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A case study on the feasibility of using static-cast fibre-reinforced concrete electric poles fully reinforced with glass fibre reinforced polymer bars and stirrups

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ABSTRACT

This study experimentally investigates the structural performance of full-scale reinforced concrete utility poles fully reinforced with glass fibre reinforced polymer (GFRP) bars. The weak bond strength between GFRP bars and concrete in full-scale structures has been addressed by using an innovative anchorage system. Furthermore, fibre-reinforced concrete (FRC) was successfully used to enhance the crack propagations of GFRP-reinforced concrete poles. Three poles were constructed from FRC, while one was constructed with normal concrete (NC) as a benchmark. Polypropylene (PP) and crimped polyolefin-based macro-synthetic (CPO) fibres were used. An innovative anchorage system was used to increase the bond strength between longitudinal bars and the concrete. Crack initiation and pattern, load-carrying capacity and vertical deflections of GFRP reinforced FRC poles were obtained and compared with their normal concrete counterparts. According to the results, the PP-FRC pole showed better crack propagation performance, while the CPO-FRC showed higher stiffness than the normal concrete poles. In addition, regardless of the concrete type, the permanent deformation of all GFRP reinforced concrete poles after different stages of loading were significantly smaller than those of typical steel reinforced concrete poles.

1. Introduction

Fibre-reinforced polymer (FRP) composites are utilised extensively in a variety of civil and construction applications [1–6], especially for reinforcing or retrofitting/strengthening concrete structures [7–11]. Several reasons can be cited, including their unique mechanical characteristics with relatively high strength and stiffness-to-weight ratio [6, 12–15], remarkable long-term durability performance in corrosive conditions [2,16,17] and ease of manufacture, handling, and installation [17–20]. In addition, composite materials are an intriguing alternative to conventional carbon steel in corrosive conditions, such as offshore projects, due to their remarkable corrosion resistance [11,18,21]. Utilising GFRP bars in concrete structures is no longer a controversial decision; however, it is vital to have a deeper understanding of how different variables affect the behaviour of the structure in terms of cracking patterns, bonding, mechanical properties, durability, etc. [21–30].

Due to its superior mechanical strength, longer life span, capacity to cover greater distances, and electrical resistance compared to ordinary poles such as wooden and steel poles, concrete reinforced utility poles have been used widely in many countries, particularly in countries expecting earthquakes and hurricane-force winds [31]. However, in corrosive settings, such as coastal regions, corrosion of the steel reinforcement and concrete cracking can cause structural safety issues in reinforced concrete poles [32–34].

According to frequent inspections on several H-shaped steel reinforced concrete electricity poles located in the south of Iran and subject to the corrosive environment (i.e. Persian Gulf seawater), all poles experienced similar damage mechanisms. They mainly show wide longitudinal cracks on each side of the pole close to the pole base (Fig. 1).
due to the reinforcement corrosion.

The use of FRP composites as an alternative to conventional carbon steel reinforcement in concrete structures that require less ductility, such as utility poles, can enhance the possible service life of such structures resulting in significant environmental and economic advantages [1, 16, 17, 35].

The manufacture of FRP-reinforced poles began when stringent restrictions for the construction of concrete structures were implemented in response to requests for a thicker protective layer of concrete, rendering traditional poles unworkable due to their increasing size and weight. Since then, numerous reinforced concrete poles with FRP (typically CFRP) have been utilised [36–39]. For instance, Shalaby et al. [40] tested the strength, deflection, and bending resistance of spun concrete poles reinforced with CFRP bars, CFRP grids, or steel spirals. When appropriately reinforced, it was determined that these elements have good bending performance, including ductility. However, after analysing the fatigue resistance of poles reinforced with prestressed CFRP reinforcement, Robert et al. [41] concluded that the absence of adhesion between cables and concrete significantly impacts the structural reactions of such poles. An innovative anchorage technique was utilised in this work to solve this issue.

Notably, most research on FRP-reinforced concrete poles has centred on spun concrete poles reinforced with CFRP tendons. However, very few studies focused on the static-cast concrete pole reinforced with GFRP bars. Henin et al. [32] proposed a new static-cast concrete pole designed with a hexagonal hollow section (not prismatic) among those few studies. The designed poles consisted of polyvinyl alcohol (PVA) fibre-reinforced self-consolidated concrete (SCC) and longitudinally reinforced GFRP bars. The results of structural testing on two 9.14-m-tall poles indicated that their structural responses were acceptable according to the design regulation.

Under design guidelines, steel-reinforced concrete elements are typically designed to yield through ductile failure via reinforcing bars [42]. However, due to the brittle failure of FRP bars, which is identical to failure by concrete, failure via concrete is typically considered a required limit state in the design of FRP-reinforced concrete elements. Therefore, concrete behaviour and crack propagations play a vital role in-service performance of FRP-reinforced concrete members.

Due to their unique cross-section, frequent service vibrations, and the relatively small concrete cover in narrow sections of the pole, H-shaped reinforced concrete poles are prone to service cracking, and thus moisture and aggressive solution diffusion. Furthermore, in their investigation of the GFRP-reinforced concrete beams’ deflection, Mousavi and Esfahani [43] concluded that the abrupt loss of concrete stiffness is caused by the low elastic modulus of GFRP bars. Fibre rupture is the usual cause of brittle failure in beams reinforced with FRP bars [44]. According to earlier studies [45], adding discrete fibres to concrete can enhance the serviceability performance of FRP-reinforced beams and generate pseudo-ductility.

Since fibres can be employed as a smeared reinforcement, fibre-

<table>
<thead>
<tr>
<th>Code</th>
<th>Fibre type</th>
<th>Weave way</th>
<th>Moisture content (%)</th>
<th>Weight (g/m²)</th>
<th>Tensile strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Warp (N/100 mm × 200 mm)</td>
</tr>
<tr>
<td>EW160T</td>
<td>E-glass</td>
<td>Plain</td>
<td>0.2</td>
<td>160</td>
<td>1000</td>
</tr>
</tbody>
</table>

Table 1
Physical and mechanical properties of E-glass cloth.
reinforced concrete (FRC) can be used to ensure a more favourable crack distribution and to restrict crack width growth at the Serviceability Limit States (SLS) [46–48]. Wang and Belarbi [49] evaluated the deformability and crack width behaviour of different GFRP and CFRP reinforced-concrete beams under flexure. To enhance the deformability properties of FRP reinforced beams, they employed micro polypropylene fibres with a fixed fibre volume percentage of 0.5%. Chellapandian et al. [47] studied the effect of adding structural synthetic fibres on the flexural characteristics of full-scale GFRP-reinforced concrete beams. Different volumetric ratios of macro-synthetic fibres (0.35%, 0.7%, and 1.0%) were used in their study. According to the reported results, fibres considerably improved the post-cracking reaction. Furthermore, it

![Fig. 3. Anchorage system configuration: (a) Actual configuration of the anchorage system; (b) Schematic configuration of the anchorage system; (c) Tensile test setup for anchored GFRP bar.](image)

![Fig. 4. (a) PP fibres; (b) CPO fibres.](image)

<table>
<thead>
<tr>
<th>Table 2</th>
<th>Polypropylene (PP) and crimped polyolefin-based macro-synthetic (CPO) fibres properties.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Fibre length (mm)</td>
</tr>
<tr>
<td>Polypropylene fibres</td>
<td>12</td>
</tr>
<tr>
<td>Crimped polyolefin-based macro-synthetic fibres</td>
<td>40</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3</th>
<th>Physical and chemical properties of the cement.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material</td>
<td>Chemical analysis (%)</td>
</tr>
<tr>
<td>SiO₂</td>
<td>21.8</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>4.85</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>3.53</td>
</tr>
<tr>
<td>CaO</td>
<td>63.43</td>
</tr>
<tr>
<td>MgO</td>
<td>1.52</td>
</tr>
<tr>
<td>SO₃</td>
<td>2.13</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.36</td>
</tr>
<tr>
<td>K₂O</td>
<td>0.56</td>
</tr>
<tr>
<td>L.O.I</td>
<td>2.4</td>
</tr>
</tbody>
</table>
was concluded that as the dosage of fibres increases, the crack widths considerably decrease under service loads.

Since SLS is a critical factor for utility-reinforced concrete poles, using FRC seems beneficial in enhancing the serviceability and structural performance of concrete poles. However, most studies have focused on normal concrete, and no study has been done on the structural performance of fully GFRP-reinforced FRC poles.

To summarise, the available research gaps are:

(i) very limited experimental data is available on structural responses of full-scale GFRP reinforced concrete utility poles;
(ii) no study on fibre-reinforced concrete utility poles fully reinforced with GFRP bars and stirrups is available;
(iv) The weak bond strength challenge between GFRP bars and concrete in full-scale structures hinders the widespread use of GFRP bars as longitudinal reinforcement for full-scale concrete members.

Therefore, the present study investigates the structural behaviour of four full-scale H-shaped poles, three FRC poles and one normal concrete (NC) pole, fully reinforced with GFRPs (both longitudinal and stirrups), to address these research gaps.

2. Experimental program

The structural responses of GFRP-reinforced FRC and NC concrete utility poles were investigated by constructing, curing, and testing four full-scale utility poles. The next sections describe the experimental protocol in detail.

2.1. Materials

2.1.1. GFRP bars and stirrups

As longitudinal reinforcements, helically wrapped pultruded GFRP bars with nominal diameters of 14 and 16 mm were employed, while 8 mm GFRP stirrups were used as transverse reinforcements. GFRP bars and stirrups were made of E-glass-fibres and vinyl ester resin. A volumetric ratio of 70\% (fibre) to 30\% (resin) was used to combine fibres and resin. Longitudinal GFRP bars and stirrups are shown in Fig. 2. Five identical samples were tested in tension following ASTM D7205 [50] to obtain the mechanical properties of GFRP bars. The average ultimate tensile strength of 555 MPa and tensile elastic modulus of 45 GPa were obtained.

2.1.2. Anchorage system

An anchorage system made of E-glass fibre cloth and vinyl ester resin, first proposed by Ashrafi et al. [51], and recently used by Shakiba et al. [22], was employed to enhance the bond strength between the longitudinal GFRP bars and the concrete. The physical and mechanical parameters of E-glass cloth are listed in Table 1. At both ends and every 3 m, all longitudinal GFRP bars were wrapped with resin-impregnated E-glass fibres. Thus, four anchorages were employed for a 9-m-long GFRP bar, three for 5- and 6-m-long bars, and two for 3-m-long bars. Each anchorage system was 50 mm in length and 20 mm in outer diameter (together with the bar). Fig. 3 depicts the configuration of the anchorage system and the tensile test setup used to determine the anchorage system’s ultimate strength. The average ultimate tensile load of 42 kN was determined by testing three samples of the anchorage system.

2.1.3. Concretes

Three types of concrete mixes were used in this study: (i) polypropylene fibre reinforced concrete (PP-FRC); (ii) crimped polyolefin-based macro-synthetic fibre reinforced concrete (CPO-FRC); (iii) normal concrete (NC). Fig. 4 shows PP and CPO fibres used in type (i) and (ii) concrete mixes. Table 2 summarises the mechanical properties of PP and CPO fibres.

All concretes were prepared using general-purpose Type II Portland cement, with a specific surface of 3050 cm²/g. The chemical composition of the cement used is detailed in Table 3. The nominal maximum size of natural river gravel used was 19 mm. Concretes were mixed following ACI-211 [52] guidelines. After 28 days, three standard cubic and three standard cylinder samples were selected from each batch and tested in compression. Table 4 displays the concrete’s mixed composition and compressive strength at 28 days.

2.1.4. Fabrication and transfer process

Commercial utility poles used are typically reinforced with carbon steel. Therefore, concrete poles used in this study were reinforced with GFRP bars and stirrups similar to steel bars and stirrups (i.e. same reinforcement numbers and arrangements). Utility poles are designed with a non-uniform H-shaped cross-section following Iran Electric Power sector regulations. All poles were reinforced with longitudinal 14 and 16 mm GFRP bars (4ф14 & 6ф16) and 8 mm GFRP stirrups. A GFRP-reinforced utility pole and its cross-sections are depicted schematically in Fig. 5. It should be noted that along the H-shaped cross-section length, GFRP stirrups were cut from the centre and placed back-to-back as shown in Fig. 5. Fig. 6 also shows the stirrups configuration and arrangements as well as the locations of the holes inside the H-shaped poles.

The design specifications for the commercial poles used in this study are listed in Table 5. Fig. 7 illustrates the longitudinal reinforcement configuration of the poles listed in Table 5.

The properties and configuration of GFRP stirrups along the pole height are shown in Table 6 and Fig. 8.

Fig. 9 depicts the production process for constructing four 9-m reinforced concrete utility poles. Steam curing was conducted in this study. Fig. 10 shows the steam curing process. After concrete pouring, all poles were sealed with plastic sheets to avoid heat and moisture transfer between the poles and the environment. Poles were kept inside the moulds for 5 h (around 3 MPa compressive strength). Then, poles were subjected to 90\% humidity water vapour with an increasing temperature rate of 20° C per hour up to the maximum temperature of 60° C for 2 h. Then, poles were kept in constant 60° C water vapour and a minimum of 90\% humidity for 15 h. Finally, poles were subjected to a decreasing temperature rate of 15° C per hour up to the ambient temperature and then the plastic sheets were removed. After steam curing, poles were immersed in tap water for four days and cured in an ambient environment for up to 28 days of concrete casting. After 28 days, all four poles were tested. It is important to note that due to the extremely low weight of GFRP cages, only two people were required to transport them (Fig. 9 (a)). Using a gantry crane shown in Fig. 9 (f), poles were moved for curing and testing purposes. Fig. 11 schematically shows the locations where the gantry crane grabbed the poles.
Fig. 5. Utility poles reinforced with GFRP bars were used in this study.
2.2. Test setup

A horizontal test method was employed to test the poles. Each pole was positioned between two sets of 1250 mm-long wooden supports to provide fixed support. Three hydraulic jacks were positioned between the wood plate and the concrete block. A winch was used to apply a pulling force 600 mm from the free end of the pole. A load cell with a capacity of 100 kN was installed in the chain line of the winch. The force was applied at a modest rate with regular load holds to get readings of displacements. Three linear variable differential transformers (LVDTs) were positioned at 3 m, 6 m, and 9 m from the base of the pole to record the beam’s vertical deflections during the test. The free-end displacement was also measured using a ruler. The design and actual test setup utilised in this investigation are depicted in Fig. 12.

3. Results and discussions

According to Fig. 13, poles were loaded in three stages following the Iran Electric Power Industry standard (1998). A pole must meet all criteria for each level to be regarded as commercially acceptable. According to the regulation, poles in stage one must have no apparent cracks. During stage two, when the maximum load is 1.5 times that of stage one (i.e. 6 kN for the designed poles in this study), poles must not have significant cracking or base failure. In addition, the maximum residual vertical deflection of the free end shall be less than 10% of the maximum deflection the pole experienced during stage 2. Lastly, at stage three of loading, poles shall not fail before bearing 2.5 times the

---

Table 5
Design regulations of the reinforced concrete poles according to Iran Electric Power industry regulation.

<table>
<thead>
<tr>
<th>Design regulation</th>
<th>Regulation</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Pole length</td>
<td>mm</td>
<td>9000</td>
</tr>
<tr>
<td>2</td>
<td>Nominal capacity</td>
<td>kN</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>Free-end dimension</td>
<td>mm</td>
<td>190 × 220</td>
</tr>
<tr>
<td>4</td>
<td>Base dimension</td>
<td>mm</td>
<td>400 × 280</td>
</tr>
<tr>
<td>5</td>
<td>Concrete volume</td>
<td>m³</td>
<td>0.48</td>
</tr>
<tr>
<td>6</td>
<td>Longitudinal bars configuration</td>
<td>number &amp; mm</td>
<td>4 ∅14 &amp; 6 ∅16</td>
</tr>
<tr>
<td>7</td>
<td>Longitudinal bars arrangement</td>
<td>–</td>
<td>See Fig. 7</td>
</tr>
<tr>
<td>8</td>
<td>a in Fig. 7</td>
<td>Size(mm)-length (mm)</td>
<td>∅16-L9000</td>
</tr>
<tr>
<td>9</td>
<td>b in Fig. 7</td>
<td>Size(mm)-length (mm)</td>
<td>∅14-L6000</td>
</tr>
<tr>
<td>10</td>
<td>c in Fig. 7</td>
<td>Size(mm)-length (mm)</td>
<td>∅16-L5000</td>
</tr>
<tr>
<td>11</td>
<td>d in Fig. 7</td>
<td>Size(mm)-length (mm)</td>
<td>∅14-L3000</td>
</tr>
</tbody>
</table>

Fig. 6. Holes locations and stirrups arrangements.

Fig. 7. Longitudinal reinforcement arrangement of GFRP reinforced poles according to Iran Electric Power industry regulation.
maximum load during stage 1 (10 kN for the designed poles in this study); i.e. the ultimate load-carrying capacity of the pole must be larger than 2.5 the maximum load of stage one. It is worth mentioning that due to the large deflection of GFRP reinforced poles (low stiffness of GFRP bars) and loading chain length limitation, poles were not tested until collapse. However, they loaded at least up to the required 10 kN.

Tables 7 and 8 summarise the pole testing results in crack formation and pole deflections for each step. Each pole’s crack patterns and load-deflection curves are presented and discussed in the following sections.

3.1. Cracking pattern

3.1.1. Normal concrete pole (pole number 1)

During pole transportation, six micro-cracks were detected in pole number 1. The first crack developed with the applied stress of 1.94 kN in the tensile zone 4.52 m from the pole’s base. As the load was increased to 4 kN (the end of stage 1), the number of cracks increased to 8. In stage two of loading, the number of cracks increased to 14 upon reaching 6 kN. Finally, by increasing the applied stress during stage three, the number of cracks grew to 24, with crack number 2 exhibiting the greatest width of 5 mm. Fig. 14 depicts the initial crack initiated at 1.94 kN and other cracks discovered at the tested pole’s support. Fig. 15 also depicts all observed crack patterns during the testing of pole 1 in schematic form.

3.1.2. CPO-FRC poles (poles number 2 and 3)

The second pole (CPO-FRC-1) was constructed from CPO fibres (for each sample, 1.25 kg of CPO fibres were mixed with concrete). Pole two has exhibited its first crack at a distance of 4.05 m from the pole’s base and at a weight of only 2.15 kN. Similar to pole 1, this crack may have resulted from the nine micro-cracks that appeared during the pole’s movements before testing. At the end of stage 1 loading (4 kN), four cracks were discovered, half the number of cracks observed in the NC concrete pole. Similar to pole number one, the number of cracks rose throughout stage 2 of loading and reached 11 at 6 kN, and they all closed during the unloading stage from 6 to 4 kN. During stage 3, the number of cracks increased to 15, with the widest crack measuring 4 mm and located 0.9 m from the pole’s base. Fig. 16 shows the initiation of the first crack at 2.15 kN as well as subsequent cracks seen at the support. Fig. 17 also depicts the first crack initiated at 1.94 kN and other cracks discovered at the tested pole’s support. Fig. 15 also depicts all observed crack patterns during the testing of pole 1 in schematic form.

3.1.3. PP-FRC pole (pole 4)

The fourth pole, made from PP-FRC, exhibited first cracking when subjected to a force of 2.7 kN, 4.72 m from its base. During the transportation of pole 4, nine micro-cracks were observed. The number of cracks grew to seven at the end of stage one. During stage two, just one further crack was identified, and all cracks were closed from 6 kN to 4 kN during unloading. At the end of stage 3, a total of 19 cracks were discovered, with the widest crack measuring 6 mm and located 3.77 m from the pole’s base. Fig. 20 depicts the onset of the first crack at 2.70 kN and numerous cracks seen at the support. Schematically, Fig. 21 depicts every crack pattern detected during the testing of pole 4.

Comparing the number of cracks detected during stages 1 and 2 of loading (SLS) between the NC pole and FRC poles, one can conclude that in both stages, the number of cracks is lower for FRP compared to NC poles. This agrees with the results reported in the literature [23] on the efficiency of using fibres in concrete structures. Fibres will stop/delay crack propagation, and consequently, fewer cracks propagate to the surface of the poles (visible cracks). The initial stretching of the fibres, followed by their rupture or withdrawal, results in the resistance that fibres offer in terms of crack bridging [47]. Therefore, for FRP reinforced concrete members compared to conventional steel reinforced concrete members, the current design standards often permit a wider crack width [53,54].

In addition, comparing the cracking performance of the pole reinforced with PP fibres with those reinforced with CPO fibres shows almost the same number of cracks for both stages 1 and 2. This shows that adding fibres will enhance the SLS performance of GFRP-reinforced concrete poles regardless of the fibres type. According to the literature, the fibre distribution has a significant impact on how reinforced concrete members respond to the propagation crack and its widening. The amount of fibres and their spacing throughout the failure section depend on the area, orientation, and dosage of the fibres. On the other hand, the addition of fibre decreases the workability of concrete, and thus using eight micro-cracks were identified during pole transportation, and these micro-cracks were the primary cause of crack initiation at relatively low applied loads (i.e. 2.10 kN). Seven cracks were discovered in total at the end of stage 1 loading. During stage two, the number of cracks only increased to ten. Similar to poles 1 and 2, all cracks were closed during the 6 kN–4 kN unloading cycle. After the completion of stage 3 loading, a total of 25 cracks were found. Maximum crack width of 3 mm for crack number 25 at a distance of 0.33 m from the base of the pole was recorded. Fig. 18 shows the initiation of the first crack at 2.10 kN, as well as further cracks seen at the support. Fig. 19 is a schematic representation of all crack patterns detected during the testing of pole 3.
Fig. 8. GFRP stirrups configurations.
Fig. 9. Production process: (a) Setting up the GFRP reinforcement cage; (b) Placement of reinforcement cage inside the framework; (c) Placement of steel rods for creating holes in poles; (d) Concrete pouring using tower crane; (e) Standard compressive samples; (f) Concrete curing (immersion stage).

Fig. 10. Steam curing process (a) Plastic sheet covering; (b) Water vapour exposure.
Fig. 11. Locations of grabbing the pole by the crane.

Fig. 12. Cantilever beam test setup.

1: Concrete block  7: Load cell
2: Three hydraulic jacks  8: Chain
3: Timber bearing plates  9: Tirfor
4: Concrete pole  10: Roller plates
5: Measurement benchmark
6: Ruler
Fig. 13. Loading protocol used.

Table 7
Crack initiation and number at different stages.

<table>
<thead>
<tr>
<th>Pole number</th>
<th>Pole ID</th>
<th>Load at the first crack (kN)</th>
<th>Total crack in stage 1</th>
<th>Total crack in stage 2</th>
<th>Total crack up to 10 kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NC</td>
<td>1.94</td>
<td>8</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>CPO-FRC-1</td>
<td>2.15</td>
<td>4</td>
<td>11</td>
<td>12</td>
</tr>
<tr>
<td>3</td>
<td>CPO-FRC-2</td>
<td>2.10</td>
<td>7</td>
<td>10</td>
<td>19</td>
</tr>
<tr>
<td>4</td>
<td>PP-FRC</td>
<td>2.70</td>
<td>7</td>
<td>8</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 8
Maximum free end deflection and permanent deflection of tested poles at the end of each testing stage.

<table>
<thead>
<tr>
<th>#</th>
<th>Pole</th>
<th>Reinforcement type</th>
<th>Stage one free end deflection (4 kN)</th>
<th>Stage one permanent deflection</th>
<th>Stage two free end deflection (6 kN)</th>
<th>Stage two permanent deflection</th>
<th>Maximum free end deflection at 10 kN</th>
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</thead>
<tbody>
<tr>
<td>1</td>
<td>NC</td>
<td>350 mm</td>
<td>15 mm</td>
<td>530 mm</td>
<td>35 mm</td>
<td>840 mm</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>CPO-FRC-1</td>
<td>325 mm</td>
<td>22 mm</td>
<td>475 mm</td>
<td>30 mm</td>
<td>820 mm</td>
<td></td>
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<tr>
<td>3</td>
<td>CPO-FRC-2</td>
<td>295 mm</td>
<td>25 mm</td>
<td>455 mm</td>
<td>37 mm</td>
<td>805 mm</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>PP-FRC</td>
<td>425 mm</td>
<td>35 mm</td>
<td>595 mm</td>
<td>45 mm</td>
<td>910 mm</td>
<td></td>
</tr>
</tbody>
</table>

Fig. 14. First crack initiation and support cracks of pole 1.

Fig. 15. Cracks pattern of pole 1.
Fig. 16. First crack initiation and support cracks of pole 2.

Fig. 17. Cracks pattern of pole 2.

Fig. 18. First crack initiation and support cracks of pole 3.

Fig. 19. Cracks pattern of pole 3.

Fig. 20. First crack initiation and support cracks of pole 4.
fibres with a high-volume fraction may result in issues including segregation, fibre balling, and honeycombing [47]. In this study, only two fibre volume fractions (i.e., 2 and 2.6 kg/m3) were used, and the difference between the post-crack and workability performances was insignificant. According to Chellapandian et al. [47], 1% fibre volume fraction was found to be appropriate to avoid workability issues. The uniform fibre distribution across the failure surface of the reinforced concrete beam was also observed using such a ratio. Therefore, for future studies on GFRP reinforced concrete poles, it is recommended to use different discrete fibre volume fractions in order to obtain the optimised value for achieving the appropriate post-crack behaviour while keeping the concrete mix workable.

3.2. Load-deflection

According to Iran Electric Power Industry regulations, as noted previously, the maximum residual free end vertical deflection must be less than 10% of the greatest deflection the pole experienced during stage 2. In light of this criterion, all four examined poles met this requirement. For instance, the permanent deflection of pole number two at the end of stage two was 30 mm, which was less than 10% of the pole’s highest free-end deflection at stage two. (27 mm > 0.1 × 475 = 47.5 mm). However, it must be mentioned that in practice, as opposed to a fixed-end condition, the pole foundation compacts the soil around the base to a depth of around 2 m. Because of this, applying lateral stresses on real poles may cause them to rotate or tilt more than the tested pole did, which could influence their deformed shape [39].

Fig. 21. Cracks pattern of pole 4.

Fig. 22 depicts the load-free end deflections of all poles examined. It should be noted that the unloading stages were not shown in Fig. 12. It should be noted that recording after each stage starts from the end of the previous stage. For example, if the free end deflection of a pole was recorded as 200 mm at the end of loading stage 1 (4 kN), the recording of stage 2 starts at 200 mm. Load-deflection curves, therefore, show one continuous line for all stages of loadings.

All evaluated poles, except CPO-FRC-2, had nearly linear performance over the stress regime. CPO-FRC-2 also had a linear behaviour of up to 12 kN, which was above the required load of the testing protocol. The continuing energy dissipation throughout the fracture process to overcome the bond strength while the fibres are being dragged out of the concrete matrix causes the pseudo-ductility of CPO-FRC-2 to increase [47].

To compare the performance of fully GFRP reinforced poles with typical steel reinforced poles, Fig. 23 depicts the load-deflection of the tested poles in this study with corresponding steel reinforced poles designed for the same load and regulations. As expected and shown in Fig. 23, due to the significantly lower stiffness of GFRP bars compared to steel bars, steel-reinforced poles exhibit significantly greater stiffness than GFRP-reinforced poles. The free end deflections of poles reinforced with longitudinal GFRP bars are much greater than those reinforced with steel bars. However, the permanent deflections of poles reinforced with GFRP bars are substantially less than those reinforced with steel bars (e.g., recorded as 150 mm for steel reinforced pole at the end of stage three loading). This is due to the material performance since GFRP exhibits a linear behaviour until failure. In contrast, steel bars yield at a specific axial load and undergo severe nonlinear deformations before failure. Low permanent deflection in utility poles is of utmost importance; therefore, adding GFRP bars to reinforced concrete utility poles could be quite advantageous.

3.3. Ductility evaluation

It is important to establish a new approach for assessing the ductility performance of FRP reinforced members because the traditional definition of ductility cannot be directly applied to FRP reinforcement concrete structures [55]. Over the past two decades, there has been a lot of studies and discussion put into this topic. Two primary methods have thus been used widely: energy-based approach and deformation-based approach. Energy-based approach has been used in the present study to evaluate the ductility of different concrete poles tested.

In an energy-based approach, ductility can be defined as the ability to absorb energy and is quantified as the proportion of elastic energy to total energy. Naaman and Jeong [56] first proposed ductility index, $\mu_E$, to be computed from Eq. (1):

$$\mu_E = 0.5 \left( \frac{E_e}{E_t} + 1 \right)$$

where $E_t$ is the total dissipated energy equivalent to the area under the load-deflection curve and $E_e$ is the elastic dissipated energy computed as the area beneath the un-loading elastic line up to the point of intersection with the maximum load (or failure load or the point correspond to 80% of the maximum load on the descending branch of the curve). $\mu_E$ in this study has been calculated for three different stages of loading: (1) stage 1 (fully elastic) up to 4 kN; (2) Stage to at 6 kN; and (3) End of the test of each sample. Table 9 presents the results of energy dissipated and ductility index using energy-based approach. From Table 9, using the ductility index values, one can conclude that all tested poles mainly remained in elastic mode up to the end of loading (about 14 kN). In addition, in all stages, FRC-CPO poles showed higher ductility compared to NC, while FRC-PP showed almost the same. It should be noted that the fibres contribution in ductility becomes more significant after significant cracking of the member, which was not the case in the present student. Further, similar to the present study, Wang and Belarbi [49] also reported that the ductility index of plain concrete and FRC beams computed based on the energy-based method did not show a significant difference. The reason was stated to be the proportional increase of elastic and inelastic energy dissipation, which results the same ratios $E_e/E_t$ ratio and consequently a constant $\mu_E$. 

![Cracks pattern of pole 4.](image-url)
Fig. 22. Load-free end deflections.
4. Conclusions

Static cantilever beam tests were conducted on four full-scale, 9000 mm-long normal and fibre-reinforced concrete utility poles reinforced with longitudinal GFRP bars and GFRP stirrups. The following conclusions are formed based on the test outcomes:

- GFRP reinforced concrete poles, regardless of concrete type, satisfy the load and deflection regulations of commercial H-shaped concrete reinforced utility poles.
- Appropriate GFRP reinforced utility pole movement is an important factor: Initial cracking is detected at relatively small loads during the SLS loading stage due to the micro-cracks developed when transferring the poles.
- During serviceability limit state loading stages, fibre-reinforced concrete poles generally showed less number of cracks than that of normal concrete poles. This is because fibres stop/delay the developed cracks’ propagation toward the pole surface and result in fewer visible cracks.
- In all loading stages, FRC-CPO poles showed higher ductility compared to NC, while FRC-PP showed almost the same. However, because of the proportional increase of elastic and total energy dissipation for the range of the applied load, the differences were not significant.
- GFRP reinforced concrete utility poles are found to be viable alternatives to steel reinforced concrete poles in corrosive environments if significant considerations are given to their stiffness, transportation and instalment.

Ethics approval statement

Not applicable, because this article does not contain any studies with human or animal subjects.

Credit author statement

Milad Shakiba: Conceptualization, Methodology, Investigation, Formal analysis, Roles/Writing – Review and Editing. Hassan Ahmadi: Conceptualization, Methodology, Investigation, Formal analysis, Roles/Writing – Review and Editing. Seyed Mohammad Reza Mortazavi: Conceptualization, Methodology, Investigation, Formal analysis, Supervision, Roles/Writing – Review and Editing. Milad Bazli: Conceptualization, Methodology, Investigation, Formal analysis, Supervision, Roles/Writing - original draft. Zahir Azimi: Formal analysis, Roles/Writing – Review and Editing.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

Data will be made available on request.

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References


Table 9

<table>
<thead>
<tr>
<th>Pole type</th>
<th>Stage 1</th>
<th>Stage 2</th>
<th>Stage 3</th>
</tr>
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<tr>
<td></td>
<td>$E_t$ (kN.mm)</td>
<td>$E_e$ (kN.mm)</td>
<td>$\mu E_t$</td>
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<tr>
<td>NC</td>
<td>617.10</td>
<td>670.10</td>
<td>1.00</td>
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<tr>
<td>FRC-CPO-1</td>
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<td>FRC-CPO-2</td>
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<td>627.72</td>
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<tr>
<td>FRC-PP</td>
<td>783.75</td>
<td>801.70</td>
<td>1.01</td>
</tr>
</tbody>
</table>

Fig. 23. Comparison between load-free end deflection curves of poles tested in this study with a typical steel reinforced concrete pole.


[34] ACI Committee 211.1-91, Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete. ACI Manual of Concrete Practice, Part 1, American Concrete Institute, Michigan (USA), 2000, p. 38.


