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Design of FRP Pole Structures for Transmission Lines in British Columbia

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SUMMARY

BC Hydro has been increasingly using Fibre-Reinforced Polymer (FRP) pole structures in its 69kV – 287kV overhead transmission lines since 2011 due to their immunity to woodpecker attack, ease of shipping, environmental friendliness, etc. This paper aims to summarize the design practice and experience from the perspective of BC Hydro. Following important issues will be covered.

Firstly, basic design methodology is described specific for the FRP pole structure design as per the BC Hydro standard ES41B-0300R0 "Design Criteria of Overhead Transmission Lines" that established the RBD methodology applicable to entire transmission line systems.

One of the most important design criteria promoted by BC Hydro from beginning is the deflection limits. FRP poles can undergo a very large deflection without mechanical failure so that they may behave like a "fishing rod" if no deflection limit is imposed. An overly flexible structure could significantly reduce ground clearance, demand a much wider right of way, and compromise the maintainability of the structure by the field crew. Setting deflection limits has proved to be an important contributor to the many successful applications in BC Hydro. Accordingly, a proper unequal ice load case plays an important role in designing FRP pole structures, as it is often the governing case for the deflection check.

The connection design is another important issue. FRP poles are essentially tapered thin tubes. The great diameter-to-thickness ratios and small modulus of elasticity dictate that the strength of an FRP module is mostly governed by the local buckling. Based on limited full-scale test results, a semi-empirical method is presented in this paper for designing typical connections associated with FRP poles. The method takes into account the interactions among local bucking, global strength, as well as local bearing strength, and the predicted connection strength correlates reasonably with the observed full-scale test data.

KEYWORDS

Connection, Deflection, Fibre-reinforced polymer (FRP), Load, Overhead line, Pole, Reliability, Strength, Transmission line.

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INTRODUCTION

BC Hydro (BCH) has been increasingly using Fibre-Reinforced Polymer (FRP) pole structures in its 69kV – 287kV overhead transmission lines (OHTLs) since 2011 (Figure 1) (Gilpin-Jackson et al. 2015). FRP poles offer the following advantages: (a) they are fairly inert to chemical attack so that they are ideal for applications in wetlands or any other environmentally sensitive areas or corrosive areas; (b) they come in modules and are very light so that they are ideal for applications in remote areas with limited access; (c) they are immune to woodpecker attack; (d) they are versatile for adjusting pole heights up to about 150ft so that they are suitable for crossing applications; (e) they tend to perform very well in withstanding wild fires; and (f) they tend to serve longer with a typical design life time of 70 years (RS 2011).

Over the years, BCH has been working closely with the FRP manufacturer RS Technologies, and has developed a systematic methodology for designing various FRP pole structures to support overhead transmission lines in various conditions. This paper aims to summarize the design practice and experience from the perspective of BCH, and will cover the important issues of basic design methodology, unequal ice load, deflection limit, and connection design.

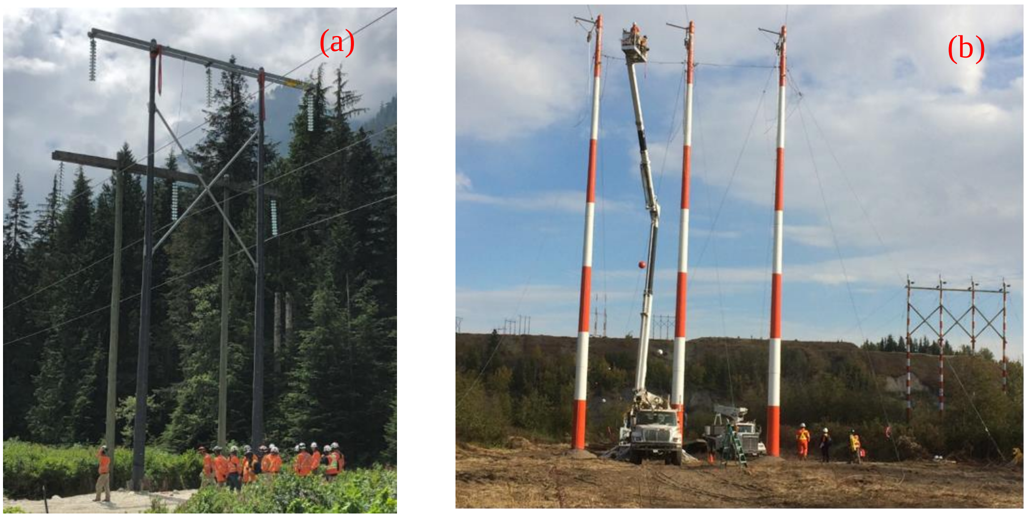

Figure 1. Installation of **(a)** a 230kV H frame; and **(b)** a 138kV deadend, both FRP structures.

DESIGN METHODOLOGY

BC Hydro has formally adopted the reliability based design (RBD) methodology with the official release of the initial version of the BC Hydro Wood Pole Structure Design Standard in 2008 (BCH 2008). The same methodology has been applied to FRP pole structures with success since the first application of FRP poles in BC Hydro in 2011. In 2019, BC Hydro issued a new standard "Design Criteria of Overhead Transmission Lines" that established the RBD methodology applicable to entire transmission line systems including FRP pole structures (BCH 2019). The BC Hydro RBD methodology is consistent with the CSA C22.3 No.1-60826 standard in principle, with a number of deviations to reflect BC Hydro's own practice. The BCH design methodology is summarized next.

Performance Criteria

All transmission structures, FRP included, shall be designed to meet the requirements of reliability, security, and safety.

Reliability requirement is provided to ensure that an OHTL be designed to withstand, without damage, the defined weather loads during its projected service life. The reliability requirement is expressed in terms of a return period, T, of the design weather loads which the OHTL is to be designed to withstand. Per the current BCH standard, the minimum acceptable target return periods, T, for reliability levels shall be:

- Level 1: T = 50 years (69 and 138 kV lines)
- Level 2: T = 100 years (230 and 287 kV lines)
- Level 3: T = 200 years (360 and 500kV lines)
- Level 4: T = 400 years (critical lines).

Security requirements correspond to special loads or measures intended to minimize the consequences of an initial mechanical or structural failure, and to prevent cascading failures of components and structures. Broken-Wire load cases are the principal security load cases, as shown in Table 1 later.

Security requirements may also include those site-specific loads due to geohazards such as earthquake, avalanche, ground slope or slide, flood, snow creep, etc.

In addition, an OHTL shall be designed and built so that it does not pose safety hazards to workers and the public. Safety requirements are mainly related to conditions encountered during construction and maintenance (C&M) work. C&M loads include loads developed during structure installation, conductor stringing, conductor snubbing and raising of equipment, as well as various maintenance related works such as climbing/mounting inspection, live line work, changing insulator, etc.

Basic Design Equation

The basic design equation for designing an FRP pole structure can be expressed by:

$$LF\ Q_T \leq SF\ R_C \quad (1)$$

where Q_T is the T-year extreme load; R_C is the characteristic strength having a lower exclusion limit (LEL) of 10% or lower; LF is the load factor, and shall take the value of unity (1.0); SF is the strength factor intended mainly for strength coordination.

Design Load Combinations

An FRP structure shall be designed to withstand the load combinations as listed in Table 1, where reliability and safety loads are applicable to all structures, and security loads are applicable to deadend structures only.

Table 1. Design loads applicable to FRP pole structures.

Load Category	Load Case	Ice	Wind	Temp.	Applicable Structures
Reliability (Intact)	IceWind	I_T	W_I	-5°C	All structures
	IceOnly	I_{OT}	0	-5°C	
	WindOnly	0	W_T	T_{AM}	
	UnEqIceT	$0.7I_T$	0	-5°C	
	Everyday	0	43Pa	T_{AM}	
	ExCold	0	0	T_{Cold}	
Security	BW-IceWind	I_T	W_I	-5°C	Deadend structures
	BW-IceOnly	I_{OT}	0	-5°C	
	BW-WindOnly	0	W_T	T_{AM}	
Safety	C&M	0	43Pa	$T_{10\%W}$	All structures

There are two basic load cases: T-year IceWind (with T-year ice thickness I_T, and its concurrent wind W_I), and T-year WindOnly (with T-year wind W_T). T-year IceOnly is a hypothetical load case with its T-year ice thickness I_{OT} assumed to be $1.5I_T$. The Everyday load is defined when conductor temperature is at the annual mean temperature T_{AM} and subject to wind of 43Pa (or 30km/hr). It is mainly intended for checking deflection. The ExCold is mainly intended to check the uplift condition when the conductor temperature is at the 50-year, extreme low temperature T_{Cold}.

The unequal ice load case "UnEqIceT" is derived from the IceWind load case. It is intended to capture the possible non-uniform ice deposit along a line. It is particularly important for FRP poles as it is often the governing load case for checking deflection. In designing a particular structure, it is assumed that the ice deposited on one side of the structure is $0.7I_T$, and no ice is deposited on the other side. This load may apply for any or all of the phases on one face at a time only (not both faces at the same time), and the remaining phase(s) shall assume full ice of $0.7I_T$ on both sides.

The broken wire load cases “BW-IceWind”, “BW-IceOnly”, and “BW-WindOnly” constitute the three security load cases, and are intended to stop cascading failure in case broken wires occur. They are all derived load cases from their respective IceWind, IceOnly, and WindOnly load cases for an intact line, by assuming any number of wires on one face at a time (not both faces at the same time) are broken with remaining wires remain intact and fully loaded.

In addition, the safety load cases are mainly related to construction and maintenance work under the 43Pa wind and the 10% low winter temperature ($T_{10\%W}$).

Strength Factors

The strength factors for various structures are given in Table 2 as per the current BCH standard (BCH 2019). Basically, deadend structure shall be stronger than an angle structure, which shall be stronger than a tangent structure. Guy wire, foundation and soil/rock shall be stronger than the associated structure. Strain structure has strain insulators but is not designed to withstand security loads.

Table 2. Strength factors for various structures

Component	Reliability (Intact)	Security		Safety
		Ice/Wind	Site Specific	
Tangent structure	0.90	N/A	1.00	0.50
Angle structure	0.80	N/A	1.00	0.50
Strain structure	0.75	N/A	1.00	0.50
Deadend structure	0.75	0.75	1.00	0.50
Guy wire	0.85(1)	0.85(1)	1.00	0.50
Foundation	0.90(1)	0.90(1)	1.00	0.50
Soil / rock	0.85(1)	0.85(1)	1.00	0.50

Note: (1) Additional strength factor. For exmaple, the strength factor for a deadend structure’s foundation is 0.90 x 0.75 = 0.675.

Deflection Limits

FRP poles can undergo very large deflections without mechanical failure so that they may behave like “fishing rods” if no deflection limits are imposed. An overly flexible structure could significantly reduce ground clearance, demand a much wider right of way, and compromise its maintainability by the field crew. Per the current BCH standard (BCH 2019), three deflection limits are set for FRP pole structures, as given in Table 3.

Table 3. Deflection limits for FRP structures

Load Case	Deflection Limit
Everyday	2%H
CSA Wind (230Pa wind at 40^{o}C)	4%H
Ultimate “Intact” Load	10%H

Note: H is the height of structure from the ground fixity point.

CONNECTION DESIGN

Usually connection design is not included in a commercial software for power line pole structure design. In addition, there is no well-established connection design standard available in the industry for pole structures. FRP poles are essentially tapered thin tubes. For example, the RS modules have wall thicknesses ranging from 9.7mm to 11.8mm, and diameters ranging from 192mm to 1035mm. In addition, the FRP modules are very flexible with their modulus of elasticity ranging from

17 to 24 GPa, about one tenth of the steel value (of 200GPa). The great diameter-to-thickness ratios and small modulus of elasticity dictate that the strength of an FRP module is mostly governed by the local buckling. In order to evaluate the FRP pole performance to withstand design loads, two full scale structures were tested by BC Hydro and RS jointly at the EUCOMSA Tower Testing Station in Spain: a 230kV H-frame structure consisting of two 100ft FRP poles, one set of double steel crossarms, and one pair of steel cross braces; and a 230kV guyed dead end structure consisting of a single 80ft FRP pole (EUCOMSA 2016). Typical design load cases were applied during testing in certain sequence.
While applying the load for the extreme wind load case to the 230kV H-frame structure, one connection between a cross brace and the FRP pole fail prematurely, presumably due to local buckling (Figure 2). To address this issue, the current BCH connection design practice was examined closely. As a result, a semi-empirical method is recommended and is described next.

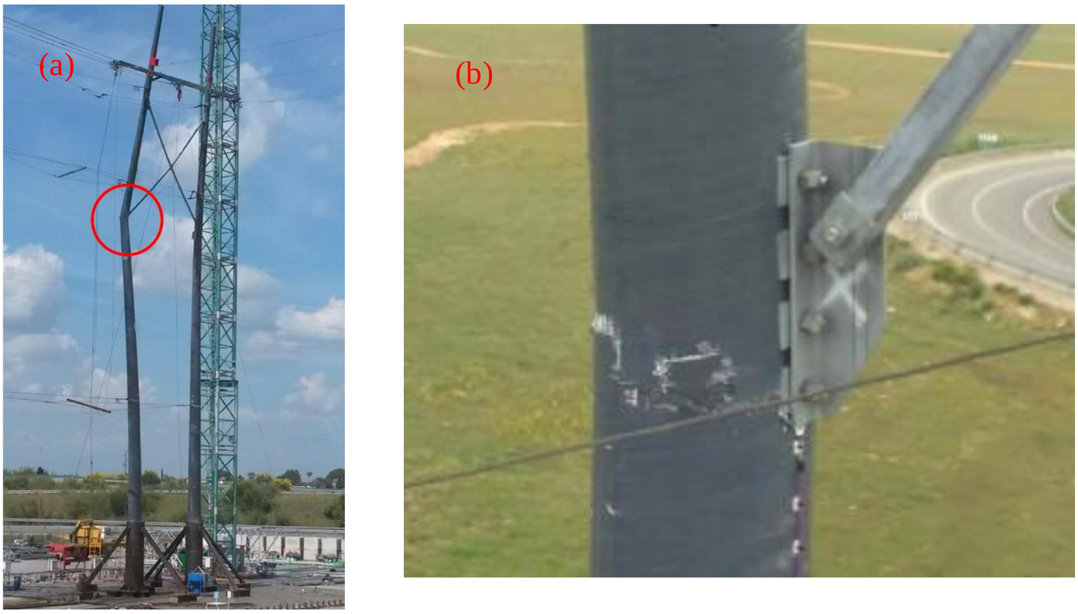

Figure 2. Showing **(a)** the failure of the FRP structure; and **(b)** the zoomed failure position.

Types of FRP Pole Connections

Typical FRP pole connections include:

- Connection between a cross arm and an FRP pole;
- Connection between a cross brace and an FRP pole;
- Connection between a guy wire and an FRP pole.

Types of Failure Modes

An FRP pole connection may possibly experience one of the following failure modes:

- Push-through failure;
- Hole bearing failure;
- Global failure;
- Bolt failure (tension, compression, shear and their combinations);
- Combinations of the above.

The connection shall be designed to withstand any of those failure modes with the appropriate strength reduction factor.

Design for Push-through

The simplified model for the push-through failure mode is illustrated in Figure 3. The maximum allowable push [P] for push-through is taken to be the minimum of [P_A], [P_B] and [P_δ], or

$$[P] = \min([P_A], [P_B], [P_\delta]) \quad (2)$$

where $[P_A]$ and $[P_B]$ account for local strength failure, and $[P_\delta]$ approximates the local buckling failure by setting deflection limit of 15%D. They are given by

$$[P_A] = F_b L t^2 / (0.954 K_A D) \quad (3)$$

$$[P_B] = F_b L t^2 / (0.954 K_B D) \quad (4)$$

$$[P_\delta] = 0.673 ELD / K_\delta / (D / t)^3 \quad (5)$$

where F_b and E are the bending strength and elastic modulus of the FRP tube, respectively. T and D are the wall thickness and the diameter of the FRP tube. L is the equivalent length of the FRP tube given by

$$L = L_w + D - E_w \quad (6)$$

where L_w is the length of the (curved) steel washer, and E_w is the end bolt's edge distance (Figure 3).

In addition

$$K_A = 1 - 0.0508\,\theta \quad (\theta < 15^o) \quad (7)$$

$$K_B = 0.5708 - 0.000018\,\theta - 0.000074\,\theta^2 \quad (\theta < 15^o) \quad (8)$$

$$K_\delta = 1 - 0.000360\theta - 0.00026\,\theta^2 \quad (\theta < 15^o) \quad (9)$$

where θ is defined by

$$\theta = B_w/(2D) \quad (10)$$

where B_w is the width of the steel washer (Figure 3).

See Appendix A for the detailed derivation of Eq. (7) – Eq. (9).

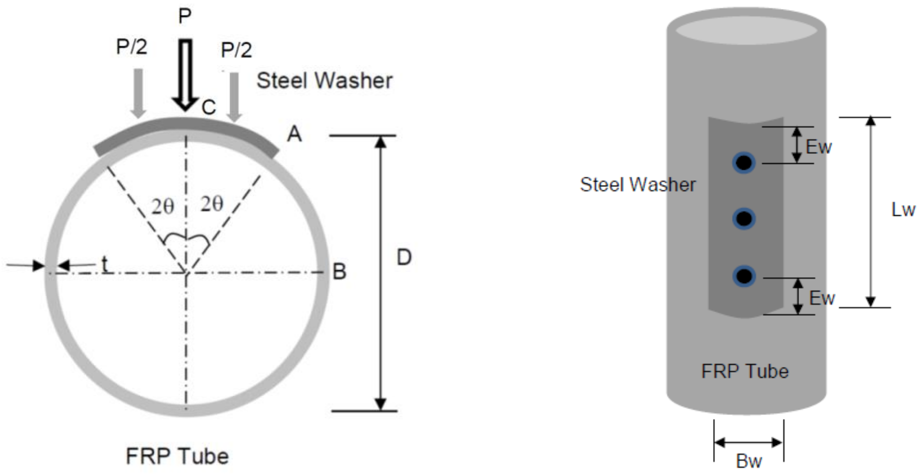

Figure 3. Simplified model for the push-through failure mode.

Design for Hole Wall Bearing Failure

The maximum allowable bearing capacity of the FRP hole wall to withstand bolts is determined by

$$[B] = n_b\, d_b\, t\, F_c \quad (11)$$

where n_b is the number of bolts, and d_b is the diameter of the bolt. F_c is the compression (or bearing) strength of the FRP.

Design Equations

A connection shall meet the following two design equations:

$$U_{PG} = (U_P^2 + U_G^2)^{0.5} < 1 \quad (12)$$

$$U_{BG} = (U_B^2 + U_G^2)^{0.5} < 1 \quad (13)$$

where, U_P is the usage for push-through, or $U_P = P/(SF[P])$ with SF being the strength factor. U_G is the global usage of the FRP pole at the connection as found from a design software such as PLS-POLE. U_B is the usage for the bolt bearing, or $U_B = B/(SF[B])$ where B is the bearing load exerted by bolts on the FRP wall. U_{PG} is the usage for the combined effect of push-through and global strength. U_{BG} is the usage for the combined effect of bolt bearing and global strength.

Additional Requirements

In addition, the bolt shall have adequate strength to avoid any failure due to tension, compression, shear, or their combinations.

Furthermore, the following guidelines for hole size and spacing shall be adhered to as per RS's recommendation (RS 2011):

- Minimum hole spacing: center-to-center distance of any two holes should be a minimum of 6d, where d is the diameter of larger of the two holes.
- Minimum edge distance: a minimum distance of 6d, where d is the hole diameter, should be maintained from the edge of the module to the center of the hole.
- Maximum hole size: hole diameters larger than 1.25in (32mm) in diameter are not recommended.

Illustrating Example

An improved T-bracket has recently been developed by BCH as the standard connection between the cross brace and the FRP pole for 230kV FRP H-frame structures as shown in Figure 4.

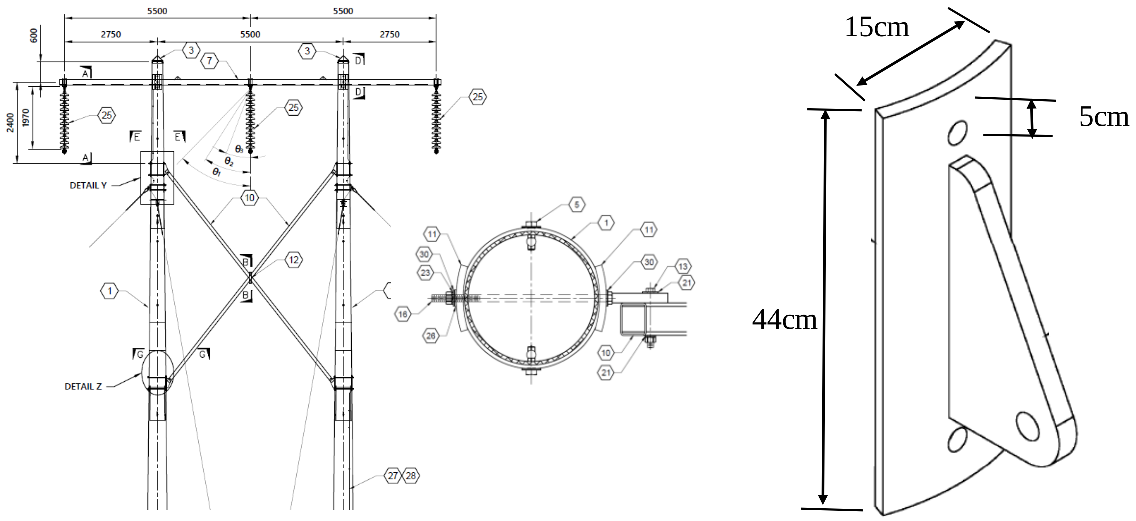

Figure 4. Sketches showing the BCH new 230kV FRP H-frame design (left) and the associated new brace bracket design (right).

Consider the T-bracket connection to RS FRP Modules M3, M4, M5, or M56. The FRP modules are assumed to have common properties of E = 20000MPa, Fb = 250MPa, and Fc = 170MPa. Two 7/8" Gr.5 bolts are used to connect the T-bracket to the FRP module. A SF of 0.9 is used for the connection design. The maximum allowable brace capacity can be determined using the proposed method above, and the results are summarized in Figure 5 for five modules M3, M4, M5, and M56, and three locations L/4, L/2, and 4L/4, respectively. Here L/4, L/2, and 3L/4 refer to the connection at the ¼, ½, and ¾ of the module length measured from the smaller end. For M3, the three curves coincide as the brace capacity is governed by the bolt bearing on the FRP wall. For both M4 and M5, brace capacity tends to decrease as the connection moves to the portion of the module with greater diameter, due to the local buckling effect. For M56, the L/4 and L/2 curves coincide as the brace capacity is governed by the bolt bearing on the FRP hole wall, while the 3L/4 curve is significantly lower due to the local buckling effect. Generally, brace capacity decreases with greater global pole usage.

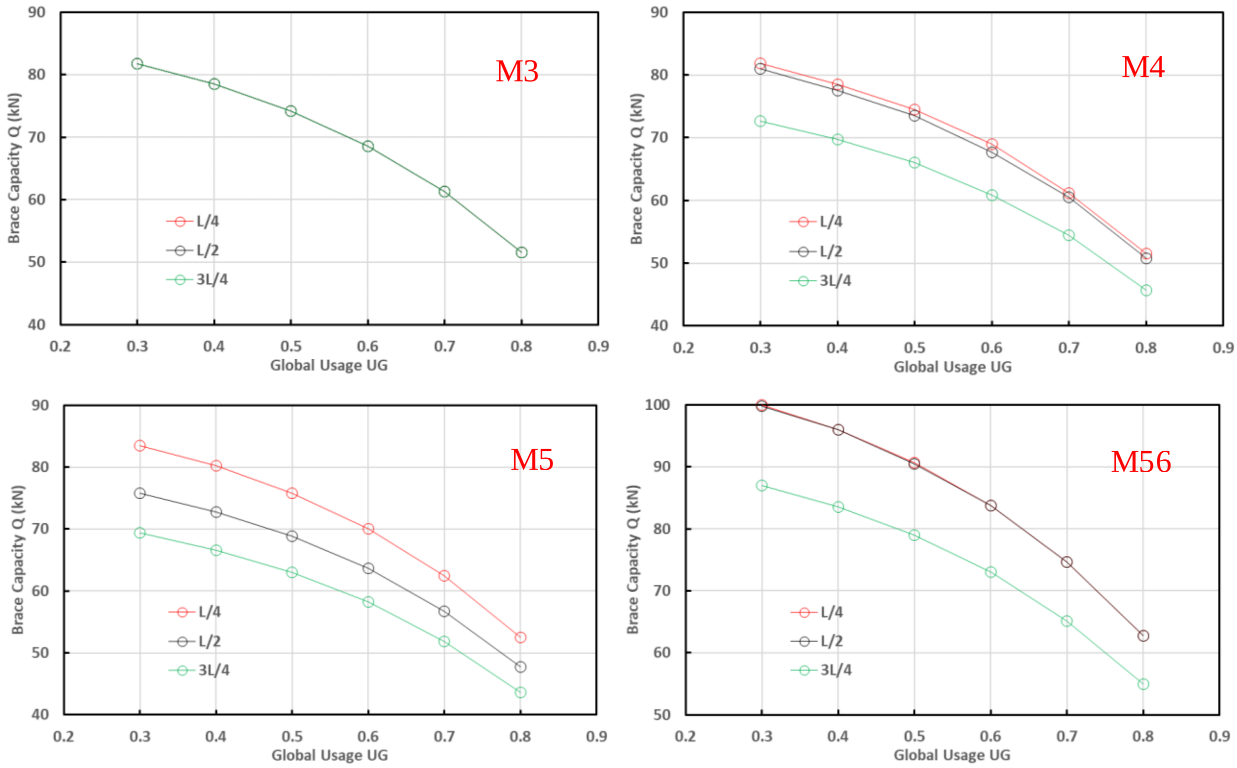

Figure 5. Summarized results of brace capacities for various RS modules.

CONCLUSION

The design methodology for FRP pole structures has been presented systematically based on the current BC Hydro standard and practice. The need for unequal ice load case and deflection limit has been emphasized. To fill the gap of missing industry standard practice on connection design, a semi-empirical design method has been proposed for designing FRP connections with the intention to account for local bucking and bolt bearing, Further full-scale, component tests are required to validate the proposed connection design method.

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APPENDIX A. DERIVATION OF THE PUSH-THROUGH MODEL FOR FRP POLE

The proposed Push-through model for FRP pole is shown in Figure 3. Here it is assumed that the total push-through load P is divided into two halves, each acting on the pole at the quarter location on either side.

Per Case No. 4 of Table 9.2 of the book “Roark’s Formulas for Stress and Strain” (2002), the bending moment M_C, the axial force N_C, the shear force V_C, and the radial deflection δ_C at the central point C are given, respectively, by

$$M_C = \frac{PD}{4\pi}[s(s-\pi+\theta)+k_2(1+c)] \tag{A1}$$

$$N_C = \frac{P}{2\pi}s^2 \tag{A2}$$

$$V_C = 0 \tag{A3}$$

$$\delta_C = \frac{PD^3}{16EI\pi}\left[\frac{\pi k_1(\pi-\theta-sc)}{2}+k_2 s(\pi-2\theta)-2k_2^2(1+c)\right] \tag{A4}$$

where s = sinθ, c = cosθ. In addition, both k_1 and k_2 may take unity for a thin ring.

The bending moments at Point A and Point B are derived from Eqs. (A1) – A(3) as below:

$$M_A = \frac{PD}{2\pi}\left[\frac{s(s-\pi+\theta)}{2}+\frac{k_2(1+c)}{2}-\frac{s^2(1-cos2\theta)}{2}-\frac{(sin2\theta-s)\pi}{2}\right] \tag{A5}$$

$$M_B = \frac{PD}{2\pi}\left[\frac{s(s-\pi+\theta)+k_2(1+c)}{2}-\frac{s^2}{2}-\frac{(1-s)\pi}{2}\right] \tag{A6}$$

Eqs. (A4) – (A6) may be re-written as

$$\delta_C = 0.223K_\delta\frac{P}{EL}\left(\frac{D}{t}\right)^3 \tag{A7}$$

$$M_A = 0.159K_A PD \tag{A8}$$

$$M_B = 0.159K_B PD \tag{A9}$$

where

$$K_\delta = 1.0696\left[\frac{\pi k_1(\pi-\theta-sc)}{2}+k_2 s(\pi-2\theta)-2k_2^2(1+c)\right] \tag{A10}$$

$$K_A = \left[\frac{s(s-\pi+\theta)}{2}+\frac{k_2(1+c)}{2}-\frac{s^2(1-cos2\theta)}{2}-\frac{(sin2\theta-s)\pi}{2}\right] \tag{A11}$$

$$K_B = \left[\frac{s(s-\pi+\theta)+k_2(1+c)}{2}-\frac{s^2}{2}-\frac{(1-s)\pi}{2}\right] \tag{A12}$$

K_A, K_B, and K_δ are curve-fitted to the following equations:

$K_A = 1 - 0.0508\ \theta$ $(\theta < 15^o)$ (A13)

$K_B = 0.5708 - 0.000018\ \theta - 0.000074\ \theta^2$ $(\theta < 15^o)$ (A14)

$K_\delta = 1 - 0.00036\theta - 0.00026\ \theta^2$ $(\theta < 15^o)$ (A15)

It can be seen from Figure A1 that the fitted equations match the original data points very well for the range of $0 < \theta < 15^o$.

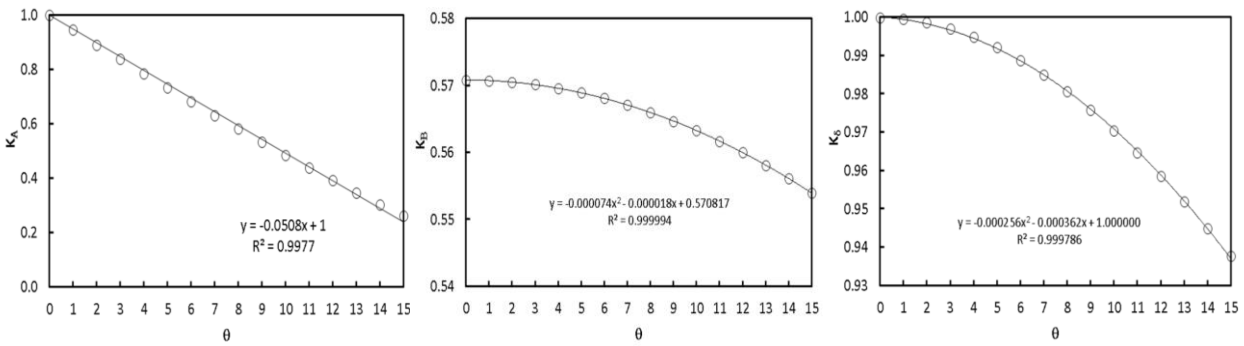

Figure A1. Curve-fitting K_A, K_B, and K_δ versus θ.