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BEHAVIOUR OF PULTRUDED GLASS FIBRE-REINFORCED POLYMER UTILITY POLES UNDER LATERAL LOADS

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Abstract: This paper discusses experimentally and numerically the behaviour of pultruded Glass Fibre-Reinforced Polymer (GFRP) utility poles under lateral loads. Electrical and telecommunication utility infrastructures, including poles, H-frames, and towers, are typically made of wood, concrete, or steel. Each of these materials has several shortcomings due to their performances under the environmental conditions and the difficulty of transportation in rough terrain. In addition, a significant number of the utility infrastructures in North America needs renewal in the coming few years because of their environmental deterioration. Currently, the governments tend to build new sustainable and cost-effective utility infrastructures. GFRP composites represent a viable alternative to the traditional materials (i.e., wood and steel) for the utility infrastructures. However, the lack in the theoretical and experimental data on the GFRP composite utility infrastructures delays their implementation. In this study, a full-scale GFRP distribution pole was tested under lateral load until failure. Then, using LS-DYNA software, a finite element (FE) parametric study was conducted to investigate the effects of the parameters: diameter, length, and wall thickness of the pole. The tested pole had a height of 10.5 m, a diameter of 254 mm, and a wall thickness of 6.35 mm. The developed FE model was validated with the experimental results which showed a good agreement with an accuracy of > 94%. The FE parametric study included pole diameters ranged from 254 mm to 457.2 mm and wall thicknesses ranged from 3.175 mm to 12.70 mm. The FE analyses showed that the maximum moment of the GFRP pole highly increased with increasing the diameter or the wall thickness. The common mode of failure of the GFRP poles is local buckling, but it could be changed to rupture in tension when the wall thickness is high.

1 INTRODUCTION

The electrical and telecommunication infrastructures (i.e., towers, poles, and H-frames) are commonly made of steel, concrete, or wood materials. There are several shortcomings of these traditional materials especially regarding their resistance under the environmental conditions. Wooden utility poles are the most common utility infrastructures in North America, but they are vulnerable to the environmental effects such as insects, leaching, rot, woodpeckers, and fire. They must be treated frequently with toxic chemicals to keep them in service. The concrete poles are more durable than the wooden poles, but the main disadvantage of them is their heavy weight which increases the transportation and erection costs, especially in the case of rough terrains. Also, the corrosion of steel reinforcements is one of the main drawbacks of the concrete poles. Although the steel poles are much lighter than the concrete poles, they are very expensive comparable to the concrete and wooden poles. In addition, they must be painted or galvanized for corrosion protection, while it is not guaranteed for a long-term protection.

Numerous electrical and telecommunication infrastructures in Canada and the United States are at the end of their life expectancy because of their deterioration under the environmental effects. It was reported by the Conference Board of Canada that in order to maintain the existing capacity of electricity in Canada, approximately $350 billion dollars will be required by 2030 (Coad et al. 2010). Consequently, it is urgent to use more durable and sustainable materials for the utility infrastructures.

Fibre reinforced polymer (FRP) composites, especially the type of glass, have been used in civil engineering structures repeatedly, and their usage continues to grow at an impressive rate because of their characteristics in terms of low weight-to-strength ratio and high durability under severe environmental conditions (e.g., Abdelkarim et al. 2019 and 2016; Abdelkarim and ElGawady 2015; Benmokrane et al. 2006). Some theoretical and experimental studies were conducted to examine the behaviour of the FRP utility infrastructures, especially the GFRP poles (McClure et al. 1992; Polyzois et al. 1999; Ibrahim et al. 2000; Metiche and Masmoudi 2007; Mohamed and Masmoudi 2009; Hernández-Corona and Ramírez-Vázquez 2015). But the previous studies investigated the behaviour of the GFRP fpoles that are manufactured using filament winding. The manufacturing technique controls the direction of fibres and the quality of the product, so it is expected to have notable effects on the behaviour of these structures. In filament winding manufacturing technique, the fibres are laying over a rotating mandrel in the desired angle, commonly ±45º-55º. On the other hand, in the pultrusion manufacturing technique, the fibres enter into a mold producing poles with fibres in the longitudinal direction. The pultrusion technique introduces stronger products in the longitudinal direction which is more suitable and economically efficient for the electrical infrastructures because they are mainly subjected to wind loads. Therefore, the pultruded GFRP poles are likely to meet the deflection requirement by the American Society of Civil Engineering (ASCE) manual of practice-104 rather than that are made of filament winding.

2 EXPERIMENTAL WORK

The utility poles are categorized into fourteen classes based on the lateral load capacity, according to the American National Standards Institute (ANSI) 05.1. A full-scale GFRP pole was manufactured by the Global PoleTrusion Group Corporation using pultrusion technique and was prepared for testing. The pole had a diameter of 254 mm, a thickness of 6.35 mm, and a height of 10.5 m. The pole was prepared for Class 5. Three coupons were cut from an extra part of the pole and tested under tension loading according to the American Society for Testing and Materials (ASTM) D3039. The displacement loading rate was 1 mm/min. The average tensile elastic modulus and strength of the coupon specimens were 32.3 GPa and 510 MPa. The poles were tested under flexural loading according to the ASTM D4923-01 at the University of Sherbrooke to identify the horizontal load capacity. Figure 1 illustrates the general configuration of the tested pole and the test setup. The pole was placed horizontally on the floor and fixed using two opposite fixed supports at 0.0 and 1.65 m from the first end. The fixed part of the pole (1.65 m) represented the embedded depth of the pole into the ground which is commonly calculated as (10% of the pole length + 600 mm). Two string potentiometers were installed at the two supports to confirm the fixation by monitoring the movement, if any. The pole was subjected to vertical loading at 600 mm from the second end using winch and steel rope, as shown in Figure 1. Another string potentiometer was attached to the pole at the loading point to measure the displacement. Figure 2 illustrates the pole during the load application.

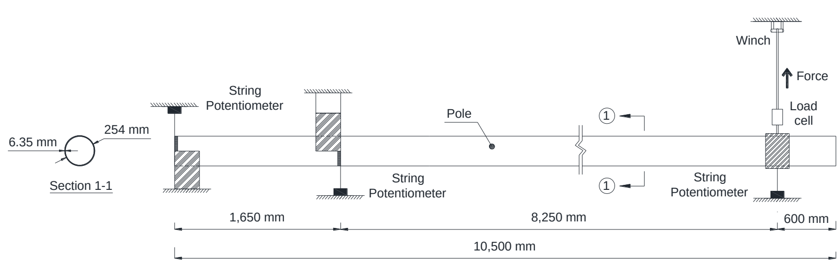

Figure 1: Pole configuration and test setup

Figure 2: Load application

3 EXPERIMENTAL RESULTS

Figures 3 and 4 illustrate, respectively, the displacement-load relation and the moment-drift relation of the tested pole. The load increased almost linearly with increasing the displacement until the failure of the pole. The maximum load and moment of the tested pole were 9.4 kN and 77.6 kN.m, respectively. The maximum displacement and drift (displacement over the shear span) at the point of load application were 1.92 m and 23%, respectively. The pole failed by local buckling as shown in Figure 5. The standard required load capacity of a pole for Class 5, according to the ANSI 05.1, is 8.45 kN. The load capacity of the tested pole was approximately 11% higher than the required strength. So, the design of the pole can be adjusted by lowering the fibre volume ratio or the wall thickness to achieve a load capacity closer to the required strength.

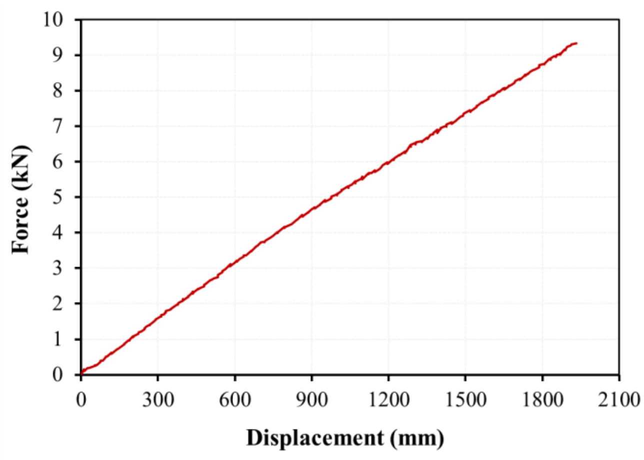

Figure 3: Displacement-force relation of the tested pole

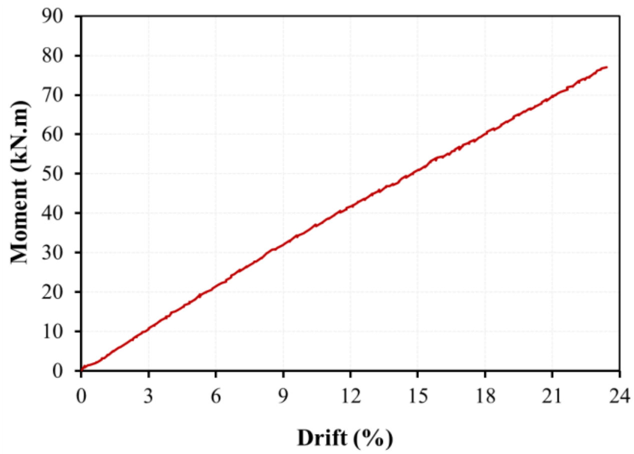

Figure 4: Moment-drift relation of the tested pole

Figure 5: Local buckling failure of the tested pole

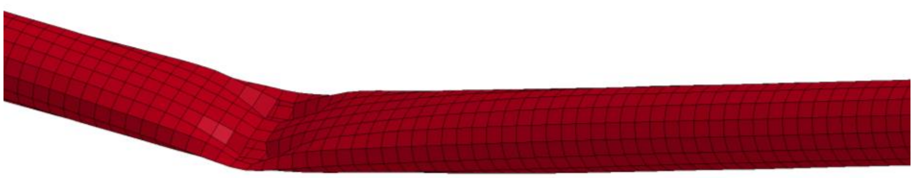

Figure 6: Local buckling failure of the pole in the finite element analysis

4 FINITE ELEMENT ANALYSIS

4.1 Model Validation

Finite element (FE) model was developed to simulate the tested pole to be used for a parametric study. The analysis was conducted using the LS-DYNA software. Fully integrated shell elements were used in modelling the pole. Material Mat_022 Composite_Damage was used to simulate the GFRP composite material. The material properties used in this model was collected from the results of the coupons' tests. The simulated pole was subjected to a ramp up lateral load at 600 mm from the top end until the pole. The pole failed numerically by local buckling as the experimental test at almost the same position from the support (see Figure 6). Figure 7 shows the moment-drift relation of the numerical versus the experimental.

The moment-drift relation from the FE results showed good agreement with the experimental results with accuracy of 94% and 97.4% for the maximum moment and drift, respectively.

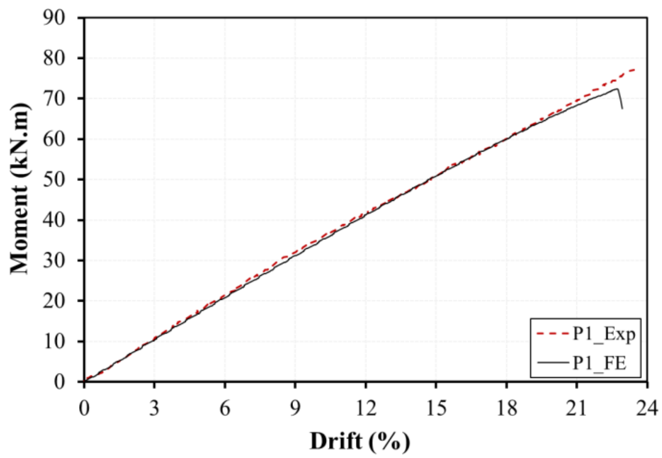

Figure 7: Finite element versus experimental moment-drift relation of the pole

4.2 Parametric Study

The validated model was used as a base for the parametric study in order to investigating the effects of changing the diameter and wall thickness on the maximum moment and maximum drift of the pole. The study included examining: three diameters: 254 mm, 355.6 mm, and 457.2 mm; four wall thicknesses: 3.175 mm, 6.35 mm, 9.525 mm, and 12.70 mm. The wall thickness was kept constant as 6.35 mm during the study of the diameter's effect. Also, the diameter was kept constant as 254 mm during the study of the wall thickness's effect. Figures 8 and 9 illustrate, respectively, the maximum moment and maximum drift of the pole with different diameters. Figure 8 shows that the maximum moment increased by 115.7% when the diameter increased from 254 mm to 457.2 mm. Figure 9 shows that the maximum drift decreased by 61% when the diameter increased from 254 mm to 457.2 mm. Figures 10 and 11 illustrate, respectively, the maximum moment and maximum drift of the pole with different wall thicknesses. Figure 10 shows that the maximum moment of the pole with a wall thickness 12.7 mm was approximately 8 times that of the pole with a wall thickness of 3.175 mm. Figure 11 shows that the maximum drift of the pole with a wall thickness of 9.525 mm was approximately 1.33 times that of the pole with a wall thickness of 3.175 mm. The maximum drift slightly decreased when the wall thickness was higher than 9.525 mm. The reason of that was because the mode of failure changed from local buckling to rupture in tension by reaching the tensile strength.

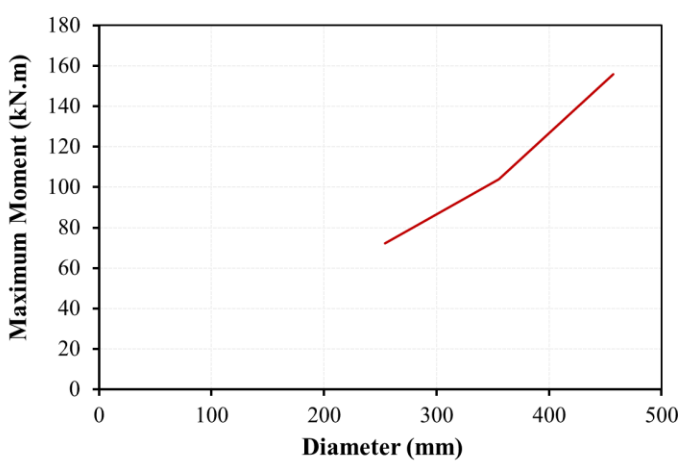

Figure 8: Maximum moment with different pole diameters

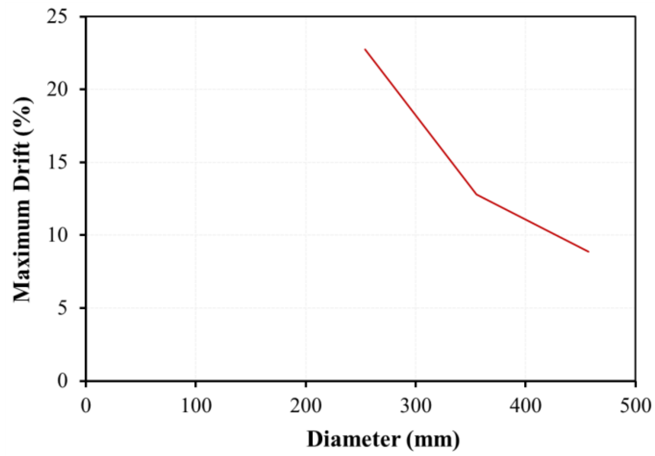

Figure 9: Maximum drift with different pole diameters

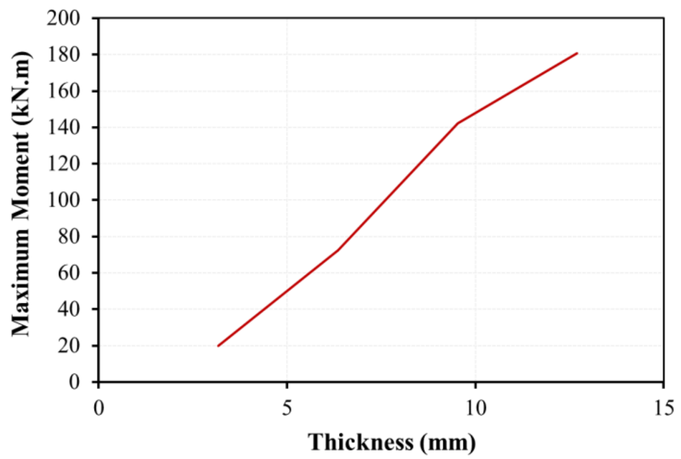

Figure 10: Maximum moment with different pole wall thicknesses

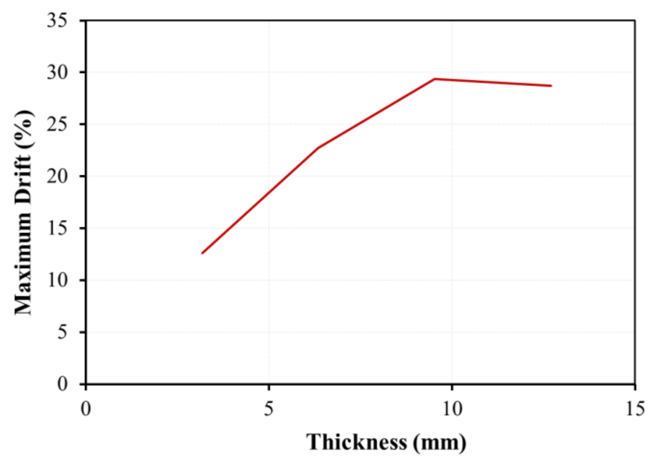

Figure 11: Maximum drift with different pole wall thicknesses

5 Conclusions

A GFRP pultruded pole was examined experimentally and numerically under lateral loading. A validated FE model was used in a parametric study to examine the effects of changing the pole diameter and wall thickness. The study showed that changing the diameter and wall thickness highly affects the maximum moment and maximum drift of the pole. The maximum moment increased by 115.7% and the maximum drift decreased by 61% when the diameter increased from 254 mm to 457.2 mm. The maximum moment of the pole with a wall thickness 12.7 mm was approximately 8 times that of the pole with a wall thickness of 3.175 mm, and the maximum drift increased until the wall thickness of 9.525 mm, then slightly decreased.

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