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Comparative Evaluation of Concrete Beams Reinforced with Welded Wired Truss and GFRP Rods

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— Abstract—

This study analyzes and compares the behavior of concrete beams reinforced by welded wire truss and the alternative use of reinforcements made with glass fiber reinforced polymers (GFRP). The purpose of the research is to determine the level of safety using GFRP compared to that provided by high-resistance steel reinforcement with the reinforcement ratio acceptable in terms of the Complementary Technical Standard for Masonry (NTC M, 2004) of the Mexican code.

The results show that the use of GFRP is feasible in terms of resistance, as an alternative for application in housing construction, replacing welded wire trusses. The experimental data used in the study come from tests carried out at the Benemérita Universidad Autónoma de Puebla (BUAP) (Sánchez Hernández, 2019). (Sánchez Hernández, 2019).

Keywords:

Reinforced polymers; glass fibers; reinforced concrete; welded wire truss.

In North America, the use of polymer composite rods such as GFRP has taken great demand as a substitute for the traditional steel reinforcement of structural concrete. Its use spreads to larger structures, as the update of the Bridge Design Guide Specifications for GFRP-Reinforced Concrete Bridge Decks and Traffic Railings (AASHTO (GFRP), 2017) seems to indicate, which is added to the already existing Guide for the Design and Construction of Structural Concrete Reinforced with Fiber-Reinforced Polymer (FRP) Bars (ACI 440 1R, 2015). This is due to the low maintenance cost obtained due to its durability and absence of corrosion before the agents that commonly damage the steel, as reported by various investigations carried out by GangaRao, Taly, and Vijay (2007); (R., Cousin, & Benmokrane, 2009); Nanni, Luca, and Jawahery (2014); (Gooranorimi, Gremel, Myers, & Antonio, 2015) and other authors cited by Sánchez Hernández J. A. (2019).

Mexico, with its wide coastline exposed to corrosive agents for traditional steel reinforcement, especially in the Gulf area, due to its low seismicity, could represent an area of opportunity to take advantage of the potential of this type of reinforcement.

Welded wire trusses made of drawn steel took great boom in self-construction due to their low cost and that, together with their high resistance, requires low reinforcement rates, which were allowed by the 2004 (NTC M, 2004) standard. At present, despite its low use, the main problem when using these trusses in homes is the rapid deterioration due to corrosion.

For this reason, the reinforcement based on GFRP rods results in a useful alternative that could surpass in benefits of resistance and durability to the current technology, based on steel, used for these housing developments.

In the present analysis, it was observed that there is an acceptable performance of this material, especially with reinforcement amounts close to the minimum requested by the current regulation (NTC M, 2017).

2. SPECIMENS AND DATA

The specimens used in the comparison were rectangular beams of 15x20 cm in section, whose dimensions are per those established in the Technical Standards of Masonry (NTC M, 2017) for the confinement slabs of the walls and that usually extend between the clearings of windows and doors.

The concrete used has an ultimate compressive strength $f'c>150$ kg/cm^2, which complies with the aforementioned masonry regulations in Mexico and whose variations between specimens are detailed in Table 1.

The value of the ultimate stress at a tension of the GFRP rod was fixed with $ffu = ff^*u = 8000$ kg/cm^2 similar stress ultimate voltage limit set by the manufacturer; $ffv = 1500$ kg/cm^2 as the shearing limit effort set by the manufacturer; and $Ef = 500000$ kg/cm^2 as the elastic modulus established by the manufacturer, for the evaluation of resistant nominal elements. While steel was taken as a reference with the values of fy = 5000 kg/cm², Ey= 2.1E6 kg/cm² and fyv=1600 kg/cm² according to the supplier's specifications.

The tests were carried out in a reaction frame with a 100t hydraulic jack to transmit the load in each element tested, the load was separated into two punctual loads through a load distribution beam rigid enough not to deform. The beam was placed in a system of isostatic supports, 2.0m in distance.

The specimens tested are described below, in Table 1, defining those assembled in the upper bed (LS) and lower bed (LI) as follows: 2D4 that should be read as two bars of 4mm in diameter because the material used, of European origin, is millimeter. While the uniform stirrups of 4mm only their spatial distribution expressed in cm of separation (E@15) was enunciated. The armed ones that were composed with hooks at the ends were designated by a letter "G".

These reinforcements were set based on a competition strategy, with traditional welded wire trusses, seeking cost-capacity optimization.

Table 1

Specimens Description

BEAM	TRUSS		f'c kg/cm²	Af / As Total cm²	Af cm²	AMOUNT		
						ρ_{fb}	$\rho_{fMÍN}$	ρ_{fREAL}
1 (V)	LS	2D4						
	LI	2D4	250.00	0.50	0.25	0.0054	0.0037	0.0009
		E@15						
2 (V1)	LS	2D4						
	LI	2D4	218.50	0.50	0.25	0.0054	0.0037	0.0009
		E@15						
3 (V2)	LS	2D4						
	LI	2D4 + G	279.80	0.50	0.25	0.0060	0.0037	0.0009
		E@15						
4 (V3)	LS	2D4						
	LI	2D4+1D6+G	279.80	1.63	0.53	0.0060	0.0031	0.0020
		E@15						
5 (V4)	LS	2D6						
	LI	3D8 +G	218.50	2.07	1.51	0.0047	0.0031	0.0057
		E@10						
6 (STEEL)	LS	2D6						
	LI	2D6	321.00	1.27	0.63	0.0245	0.0023	0.0023
		E@15						

Source: Own elaboration

The reinforcement areas for steel (As) and fibers (Af) were obtained geometrically considering the area a circle of similar diameter to that of the rod. While the amount of actual reinforcement was calculated as $\rho = A/(b\ d)$, with "A" being the area of reinforcement in the lower bed calculated as indicated and "b" the base of the cross-section, and "d" being the effective cant with a free coating of 2.0 cm.

According to NTC M (2017), considering the fy of welded wire trusses ($fy=5000\ kg/cm^2$) and the minimum acceptable strength for concrete ($f'c=150\ kg/cm^2$ in slabs), the minimum total longitudinal reinforcement corresponds to:

$$As\ min = 0.2\ (f'c/fy)\ b\ h$$

Thus, for the steel reinforced trusses, the minimum will be:

$$As\ min = 0.2\ (150/5000)\ (15)(20) = 1.80\ cm^2 \ldots\ (Q1)$$

Meanwhile, for GFRP, and applying the same criteria, the minimum reinforcement is:

$$ffd = CE\ ff^*u \qquad (ACI\ 440\ 1R\ ,\ 2015)(6.2\ a)$$

Where: $ff^*u = 8{,}000\ kg/cm^2$ y $ffd = CE\ ff^*u = 6{,}000\ kg/cm^2$

$$Af\ mín = 0.2\ (150/6000)\ (15)(20) = 1.50\ cm^2 \ldots\ (Q2)$$

CE corresponds to the Environmental Quotient, established by the ACI 440.1R for different exposure levels, which was given the value of 0.75.

Based on these calculations, the GFRP reinforced beams, listed in Table 1, from 1 to 3 have armed (Af) lower than the minimum calculated in Q2, while beams 4 and 5 would be acceptable to confine according to the NTC M (2017). However, the present work focuses on comparing the behavior to bending and cutting through its resistant mechanical elements, between the slabs (confinement beams) armed with the materials described above. Therefore, all these elements will be compared with beam 6 reinforced with metal elements and with an assembly area (As) described in the same table less than the normative minimum (Q2).

The actual, balanced, and minimum amounts reflected in Table 1, for steel reinforcement, were calculated using the following expressions represented in the NTC CR (2017) (5.1.5).

$$\rho = As\ /\ (b\ d)$$
$$\rho\ mín = 0.7\ (f'c)^{1/2}/fy$$
$$\rho b = (f''c/fy)\ [(6000\ \beta 1)/(fy+6000)]$$

Where "As", is the reinforcement to bending; "d", is the effective cant and b=15 cm, the cross-section base; being $\beta 1=0.85$ and $f''c=0.85f'c$.

While the assessment of the amounts for GFRP was carried out according to the ACI design code (ACI 440 1R , 2015) using units of kg and cm.

$$\rho = Af / (b\ d)$$
$$\rho fb = 0.85\beta 1\ (f'c / ffd)\ [0.003{*}Ef / (0.003{*}Ef+ffd)]$$
$$\rho mín = 1.31\ (f'c^{1/2}) / ffd \geq 23.43 / ffu$$

The load history and deflections were obtained by electronic and digital sensors, from the reports generated from the tests carried out, except in beam 1, whose measurement was made with a micrometer with the equipment and procedure of application of the load described by Sánchez Hernández (2019).

3. ANALYSIS OF MECHANICAL ELEMENTS

The determination of the resistant nominal moments was made for the reinforcement with steel, per the provisions of the NTC Mexico City (2017) for concrete beams reinforced with steel rods in its section (5.1.3) relating to Flexural strength, as well as section (5.3.3) corresponding to Shear strength resistance. While for those relating to GFRP reinforced beams, what is stated in ACI 440 1R (2015), chapter 7.2 on Flexural strength and chapter 8 on Shear stregnth were used. The doubts of interpretation on the design with GFRP were resolved supported by the work of Wainshtok Rivas, Hernández Caneiro, and Díaz Pérez (2015).

4. COMPARISON AND ANALYSIS OF RESULTS

Bending moment

Figure 1 shows the comparative graph of moments between the tests corresponding to the beams armed with GFRP (beams 1 to 3) and the welded wire truss beam (beam 6), all of them with a reinforcement amount (ρ) lower than those which Mexican regulations (NTC 2017 M) allow as reinforcement of these elements (see Table 1).

In graph 1, the progress of the load could be observed that both the GFRP beams and the metal reinforcement beams presented slippage by adhesion and load redistribution, until failure. Beam 3, among those that were armed with GFRP, showed a better behavior by developing its resistance with greater uniformity and lower deflections, this is possibly due to the existence of hooks at the ends of its assembly in the lower bed, which shows the importance of its use. Likewise, in this graph, the resistant moment calculated by the expressions of each regulation was represented with horizontal lines, as cited in title 3 of this work.

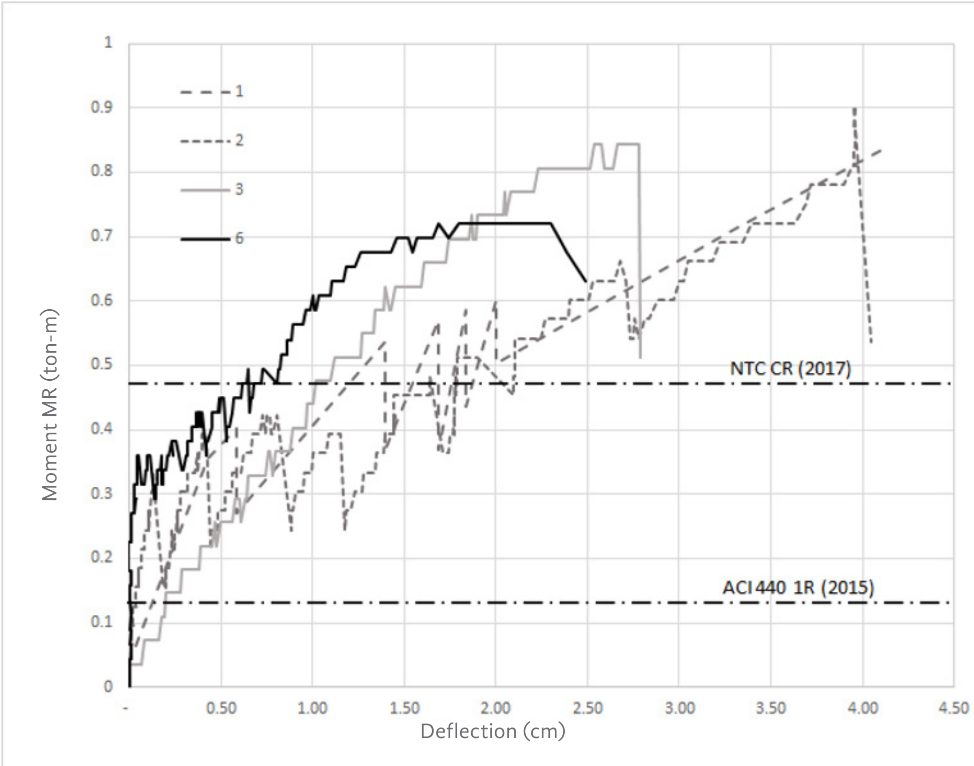

Figure 1. Comparison Chart of the Moments between GRP beams 1 to 3 and steel beam 6.
Source: Own elaboration

Graph 2 shows GFRP beams whose reinforcements are more attached to those calculated at a minimum (Q1 and Q2) according to the NTC M (2017) for the confinement of masonry walls. These beams (4 and 5) showed a better performance in terms of the development of bending resistance, being even greater than the resistance achieved by the beam reinforced with drawn steel. Specifically, beam 4 is very close in real amount (from Table 1, $\rho = 0.0020$) to that used in the bending reinforcement of beam 6 ($\rho = 0.0023$), however, its resistance with GFRP at the moment reaches almost twice the moment achieved by the steel reinforcement. Even with this, both reinforcements reached failure values higher than those evaluated according to their applicable standards.

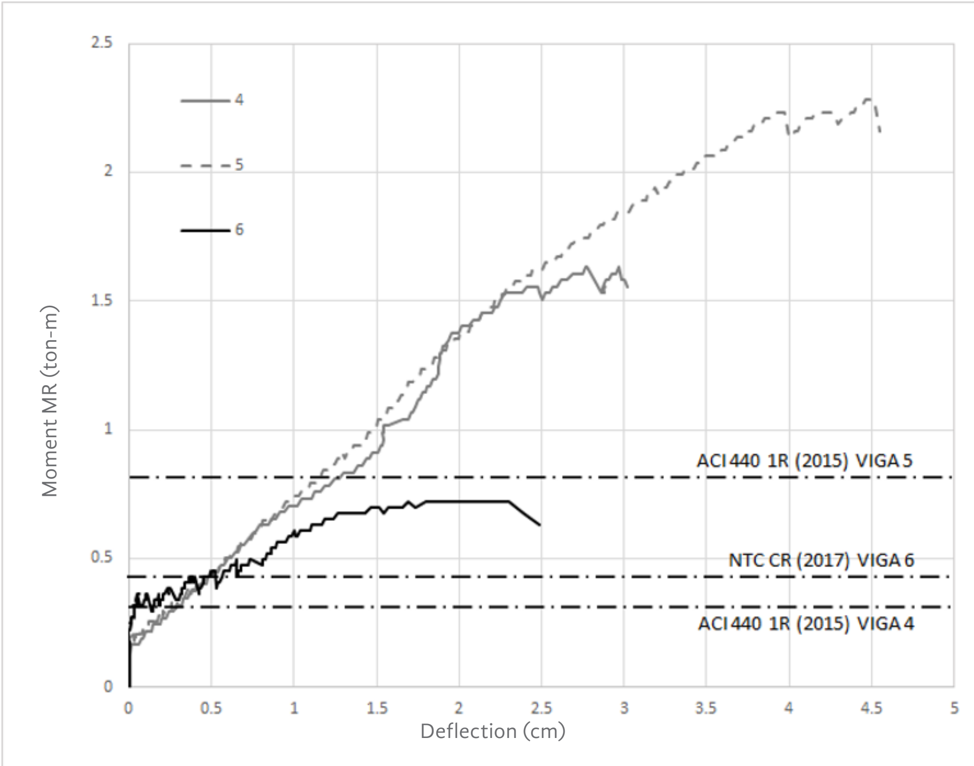

Figure 2. Comparison Chart of the Moments between GRP Beams 4 and 5 and Steel Beam 6.
Source: Own elaboration

In Table 2, it can be seen that the nominal resistant moment (MR), calculated according to ACI 440 1R (2015) for beams armed with GRP, results overall in more conservative values than those referred to by the NTC CR (2017).

However, to make an adequate comparison between the values reached by the design against the experimental values, we ended up using a safety factor defined as FS=Mmax/MR, in which the wide margin of safety in the bending design for these elements under-forced with GFRP can be appreciated. Similar conclusion on safety in GFRP design is made by Joaquím L. and Diaz Pérez (2017) in the reception work called "Estructuras de hormigón armado con barras de Polímero Reforzado con Fibras de Vidrio (PRFV). Estado del arte.", in which he mentions that "The formulas proposed by the ACI 440 ... were considered too conservative because they are based on the controlled compression domain (concrete crush failure)."

Table 2
Safety Factor Design per Bending Moment

BEAM	BENDING		
	MR_{CALC} Kg-m	MR_{REAL} Kg-m	FS
1 (V)	146.39	836.00	5.71
2 (V1)	146.39	806.67	5.51
3 (V2)	146.39	863.33	5.90
4 (V3)	302.44	1,596.67	5.28
5 (V4)	833.91	2,246.67	2.69
6 (STEEL)	487.79	836.00	1.71

Source: Own elaboration

Shear strength

The comparative evaluation of the shear strength also shows the same behavior in terms of the safety factors (FS=Vmax/VR) obtained from the experimental values and those obtained by the conventional analysis methodology according to the applicable design code according to the reinforcement material. GFRP designs represent higher safety indexes against the evaluation of steel assemblies (see Table 3).

It is worth observing that beam 4 reinforced with GFRP and beam 6 reinforced with steel have a very similar reinforcement amount, as far as we notice. However, the shear strength of the steel-reinforced beam, contrary to the result seen in bending, was higher than that of GFRP (Figure 3). That is, the beam with welded wire truss when compared with those of GFRP of a similar amount (beam 6 with 4) presented a better performance in resistance to cutting and displacement, as well as an FS very similar to that obtained with that of GFRP.

It can be observed that the development of the failure to cut in the beams reinforced with GFRP stirrups was linear to the limit. This behavior observed in these beams reinforced with GFRP stirrups suggests paying attention to the adequate design of the shear reinforcement due to the fragility of the failure and the lower margin of clearance obtained in the FS, when compared to that obtained by bending of the same beams.

Table 3

Safety Factor Design per Shear Strength

BEAM	SHEAR		
	VR_{CALC} Kg	VR_{REAL} Kg	FS
1 (V)	596.58	1,200.00	2.01
2 (V1)	664.48	1,210.00	1.82
3 (V2)	604.16	1,295.00	2.14
4 (V3)	704.88	2,395.00	3.40
5 (V4)	1,230.31	3,370.00	2.74
6 (ACERO) 6 (STEEL)	1,491.31	2,660.00	1.78

Source: Own elaboration

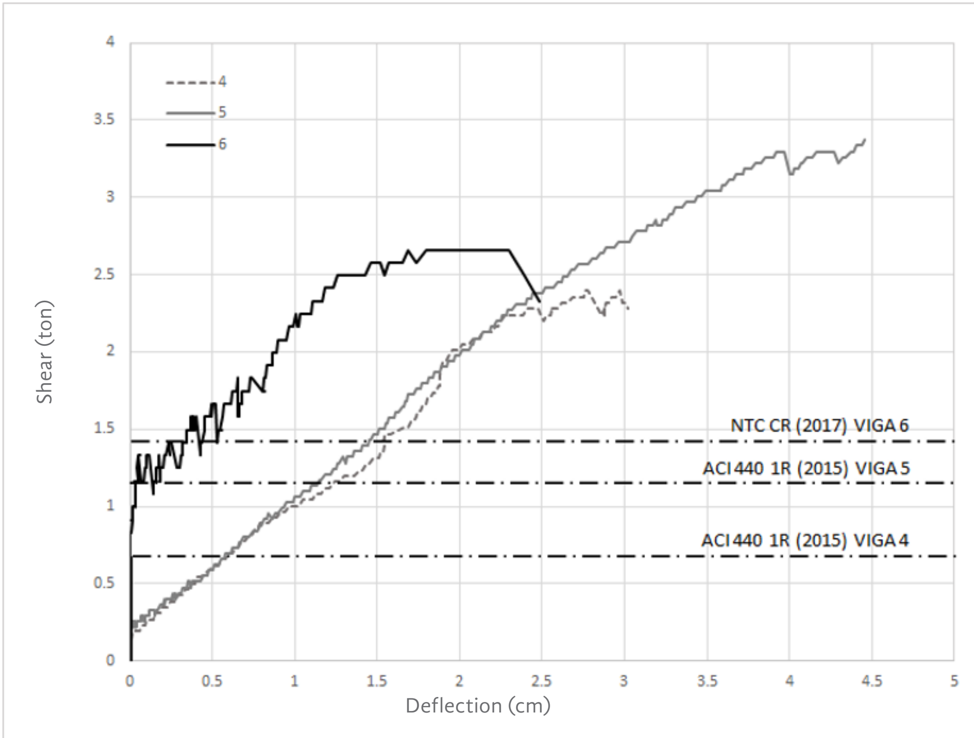

Figure 3. Shear Comparative Chart between GRP beams 4 and 5 and steel beam 6. Source: Own elaboration

Figure 4. Failure in beams, collapse by shear: beam 4. Source: Own elaboration

The FS obtained, which relates the resistance empirically obtained in the tests with that evaluated by the normative theoretical models, seems to be very promising. However, it will have to take into account that most of the GFRPs available on the market do not yet have standardized capacities, so these are subject to the manufacturer's formulas and standardization of their quality processes, which must normatively guarantee 2.5 times the Coefficient of Variation (2.5Cv) or three standard deviations of the sample (3σ), for each of the resistance values they provide us, as mentioned by Sánchez (2020).

The problem of adhesion that was appreciated in the load variations remains pending, although they were comparable to those of the welded wire truss and did not affect the comparative of the test, for beams of higher load and/or clear index could be meaningful.

5. CONCLUSION

The comparative study between GFRP-based composite materials shows an acceptable behavior for use in residential buildings based on confined masonry and to some extent safe in self-construction, within the same parameters with which high-strength welded wire truss have been used as reinforcement against gravitational loads and relatively short clearings.

Bending strength results in better performance than steel within the limits of this test with high safety margins against design conditions. However, the shear strength could be noticed at a disadvantage when compared with that obtained from the drawn steel of the welded wire truss, particularly in the first beams (1, 2, and 3), whose capacity was exceeded especially because its reinforcement index was much lower than that of the steel beam (6). Likewise, it was observed that improving the amount of longitudinal reinforcement in GFRP beams also improves their shear response (beam 4).

The above suggests the shear design could be critical for use in clearings greater than 2.0m, so it requires an adequate revision and structural design, with robust transverse reinforcements, to save even clear bridges such as those mentioned in the AASHTO manual (AASHTO (GFRP), 2017).

Thanks

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