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**RESISTANCE FACTOR FOR WELDED WIRE FABRIC STEEL REINFORCEMENT**

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**Abstract:** Welded Wire Fabric (WWF) is increasingly used to resist shear in precast, prestressed bridge girders and to resist flexure in deck slabs. A resistance factor for Welded Wire Fabric is calibrated in accordance with CSA S408 Guidelines for the Development of Limit States Design Standards. Statistical parameters for WWF yield stress at 0.5% strain were derived from a database of 447 test results. Target reliability indices for WWF resisting shear are identical to those for sections with mild steel reinforcement, whereas those for flexure are slightly higher because the response of the WWF-reinforced section is slightly less ductile. The recommended resistance factor is 0.95.

1. **INTRODUCTION**

The first machine to manufacture Welded Wire Fabric (WWF) was patented by Massachusetts inventor John Perry in 1901, who envisaged that the wire sheets would be used for fences (WRI 2014). By 1906, catalogs listed WWF as reinforcement for concrete, and by the time of the First World War it was used to reinforce concrete pavements. Its use was extended to concrete buildings, such as the Empire State Building in New York City, in the ‘30s and since 2004 it has been used as shear reinforcement in precast, prestressed concrete bridge girders (WRI 2014). Two recent extensive infrastructure projects in Ontario, the Rt. Hon. Herb Grey Parkway in Windsor and the Highway 407 East Project, feature large numbers of precast NU girders with WWF shear reinforcement (Stang 2014, 407 EDG 2013).

Figure 1 shows typical stress-strain responses for conventional mild steel and wire. The mild steel has a clearly defined yield plateau and exhibits significant strain hardening before failure. In contrast, the wire does not exhibit a defined yield point or strain hardening due to the cold-drawing manufacturing process (WRI 2014). ASTM Standard A1064/A1064M *Standard Specification for Carbon-steel Wire and Welded-wire Reinforcement, Plain and Deformed, for Concrete* (ASTM 2016a) therefore permits the yield strength of wire reinforcement to be that corresponding to an offset stress of 0.1%, fy,0.1%off, or at absolute strains of 0.35% or 0.5%, fy,0.35% or fy,0.5%, respectively, as shown in Figure 2.

The typical specified yield strength of WWF is 500 MPa (ASTM 2016a), which is 25% greater than the typical yield strength of reinforcing steel, 400 MPa, allowing relatively smaller volumes of WWF to be used to provide a given resistance.

Currently, the Canadian Highway Bridge Design Code (CHBDC), (CSA 2014a) does not specify a unique resistance factor for WWF. The investigation summarized in this paper therefore had the objective of deriving a suitable resistance factor, accounting for the specific strength characteristics of the material.

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| Figure 1: Stress-strain responses | Figure 2: WWF yield stress definitions |

Section 2 of the paper summarizes statistical parameters defining the yield strength of WWF. Section 3 presents the mathematical basis of the calibration including the statistical parameters used to quantify the other load and resistance parameters. Sections 4 and 5 summarize the target reliability indices and associated resistance factors for WWF contributing to the flexural and shear resistances, respectively, of a concrete section. Section 6 presents brief conclusions.

1. **STATISTICAL ANALYSIS OF WWF YIELD STRESS DATA**

Approximately 500 NU girders fabricated for the Rt. Hon. Herb Grey Parkway were removed from the project in response to concerns that individual WWF panels used to provide shear reinforcement had been connected by tack welding (Stang, 2014). Using hydrodemolition, the WWF reinforcement was exposed in six girders and a large quantity of specimens was obtained for tensile testing. These data were made available for the present investigation.

Yield strengths defined using the 0.1% offset and 0.5% strain methods, shown as fy,0.1% off and fy,0.5%, respectively, in Fig. 2, were analysed. The data were checked for outlying values using ASTM E178 *Standard Practice for Dealing with Outlying Observations* (ASTM 2016b) and data from different girders were combined when conventional f- and t-tests demonstrated this was appropriate (Zhang 2017).

The sample statistics (i.e., the number of specimens, n, mean, x̅, standard deviation, s, and coefficient of variation, V) for the 0.1% offset and 0.5% strain yield strengths are shown in Table 1. The 4 MPa difference between the mean yield strengths is not statistically significant (p = 0.35).

Table 1: Sample yield strength statistics

|  |  |  |
| --- | --- | --- |
|  | fy,0.1%off | fy,0.5% |
| n | 87 | 447 |
| x̅ (MPa) | 618 | 622 |
| s (MPa) | 31.3 | 37.4 |
| V = s/x̅ | 0.051 | 0.060 |

The 0.5% strain yield strengths are graphed on normal probability paper in Figure 3. The graph is reasonably linear, indicating that the strengths can be assumed to be normally distributed.



Figure 3: 0.5% strain yield strength data graphed on normal probability paper.

As it is conventional to base calibration of resistance factors for steel on the static yield strength, the mean static yield strength of WWF will be taken 29.3 MPa less than the strength at the tested strain rate (Schmidt et al 2002). For the mean static yield strength of (622-29.3=) 592.7 MPa, the bias coefficient, , is (592.7/500=) 1.185 and the coefficient of variation, V, is (37.4/592.7=) 0.0631. These values will be used in the reliability analysis.

1. **CALIBRATION PROCEDURE AND STATISTICAL PARAMETERS**

In accordance with the provisions of CSA S408 *Guidelines for the Development of Limit State Design Standards* (CSA 2011), the reliability index, , is computed as:

[1] $ ^{ln (\overbar{R}/\overbar{S})}/\_{(V\_{R}^{2}+V\_{S}^{2})^{0.5}} $

where $\overbar{R}$ and $\overbar{S}$ are the mean values of the resistance and load effect, respectively, and VR and VS are the coefficients of variation of the resistance and load effect, respectively. The calibration process is as follows:

1. Assess the relative ductility of concrete elements reinforced with conventional mild steel reinforcement and those reinforced with WWF.
2. On this basis, assess whether the target reliability index for an element reinforced with WWF should be more stringent than that for an element reinforced with conventional mild steel reinforcement.
3. Determine the reliability index, s, for a component with conventional mild steel reinforcement that meets the design criteria specified in the CHBDC (CSA 2014).
4. Based on the result of Step 2, determine the target reliability index for a component reinforced with WWF, WWF.
5. Determine the resistance factor, WWF, that achieves the target reliability index determined in Step 4.

Table 2 shows the statistical parameters assumed for the various random quantities used to compute resistance. For simplicity, the various geometric quantities (e.g.: effective depth, d; element width, b; steel area, As; WWF area, AWWF; stirrup spacing, s) are assumed deterministic. The cracking strength of concrete, fcr, is conventionally assumed to be proportional to the square root of the compressive strength f’c, so the bias coefficient of fcr, is taken as the square root that for f’c. The variability of cracking strengths is considerable, however, so the coefficient of variation for fcr is assumed to be the same as that for f’c. The statistical parameters for yield strength of conventional reinforcement are from Ellingwood et al (1980), and would likely have been used for the original calibration of s = 0.90. More recently, Nowak et al (2003) have suggested somewhat more favourable statistical parameters for yield strength,  = 1.145 and V = 0.05. These values do not correspond to the static yield strength, however, so the values shown in Table 2, which are modified to pertain to the static yield strength, will be adopted.

Table 2 also shows the professional factors adopted for the computed resistance in bending and shear. The professional factor accounts for bias and uncertainty of the model used to compute the resistance.

Table 2: Statistical parameters for resistance quantities

|  |  |  |  |
| --- | --- | --- | --- |
| Variable |  | V | Source |
| f’c | 1.31 | 0.21 | (Bartlett 2007) |
| fcr = 0.4 √ f’c | 1.145 | 0.21 | — |
| fy (Mild Steel) | 1.125 | 0.098 | (Ellingwood et al 1980) |
| fy (Mild Steel) | 1.072 | 0.054 | (Nowak et al, 2003) – see text |
| fy (WWF) | 1.185 | 0.063 | — |
| Professional factor (shear) | 1.27 | 0.12 | (Bentz et al, 2017) |
| Professional factor (bending) | 1.02 | 0.06 | (Ellingwood et al 1980) |

Table 3 shows the statistical parameters assumed for the various load effects. Dead loads D1, D2 and D3 correspond to the weights of factory-produced components, field-cast components and asphalt wearing surfaces, respectively. The total live load effect is the sum of the static effect, L, and the allowance for the dynamic effect of the live load, DLA. All are taken from the Commentary to Sections 14.1 to 14.3 of the CHBDC (CSA 2014b).

Table 3: Statistical parameters for D1, D2, D3, L, and DLA (CHBDC 2014b)

|  |  |  |  |
| --- | --- | --- | --- |
| Demand |  |  | V |
| D1 | 1.10 | 1.03 | 0.08 |
| D2 | 1.20 | 1.05 | 0.10 |
| D3 | 1.50 | 1.03 | 0.30 |
| L | 1.70 | 1.35 | 0.035 |
| DLA | 1.70 | 0.4 | 0.80 |

1. **RESISTANCE FACTOR FOR SECTIONS REINFORCED WITH WWF RESISTING BENDING MOMENTS**
	1. **Target Reliability Index for WWF**

Figure 4 compares moment-curvature (M- relationships determined using Response-2000 (Bentz, 2000) for sections with conventional mild steel reinforcement, As, and different quantities of WWF reinforcement. Two sets of relationships are shown, for sections with steel reinforcement ratios, s, of 0.6% and 1.2%. The program does not have a pre-defined stress strain relationship for WWF so an elastic-perfectly plastic idealization, with a yield strength of 600 MPa and a rupture strain of 8%, was adopted. For the lightly reinforced sections, s = 0.6%, strain hardening of the mild steel occurred so the yield moment corresponds to that of the section with AWWF = 4/6 As, whereas the ultimate moment corresponds more closely to that of the section with AWWF = 4/5 As. For the heavily reinforced sections, s = 1.6%, little strain hardening of the mild steel occurred so the yield and ultimate capacities correspond roughly to those for the section AWWF = 4/6 As and are less ductile than those for the section with AWWF = 4/5 As . The ductility of sections with near-identical ultimate capacities is similar, though the sections with mild reinforcement are consistently more ductile than those with WWF reinforcement.

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| Figure 4: Moment-curvature relationships for WWF- and mild-steel-reinforced sections | Figure 5: Curvature ductility ratios for different moment capacities |

Figure 5 compares the curvature ductility ratios, u/y , for sections with WWF and mild steel reinforcement that have identical ultimate flexural capacities, MMax. At a given flexural capacity, the section reinforced with WWF always has a smaller ductility ratio than the section reinforced with conventional mild steel. For example, for MMax/bd2f’c = 0.08, u/y equals approximately 12 and 9.8 for the sections reinforced with conventional mild steel and WWF, respectively. Conservatively, this difference is deemed to be similar to that between Element Behaviour Categories E3 and E2 in Clause 14 of the CHBDC (CSA 2014a): E3 corresponds to “gradual failure with warning” and E2 corresponds to “failure with little or now warning but retains post-failure capacity.” The target reliability indices are consistently 0.25 less for Category E3 than for Category E2. Thus the target reliability index for sections reinforced with WWF, WWF , is taken to be:

[2] WWF = s + 0.25

where s is the reliability index obtained for sections reinforced with conventional mild steel.

* 1. **Calibration of WWF for Flexure**

The calibration considered rectangular sections with 0.002 ≤ s ≤ 0.02. The steps were as follows:

1. Compute the design flexural resistance, Mr, in accordance with the CHBDC (CSA 2014a).
2. Compute nominal demands due to the cast-in-place concrete, Ms, asphalt, Ma, live load, ML, and dynamic effect of the live load, MDLA, to exactly satisfy the design criterion in the CHBDC, specifically:

[3] Mr = 1.2 Ms + 1.5 Ma +1.7 (ML + MDLA)

It is assumed that Ms/Ma = 2.67, which is representative of a 235 mm slab with 90 mm asphalt, and (ML+MDLA)/(Ms+Ma) = , where 0.5 ≤  ≤ 2.5.

1. Compute the mean resistance $\overline{R}$ as:

[4] $\overline{R}$ = Pbd2 [$\overline{f}\_{y}$- ($\overline{f}\_{y}$)2/(21$\overline{f}'\_{c}$)]

where Pis the bias coefficient for the professional factor from Table 2, $\overline{f}\_{y}$ and $\overline{f}'\_{c}$ are the mean values of fy and f’c, respectively, and the stress block parameter, 1 = 0.85 – 0.0015f’c.

1. Compute the standard deviation of the resistance, R, neglecting the professional factor, by a Taylors Series approximation as:

[5] R =$\sqrt{\left(\frac{∂R}{∂f\_{c}^{'}}\right)^{2}\_{f'c}^{2}+\left(\frac{∂R}{∂f\_{y}^{}}\right)^{2}\_{fy}^{2}}$

where f’c and fy are the standard deviations of f’c and fy, respectively,

[6] $∂$R/$∂$f’c = bd2 [($\overline{f}\_{y}$)2/(21$\overline{f}'\_{c}$2)]

and

[7] $∂$R/$∂$fy = bd2 [ - 2$\overline{f}\_{y}$) /(1$\overline{f}'\_{c}$)]

1. Compute the coefficient of variation of the resistance, VR, including the professional factor, as:

[8] VR =$\sqrt{\left(\frac{\_{p}\_{R}}{\overline{R}}\right)^{2}+ V\_{p}^{2}}$

1. Compute s using Eq. [1] and the target WWF using Eq. [2].
2. For an assumed value of WWF, compute WWF to satisfy Eq. [3] and determine WWF using Equations [4] through [8].
3. Compare the obtained WWF to the target value from Step 6: if the obtained value exceeds the target, repeat Step 7 with a increased value of WWF.

**4.3 Recommended WWF Values**

Table 4 shows the ratios of the average actual-to-target WWF values obtained for various WWF values and different live-to-dead load ratios, , where the target WWF are determined from s values computed using the statistical parameters for fy reported by Ellingwood et al (1980). The use of WWF = 0.90 is clearly conservative, as it yields WWF values that are on average 24% greater than the target values. The influence of the live-to-dead load ratio, , on the actual-to-target WWF values is slight. To achieve the target WWF values, a resistance factor WWF of approximately 1.025 is appropriate.

Table 4: Actual/target WWF ‒ fy statistics per Ellingwood et al (1980)

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| WWF |  = 0.5 | 1.0 | 1.5 | 2.5 | Avg. |
| 0.90 | 1.22 | 1.24 | 1.25 | 1.25 | 1.24 |
| 0.95 | 1.12 | 1.15 | 1.15 | 1.16 | 1.14 |
| 1.00 | 1.02 | 1.05 | 1.06 | 1.07 | 1.05 |
| 1.05 | 0.93 | 0.96 | 0.98 | 0.99 | 0.96 |
| 1.10 | 0.84 | 0.88 | 0.90 | 0.91 | 0.88 |

Table 5 shows the ratios of average actual-to-target WWF values where the target WWF are determined from s values computed using the statistical parameters for fy reported by Nowak and Szerszen (2003) as modified to pertain to the static yield strength. The use WWF = 0.90 is less conservative and a resistance factor WWF slightly greater than 0.95 is appropriate. For WWF = 0.95 and  = 0.5, the WWF values increase from 4.6 to 5.4 as the reinforcement ratio increases from the minimum to maximum values. For WWF = 0.95 and  = 2.5, the WWF values increase from 5.7 to 6.6. It would therefore seem appropriate to adopt WWF = 0.95.

Table 5: Actual/target WWF ‒ fy statistics per Nowak and Szerszen (2003)

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| WWF |  = 0.5 | 1.0 | 1.5 | 2.5 | Avg. |
| 0.90 | 1.10 | 1.09 | 1.09 | 1.09 | 1.09 |
| 0.95 | 1.01 | 1.01 | 1.01 | 1.01 | 1.01 |
| 1.00 | 0.92 | 0.93 | 0.93 | 0.93 | 0.93 |
| 1.05 | 0.84 | 0.85 | 0.85 | 0.86 | 0.85 |
| 1.10 | 0.76 | 0.77 | 0.78 | 0.79 | 0.78 |

1. **RESISTANCE FACTOR FOR SECTIONS REINFORCED WITH WWF RESISTING SHEARING FORCES**
	1. **Target Reliability Index for WWF**

Figure 6 shows the load-deflection response of WWF- and conventional-mild-steel-reinforced beams with transverse reinforcement ratios of 0.6% as predicted using Response-2000 (Bentz 2000). The response is essentially linear-brittle in all cases, indicating that the use of WWF does not impact the ductility of an element subjected to shearing force. The analysis was repeated for beams with transverse reinforcement ratios between 0.1% to 0.6% (Zhang 2017), yielding similar conclusions. Figure 7 shows the deflection at maximum load for all sections investigated: there is a clear linear trend and the use of WWF instead of conventional reinforcement has no impact. It was therefore deemed appropriate to take the target reliability index for sections reinforced with WWF, WWF, to be identical to that computed for sections reinforced with conventional reinforcement, s.

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| Figure 6: Load-deflection relationships | Figure 7: Deflection at maximum load |

* 1. **Calibration of WWF for Shear**

Figure 8 shows the cross-section of the composite precast CPCI girder/cast-in-place slab bridge used as the basis of the calibration. The bridge consists of two 37-m spans that are continuous over the interior support. The construction sequence causes the girder self weight and slab to be carried by the simply supported precast girder, and the superimposed dead and live load to be carried by the indeterminate system.

The required area of conventional steel reinforcement was determined at 10th points along the span in accordance with the requirements of Clause 8.9 of the CHBDC (CSA 2014a). The total shear resistance is:

[9] Vr = Vc + Vs + Vp



Figure 8: Composite CPCI girder/cast-in-place slab cross section

where $V\_{c}$ and $V\_{s}$ are the factored shear resistances provided by the concrete and the reinforcing steel, respectively. The factored shear resistance provided by the draped prestressing strands,$ V\_{p}$, was assumed negligible. The resistance provided by the concrete is computed as:

[10] Vc = 2.5  c fcr bv dv

where  is a factor used to account for the shear resistance of cracked concrete, c is the resistance factor for concrete, 0.75, fcr is the cracking strength of concrete (see Table 2) and bv is the effective width within the effective shear depth, dv. The required stirrup area, Av, was computed as:

[11] $\frac{A\_{v}}{s}=\frac{V\_{s}}{∅\_{s}f\_{y}d\_{v}cotθ}$

where s is the stirrup spacing, s is the resistance factor for reinforcing steel, 0.90, and  is the angle of inclination of the principal compression stresses. Further details are provided in Zhang (2017).

The calibration procedure was similar to that described in Section 4.2 above except that the statistical parameters for the resistance were computed from a Taylor Series expansion that included the concrete cracking strength, fcr, instead of the compressive strength, f’c. Values of WWF were selected to determine the necessary AvWWF/s, and the associated WWF was computed and compared to the target value.

* 1. **Recommended WWF Values**

Table 6 shows the results obtained using the using the statistical parameters for fy reported by Nowak and Szerszen (2003) as modified to pertain to the static yield strength and WWF = 0.95. The tenth point labelled 0 is at the abutment and the tenth point labelled 10 is at the interior support. The design based parameters are computed from the provisions of Clause 8.9 of the CHBDC (CSA 2014a): the required Vs is that necessary to ensure that the factored resistance, Eq. [9], exactly equals the factored demand. The required area of steel reinforcement, Avs/s, is computed for s = 0.9, and the associated reliability index, s, is computed using the statistical parameters for the resistance and load variables shown in Tables 2 and 3, respectively.

In Table 6, the required area of WWF reinforcement, AvWWF/s, is computed using WWF = 0.95. The associated reliability indices, WWF, are 3-4% greater than the target values, and are essentially independent of the ratio AvWWF/s. In this case, WWF equals s for WWF = 0.98.

Similar calculations were carried out using the statistical parameters for fy reported by Ellingwood et al (1980). In this case the ratio of WWF/s equals 1.0 for WWF = 1.01.

Table 6: Details of calibration of WWF for shear

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
|  | Design-based parameters | Steel Reinforcement | WWF Reinforcement |  |
| 10th point |  |  | Vc | Req’dVs | Req’d Avs/s | s | Req’d AvWWF/s | WWF | WWFs |
|  |  | (deg) | (kN) | (kN) | (mm2/mm) |  | (mm2/mm) |  |  |
| 0 | 0.24 | 32.0 | 372 | 998 | 0.962 | 4.95 | 0.729 | 5.13 | 1.04 |
| 1 | 0.15 | 36.6 | 232 | 894 | 1.026 | 5.04 | 0.777 | 5.22 | 1.04 |
| 2 | 0.12 | 40.2 | 180 | 707 | 0.922 | 5.08 | 0.699 | 5.27 | 1.04 |
| 3 | 0.10 | 42.4 | 158 | 493 | 0.693 | 5.12 | 0.525 | 5.31 | 1.04 |
| 4 | 0.10 | 43.1 | 152 | 269 | 0.387 | 5.06 | 0.293 | 5.23 | 1.03 |
| 5 | 0.10 | 43.0 | 153 | 240 | 0.344 | 5.35 | 0.261 | 5.51 | 1.03 |
| 6 | 0.10 | 42.4 | 158 | 473 | 0.666 | 5.36 | 0.505 | 5.54 | 1.03 |
| 7 | 0.12 | 40.5 | 176 | 684 | 0.903 | 5.26 | 0.685 | 5.44 | 1.03 |
| 8 | 0.14 | 37.4 | 218 | 865 | 1.022 | 5.17 | 0.774 | 5.35 | 1.04 |
| 9 | 0.20 | 33.7 | 305 | 992 | 1.021 | 5.08 | 0.774 | 5.26 | 1.04 |
| 10 | 0.13 | 38.4 | 202 | 1313 | 1.606 | 5.05 | 1.217 | 5.24 | 1.04 |

It is recommended that the resistance factor for Welded Wire Fabric in the Canadian Highway Bridge Design Code be set equal to 0.95. This value is identical to that currently used for prestressing strands, which are also cold-drawn wires. The research reported in this paper has demonstrated for WWF = 0.95 yields reliability indices that markedly exceed those computed for concrete reinforced with conventional mild steel reinforcement when the statistical parameters for yield strength, fy, are as reported by Ellingwood et al (1980). It yields reliability indices that are close to those computed using the statistical parameters for fy reported by Nowak and Szerszen (2003) and modified to pertain to the static yield strength.

1. **SUMMARY AND CONCLUSIONS**

Welded Wire Fabric (WWF) is increasingly used to resist shear in precast, prestressed bridge girders and to resist flexure in deck slabs. A resistance factor for Welded Wire Fabric is calibrated in accordance with CSA S408 Guidelines for the Development of Limit States Design Standards. Statistical parameters for WWF yield stress at 0.5% strain were derived from a database of 447 test results. Target reliability indices for WWF resisting shear are identical to those for sections with mild steel reinforcement, whereas those for flexure are slightly higher because the response of the WWF-reinforced section is slightly less ductile.

It is recommended that the resistance factor for Welded Wire Fabric in the Canadian Highway Bridge Design Code be set equal to 0.95. This value is identical to that currently used for prestressing strands, which are also cold-drawn wires. Use of WWF of 0.95 yields reliability indices that markedly exceed those computed for concrete reinforced with conventional mild steel reinforcement when the statistical parameters for yield strength, fy, are as reported by Ellingwood et al (1980). It yields reliability indices that are close to those computed using the statistical parameters for fy reported by Nowak and Szerszen (2003) and modified to pertain to the static yield strength.

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**References**

407 East Development Group (407EDG). 2013. *Highway 407 East Phase One Design and Construction Report #1-6*. 407 East Development Group. Pickering, Ontario, Canada.

American Society for Testing and Materials (ASTM). 2016a. *A1064/A1064M-16a: Standard Specification for Carbon-Steel Wire and Welded Wire Reinforcement, Plain and Deformed, for Concrete*. ASTM, West Conshohocken, PA, USA.

American Society for Testing and Materials (ASTM). 2016b. *E178-16: Standard Practice for Dealing With Outlying Observations.* ASTM, West Conshohocken, PA, USA.

Bartlett, F. M. 2007. Canadian Standards Association Standard A23.3-04 Resistance Factor for Concrete in Compression. *Canadian Journal of Civil Engineering*, **34**(9): 1029-1037.

Bentz, E. C. 2000. *Sectional Analysis of Reinforced Concrete Members*, PhD Thesis. Department of Civil Engineering, University of Toronto. 310 pages.

Bentz, E. C., and Collins, M. P. 2017. Updating the ACI shear design provisions. *Concrete International*, **39** (9): 33-38.

Canadian Standards Association (CSA). 2011. *S408-11 Guidelines for the Development of Limit States Design Standards*. Canadian Standards Association. Mississauga, Ontario, Canada.

Canadian Standards Association (CSA). 2014a. *CAN/CSA-S6-14 Canadian Highway Bridge Design Code*. Canadian Standards Association, Mississauga, Ontario, Canada.

Canadian Standards Association (CSA). 2014b. *CAN/CSA-S6.1-14 Commentary on CSA S6-14, Canadian Highway Bridge Design Code*. Canadian Standards Association, Mississauga, Ontario, Canada.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A. 1980. *Development of a Probability Based Load Criterion for American National Standard A58: Building Code Requirements for Minimum Design Loads in Buildings and Other Structures*. US Department of Commerce, National Bureau of Standards. USA.

Nowak, A. S., and Szerszen, M. M. 2003. Calibration of design code for buildings (ACI 318): Part 1-Statistical models for resistance. *ACI Structural Journal*, **100**(3): 377-382.

Schmidt, B.J., & Bartlett, F.M. 2002. Review of Resistance Factor for Steel: Data Collection. *Canadian Journal of Civil Engineering*, **29**(1): 98-108.

Stang, R. (2014). Windsor’s PSI Wins Big on Parkway Girder Job. Media Release. <http://dailycommercialnews.com/Labour/News/2014/6/Windsors-PSI-wins-big-on-parkway-girder-job-DCN060739W/> Accessed 22 September, 2016

Wire Reinforcement Institute (WRI). 2014. *Historical Data o Wire, Triangular Wire Fabric/Mesh and Welded Wire Concrete Reinforcement (TF 101 R 14)*. Wire Reinforcement Institute, <http://wirereinforcementinstitute.org/resources/technical-documents/>, accessed 14 Jan 2018.

Zhang, L.H. 2017. *Resistance Factor Calibration for Welded Wire Fabric Steel Reinforcement*. BESc thesis, Department of Civil and Environmental Engineering, University of Western Ontario, London.