5 for normal weight concrete panels and 2.9 to 6.8 for lightweight concrete panels, which indicates that the predicted deflection under-estimated the deflection at service load. For ultimate load, the ratio of experimental deflection to predicted deflection ranged from 0.8 to 1.2 for normal weight concrete panels, 0.9 to 2.1 for lightweight concrete panels. The ratios of experimental to predicted deflection under both service and ultimate load are comparable for normal weight concrete panels and lightweight concrete panels. The service deflection requirement in AASHTO[27] is $L/800$, where L is the span length; thus the deflection requirement at service load is 0.12 in. and 0.14 in. for the 8 ft span and 9 ½ ft span panels, respectively.

Comparing the experimental deflection measured at the service load, it is found that all panels designed according to ACI guideline satisfy the service deflection requirement of AASHTO[27].

Table 5. Deflection of Panels at Service and Ultimate Load

Specimen	Service load			Ultimate load		
	$\Delta_{experiment}$ (in.)	$\Delta_{prediction}$ (in.)	$\Delta_{experiment}/\Delta_{prediction}$	$\Delta_{experiment}$ (in.)	$\Delta_{prediction}$ (in.)	$\Delta_{experiment}/\Delta_{prediction}$
#1 B1NW	0.079	0.016	4.94	1.486	1.839	0.81
#2 B2NW	0.040	0.015	2.67	1.719	1.802	0.95
#3 B2NW	0.048	0.017	2.82	1.624	1.660	0.98
#4 B1LW	-	-	-	-	-	-
#5 B1LW	0.066	0.020	3.30	2.874	1.390	2.07
#6 B2LW	-	-	-	-	-	-
#7 B1LW	-	-	-	-	-	-
#8 B1NW	0.120	0.016	7.50	1.633	1.645	0.99
#9 B2NW	0.057	0.018	3.17	2.031	2.140	0.95
#10 B1LW	0.074	0.022	3.36	1.524	1.588	0.96
#11 B2LW	0.068	0.023	2.96	1.624	1.689	0.96
#12 B1NW	0.071	0.015	4.73	2.054	1.745	1.18
#13 B2NW	0.074	0.017	4.35	2.008	1.434	1.40
#14 B1LW	0.069	0.021	3.29	1.368	1.240	1.10
#15 B1LW	0.108	0.021	5.14	1.536	1.320	1.16
#16 B2LW	0.110	0.022	5.00	2.631	1.091	2.41
#17 B2LW	0.072	0.022	3.27	1.222	1.393	0.87
#18 B1NWE	0.063	0.015	4.20	2.430	2.023	1.20
#19 B1LWE	0.143	0.021	6.81	2.088	1.886	1.11
#20 B2LWE	0.146	0.022	6.64	1.920	1.638	1.17

ANALYTICAL RESULTS

ULTIMATE SHEAR CAPACITY

In the ACI 440.1R guidelines[25] the concrete shear capacity V_c of flexural members reinforced with GFRP bars that fail in one-way shear is given as:

$$V_c = 5\sqrt{f_c^{'}}b_w c \tag{1}$$

$$c = kd \tag{2}$$

$$k = \sqrt{2\rho_f n_f + \left(\rho_f n_f\right)^2} - \rho_f n_f \tag{3}$$

$$E_c = \left[40,000\sqrt{f_c^{'}} + 10^6\right]\left(\frac{w_c}{145}\right)^{1.5} \tag{4}$$

where c = cracked transformed section neutral axis depth, in.; d = distance from extreme compression fiber to the centroid of tension reinforcement, in.; n_f = ratio of modulus of elasticity of FRP bars to modulus of elasticity of concrete; k = ratio of depth of neutral axis to reinforcement depth; E_c = modulus of elasticity of high-strength concrete, psi.

Tables 1-4 show the experimental shear capacity for each panel; the shear capacity shown includes the dead load of the panels. Comparing the shear capacity of normal weight and lightweight concrete panels, it is found that the lightweight concrete panels have a shear strength that ranges from 82% to 89% of the normal weight concrete panels for the specimens designed with a reinforcement ratio according to ACI 440.1R recommendations, and a ratio of 85% for the specimens with a reduced reinforcement ratio (Series D).

Shear strength data collected from additional research carried out by other investigators (references[10-22]) were also compared with the lightweight concrete shear strength carried out in the current research. The collected additional research data is based on the following requirements: the tested specimens failed in one-way shear; no transverse reinforcement was provided; all the tested specimens were reinforced with GFRP bars. The ratio of experimental to predicted shear strength is shown in Fig. 7.

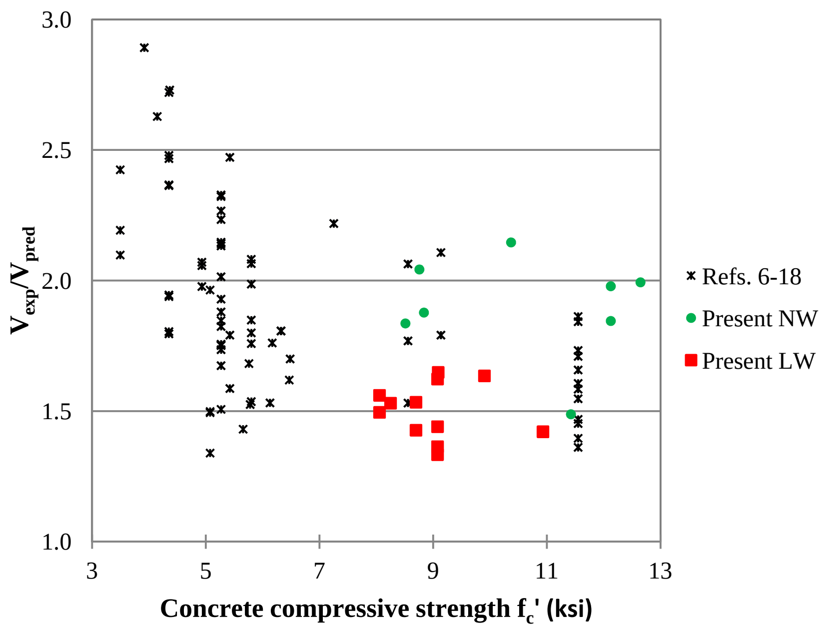

Fig. 7 Correlation of experimental-to-predicted shear strength from ACI 440.1R for extended database

Figure 7 shows that the lightweight concrete panels have a lower ratio of experimental to predicted shear strength than normal weight concrete. The average ratio of experimental to predicted shear strength is 1.91 and 1.50 for normal weight and lightweight concrete beams or slabs, respectively. The coefficient of variation is 18% and 7% for normal weight concrete members and lightweight concrete members, respectively. Figure 7 shows that the type of concrete, normal weight or lightweight, affects the ultimate shear strength of concrete members reinforced with GFRP bars.

PROPOSED EQUATION FOR SHEAR CAPACITY

From the previous discussion, the type of concrete should be considered in predicting the shear capacity using ACI 440.1R. A reduction factor for one-way shear prediction is presented herein by the same procedure used in the ACI Building Code Requirements for Structural Concrete ACI 318[28] to consider the effect of lightweight concrete and introduce the same level of conservatism as for normal weight concrete. The reduction factor is defined as λ in ACI 318 for steel reinforced lightweight concrete members; the value of λ for steel reinforced sand-lightweight concrete is given as $\lambda = 0.85$. Equation (1) is modified for the shear capacity prediction of lightweight concrete panels reinforced with GFRP bars as follows:

$$V_{MOD} = 5\lambda_f \sqrt{f_c^{'}} b_w c \tag{5}$$

The tests carried out in this research and an extended database with 97 beams and one-ways slabs reinforced with GFRP bars for flexure only are used to investigate the applicability of Eq. (5); an appropriate value of the reduction factor λ_f for sand lightweight concrete panels reinforced with GFRP bars is determined. The comparison suggests that a lower-bound reduction value of λ_f equal to 0.80 is appropriate, so that the lightweight concrete members have the same conservatism as normal weight concrete members. Compared with steel reinforcement, GFRP reinforcement has much smaller modulus of elasticity, which indicate that under the same load, GFRP reinforced concrete members have larger deflections and crack widths. The larger crack widths in GFRP reinforced concrete members decrease aggregate interlock between cracks, which accordingly reduces the ultimate shear strength of the members. And because of the higher deflection in GFRP reinforced concrete members, the neutral axis of the section is higher compared with steel reinforced concrete members, which related with smaller area of concrete in compression, it also reduces the shear capacity of concrete members. In addition, the lightweight coarse aggregate used in the research has a diameter of ½ in. while normal weight coarse aggregate has a diameter of ¾ in. Smaller size coarse aggregates in lightweight concrete are surrounded by more cement pastes; when concrete crack, cracks go through from more pastes in lightweight concrete than in normal weight concrete, which also reduces the ultimate shear strength of the concrete members. Considering the two situations, it is reasonably to have a reduction factor of 0.80 compared with 0.85 in ACI 318-08 which was used for steel reinforced concrete members. The ratio of experimental to predicted shear strength using the modified equation is shown in Fig. 8.

Comparing Figs. 7 and 8, it is clear that the proposed modification yields a similar conservatism for normal weight and lightweight concrete panels reinforced with GFRP bars. Using the modified equation, the ratio of experimental to predicted shear strength is 1.91 and 1.88 for normal weight and lightweight concrete members, respectively; the coefficient of variation is 18% and 7% for normal weight and lightweight concrete members, respectively. The comparison with the prediction of Eq. (1) shows that the modified equation yields predictions that are more rational for evaluating the shear strength of sand-lightweight concrete panels reinforced with GFRP bars without shear reinforcement.

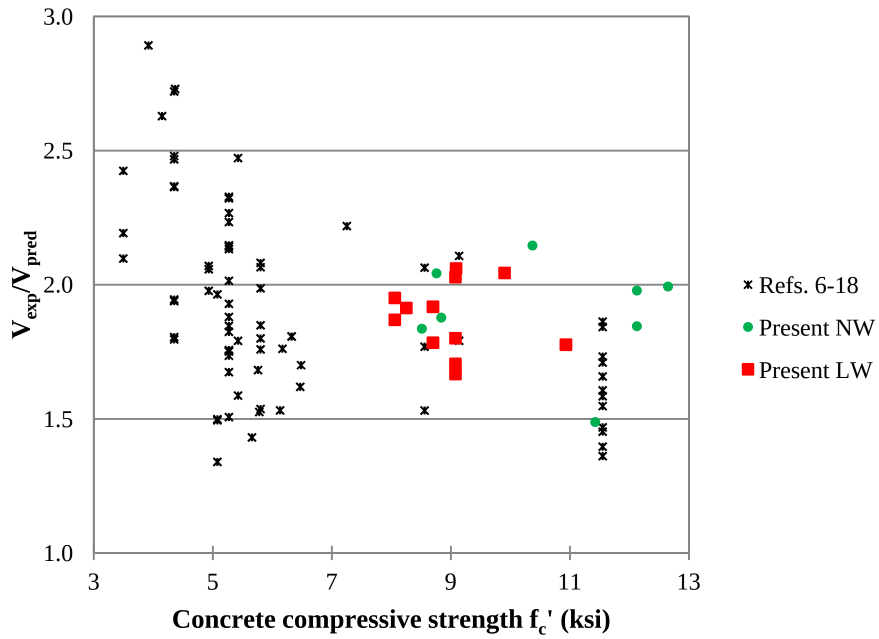

Fig. 8 Correlation of experimental-to-predicted shear strength using modified Eq. (5) for extended database

CONCLUSIONS

The performance of normal weight and lightweight concrete panels reinforced with GFRP bars is investigated in this research. The performance regarding the maximum load, service and ultimate deflection, and ultimate shear capacity of the normal weight and lightweight concrete panels was compared. The main findings of this research can be summarized as follows:

1. All panels failed in one-way shear failure mode, regardless of the concrete type. The load deflection diagrams are generally bilinear.
2. The ACI 440.1R guidelines underestimate the deflection at service load, but predict the deflection at ultimate load accurately. The ratios of experimental to predicted deflection under both service and ultimate load are comparable for normal weight concrete panels and lightweight concrete panels.
3. All panels designed according to ACI 440.1R satisfied the service deflection requirement of the AASHTO Specifications.
4. The average ratio of experimental to predicted shear strength is 1.91 and 1.50 for normal weight and lightweight concrete beams or slabs, respectively. A reduction factor of 0.80 could be used to consider the use of lightweight concrete when GFRP bars are used as reinforcement. Using this reduction factor, the ratio of experimental to predicted shear strength is 1.91 and 1.88 for normal weight and lightweight concrete members, respectively.

5. Lightweight concrete panels reinforced with GFRP bars were shown to satisfy the service deflection requirement; using a reduction factor, the lightweight concrete panels could have the same conservatism regarding ultimate shear capacity as normal weight concrete panels. Lightweight concrete could be used when GFRP bars are used as reinforcement.

REFERENCES

1. Yunovich, M., and Thompson, N., "Corrosion of highway bridges: Economic impact and control methodologies," *American Concrete Institute, Concrete International,* V. 25, No. 1, Jan 2003, pp. 52-57.
2. El-Salakawy, E., and Benmokrane, B., "Serviceability of Concrete Bridge Deck Slabs Reinforced with Fiber-Reinforced Polymer Composite Bars," *ACI Structural Journal*, V. 101, No. 5, Sept.-Oct. 2004, pp. 727-736.
3. Benmokrane, B., Chaallal, O., and Masmoudi, R., "Flexural Response of Concrete Beams Reinforced with FRP Reinforcing Bars," *ACI Structural Journal*, V. 91, No. 2, May-June 1995, pp. 46-55.
4. Michaluk, C. R., Rizkalla, S. H., Tadros, G., and Benmokrane, B., "Flexural Behavior of One-Way Concrete Slabs Reinforced by Fiber Reinforced Plastic Reinforcements," *ACI Structural Journal*, V. 95, No. 3, May-June 1998, pp. 353-365.
5. Masmoudi, R., Theriault M. and Benmokrane, B., "Flexural Behavior of Concrete Beams Reinforced with Deformed Fiber Reinforced Plastic Reinforcing Rods," ACI Structural Journal, V. 95, No. 6, Nov.-Dec. 1998, pp. 665-676.
6. Yost, J. R., Goodspeed, C. H., and Schmeckpeper, E. R., "Flexural Performance of Concrete Beams Reinforced with FRP Grids," *Journal of Composites for Construction*, V. 5, No. 1, Feb. 2001, pp. 18-25.
7. Yost, J. R., and Gross, S. P., "Flexural Design Methodology for Concrete Beams Reinforced with Fiber-Reinforced Polymers," *ACI Structural Journal*, V. 99, No. 3, May-June 2002, pp. 308-316.
8. Prachasaree, W., Gangarao H. V. S., and Shekar V., "Performance Evaluation of FRP Bridge Deck under Shear Loads," *Journal of Composite Materials*, V. 43, No. 4, Feb. 2009, pp. 377-395.
9. El-Mogy, M., El-Ragaby, A., and El-Salakawy, E., "Flexural Behavior of Continuous FRP-Reinforced Concrete Beams," *Journal of Composites for Construction*, V. 14, No. 6, Dec. 2010, pp. 669-680.
10. Swamy, N., and Aburawi, M., "Structural implications of using GFRP bars as concrete reinforcement," *Non-Metallic (FRP) Reinforcement for Concrete Structures, Proc. 3rd Int. Symp.*, Vol. 2, 1997, pp. 503-510.
11. Deitz, D.H., Harik, I.E., and Gesund, H., "One-way slabs reinforced with Glass Fiber Reinforced Polymer reinforcing bars," *Proc., 4th Int. Symp. on Fiber Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, Baltimore, 1999, pp. 279-286.
12. Alkhrdaji, T., Wideman, M., Belarbi, A., and Nanni, A., "Shear Strength of GFRP RC Beams and Slabs," *Proceedings, Composites in Constructions*, J. Figueiras, L. Juvandes, and R. Faria, eds., Balkema, Lisse, the Netherlands, 2001, pp. 409-414.

13. Yost, J.R., Gross, S.P., and Dinehart, D.W. “Shear strength of normal strength concrete beams reinforced with deformed GFRP bars,” *Journal of Composites for Construction*, V. 5, No. 4, Oct. 2001, pp. 263-275.
14. Tureyen, A.K., and Frosch, R., “Shear tests of FRP-reinforced concrete beams without stirrups,” *ACI Structural Journal*, V. 99, No. 4, July 2002, pp. 427-434.
15. Gross, S.P., Yost, J.R., Dinehart, D.W., Svensen, E., “Shear strength of normal and high strength concrete beams reinforced with GFRP bars,” *High performance materials in bridges, Proceedings of the International Conference*, 2003, pp. 426-437.
16. Ashour, A.F., “Flexural and shear capacities of concrete beams reinforced with GFRP bars,” *Construction and Building Materials*, V. 20, No. 10, Dec. 2005, pp. 1005-1015.
17. El-Sayed, A., El-Salakawy, E., and Benmokrane, B., “Shear strength of one-way concrete slabs reinforced with Fiber-Reinforced Polymer composite bars,” *Journal of Composites for Construction*, V. 9, No. 2, April 2005, pp. 147-157.
18. El-Sayed, A.K., El-Salakawy, E.F. and Benmokrane, B., “Shear strength of FRP-reinforced concrete beams without transverse reinforcement,” *ACI Structural Journal*, V. 103, No. 2, March 2006, pp. 235-243.
19. El-Sayed, A.K., El-Salakawy, E.F. and Benmokrane, B., “Shear capacity of high-strength concrete beams reinforced with FRP bars,” *ACI Structural Journal*, V. 103, No. 3, May 2006, pp. 383-389.
20. Alam, M.S., Hussein, A., “Shear strength of concrete beams reinforced with Glass Fiber Reinforced Polymer (GFRP) bars,” *Proceedings of the 9th International Symposium of the Fiber-Reinforced Polymer Reinforcement for Reinforced Concrete Structures*, Sydney, Australia, 2009.
21. Jang, H., Kim, M., Cho, J., Kim, C., “Concrete shear strength of beams reinforced with FRP bars according to flexural reinforcement ratio and shear span to depth ratio,” *Proceedings of the 9th International Symposium of the Fiber-Reinforced Polymer Reinforcement for Reinforced Cocnrete Structures*, Sydney, Australia, 2009.
22. Bentz, E.C, Massam, L., and Collins, M.P., “Shear Strength of Large Concrete Members with FRP Reinforcement,” *Journal of Composites for Construction*, V. 14, No. 6, Dec. 2010, pp. 637-646.
23. Japan Society of Civil Engineers (JSCE), “Recommendation for Design and Construction of Concrete Structures Using Continuous Fiber Reinforcing Materials,” *Research Committee on Continuous Fiber Reinforcing Materials*, No. 23, 1997, 325pp.
24. Canadian Standards Association (CAN/CSA S806), “Design and Construction of Building Components with Fibre Reinforced Polymers,” *Canadian Standards Association*, Rexdale, Ontario, Canada, 2002, 177pp.
25. American Concrete Institute (ACI 440.1R), “Guide for the Design and Construction of Structural Concrete Reinforced with FRP Bars,” *American Concrete Institute*, Farmington Hills, Mich. 2006, 44pp.
26. American Association of State Highway and Transportation Officials (AASHTO), “AASHTO LRFD Bridge Design Guide Specifications for GFRP Reinforced Cocnrete Decks and Traffic Railings,” 1st edition, *AASHTO*, Washington, D. C., 2009, 66pp.
27. American Association of State Highway and Transportation Officials (AASHTO), “AASHTO LRFD Bridge Design Specifications,” 4th Edition, *AASHTO*, Washington, D.C., 2007, 1526pp.

28. ACI Committee 318, “Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (318-08),” *American Concrete Institute*, Farmington Hills, Mich, 2008, 430pp.