

A State-of-the-Art Review of Testing by Analysis in Cold-Formed Steel Design

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Abstract

New product development is crucial to allow innovation in the cold-formed steel structural industry. However, the required physical testing of new components and assemblies are often a cost barrier which prevents implementation and slows new product development. Testing by analysis can be a good alternative to physical testing as it reduces the expense and time for performing physical experiments, however, two considerations are necessary to ensure accurate results. First, it requires a rational engineering analysis to calculate the capacities and deformations of the system, and the requirements to produce accurate analyses must be explicitly stated. Second, it is necessary to understand if the software used is capable of correctly modeling the behavior of standard thin-walled and nonsymmetric structural members and systems. This study aims to evaluate existing design standards that include numerical test-based design for both cold-formed steel and other industries. Recommendations for the use of testing by analysis based on the design standards and recent research relevant to testing by analysis are presented. The results of this study will assist with determining recommended requirements for accurate design and testing by analysis.

1. Introduction

Physical testing of cold-formed steel (CFS) members and systems may be technically difficult and can be influenced by many uncertainties, therefore resulting in time and cost inefficiencies. To improve process efficiency and productivity, researchers and engineers have paid increasing attention to testing by analysis, such as by finite element (FE) analysis. As testing by analysis examines the performance of structural members and systems, unclear effects resulting from the uncertainties in the physical testing can be checked in advance.

To reduce costs, virtual testing is beneficial in the initial design phase of new products. It is important to determine the capacities of new shapes being developed, but also to understand how the various elements in the cross-section move and interact. A new product is often designed for a specific use or span, but it is necessary to understand how the new product will behave in other less common loading and structural scenarios.

Testing by analysis can be a good alternative to physical testing since it allows researchers and engineers to reduce the expense and time in performing physical experiments.

In order to perform testing by analysis, a rational engineering judgement is required to determine the capacities of the structures. Although the use of testing by analysis has been increased and computational capability for modeling has been developed in recent years, most standards do not have detailed requirements for design by analysis. Design by analysis must consider all relevant inputs, such as material properties, imperfections, second-order effects, modeling selections, connection effects, and uncertainties.

This paper aims to provide an overview of testing by analysis in both existing cold-formed steel design standards, structural steel design standards and recent research in order to determine which test-based design procedures should be implemented when designing by analysis. The cold-formed steel design standards discussed herein include Chapter C and K of AISI S100-16 [1] which provide requirements for the design for stability and test-based design, Chapter 5 and 9 of Eurocode 3 (EN 1993-1-3) [2] to cover provisions for structural analysis and design by testing, and Appendix B of the Australia / New Zealand standard AS/NZS 4600 [3] that contains provisions for the structural analysis. The discussed structural steel standards for hot-rolled members include Chapter C and Appendix 1 of AISC 360-16 [4] that contain requirements for the design for stability and structural analysis by advanced methods, Chapter 5 of Eurocode 3 (EN 1993-1-1) [5] to describe modeling for structural analysis, Chapter 4 and Appendix D of Australian / New Zealand standard AS/NZS 4100 [6] which provide the requirements

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for the methods of structural analysis and advanced analysis, and Chapter 8 and Annex O of the Canadian standard CSA S16 [7] to cover structural analysis including advanced analysis. Furthermore, EN 1993-1-3 states “For a approach with FE-methods (or others) see EN 1993-1-5, Annex C”, therefore Eurocode 3 Part 1-5: Plated Structural Elements [8] is included. Plated structural elements can be applicable to cold-formed steel members in addition to hot-rolled steel members such as plate girders or slender I-beams. Recommendations for testing by analysis based on current design standards and research is presented.

2. Recommendations

2.1 Material

Numerical modeling requires correct representation of the material stress-strain relationship in order to obtain an accurate prediction of structural responses by considering the material stiffness and effects due to yielding and plasticity. The standards for CFS design, EN 1993-1-3 [2] and AS/NZS 4600 [3], allow the use of nonlinear material stress-strain relationships for advanced analysis. Annex C.6 of EN 1993-1-5 [8] specifies that material properties should be taken as characteristic values and four types of material behavior may be used as illustrated in Figure 1: elastic-plastic without strain hardening, elastic-plastic with a nominal plateau slope, elastic-plastic with linear strain hardening, and true stress-strain curve modified from the test results. True stress and strain are approximated by $\sigma_{true} = \sigma(1 + \epsilon)$ and $\epsilon_{true} = \ln(1 + \epsilon)$, respectively, where σ is stress and ϵ is strain. In addition to these material behaviors, material models recognized for CFS can be adopted [3].

Gardner and Yun in 2018 [9] developed an accurate stress-strain model of CFS described by a two-stage Ramberg-Osgood model. Predictive expressions to model the stress-strain curve were developed based on 700 experimental stress-strain curves, covering a wide range of steel grades, thicknesses, and cross-section types. The accuracy of the proposed model is demonstrated even if only the value of the yield strength is known. As such, this model can be considered as appropriate for use in design by advanced computational analysis.

For design by analysis, it is recommended to consider the nonlinear stress-strain relationships to capture inelastic behavior of structural components or structures. The authors recommend to use the Ramberg-Osgood model proposed by Gardner and Yun [9], which is a straight-forward approach to accurately model cold-formed steel materials.

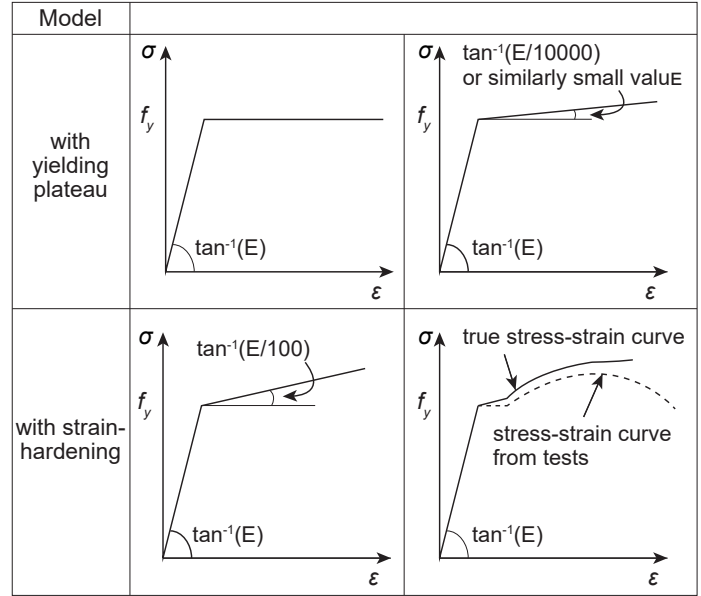


Figure 1: Modeling of material behavior from EN 1993-1-5 [8]

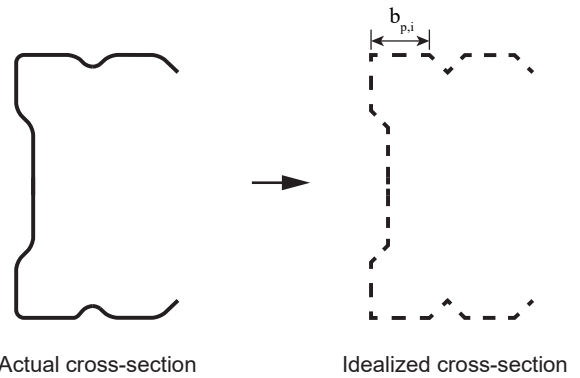


Figure 2: Approximate allowance for rounded corners from EN 1993-1-3 [2]

2.2 Modeling of Cross Section

The cross-section properties affect the analysis of structural members and systems, especially for nonsymmetric cross-sections, and must be correctly accounted for. Section 5.1 of EN 1993-1-3 [2] has provisions for considering the effect of rounded corners when determining section properties. If the internal radius $r \leq 5t$ and $r \leq 0.1b_p$, the rounded corners may be neglected and instead the cross-section can be assumed to consist of sharp corners as shown in Figure 2, where b_p is the notional flat widths measured from the midpoints of the adjacent corner elements. For cross-section stiffness properties, the effect of rounded corners should always be considered.

Liu et al. [10] investigated an improvement on an existing beam-column line element formulations for accurately simulating the axial buckling behavior of arbitrarily-shaped

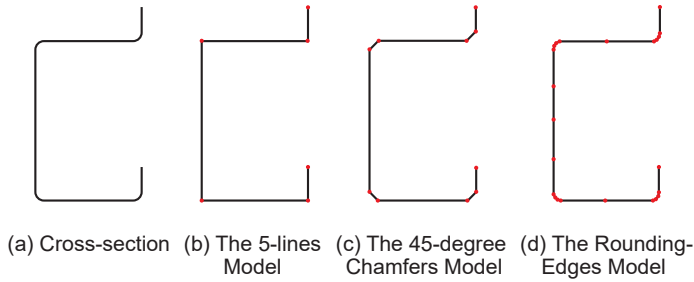


Figure 3: Three cross-section models from Liu et al. [10]

open-sections. One of the asymmetric sections studied was a lipped-C shape consisting of one lip that is turned outward and one inward. To study the effects of the rounded corners on the section properties, three different modeling methods to consider the corners were created as shown in Figure 3. The three cross-section models are established based on line-elements with (1) neglecting the rounded corners (Figure 3b), (2) considering the rounded corners as 45-degree line-elements (Figure 3c), and (3) full consideration of the rounded corners with three elements in a corner (Figure 3d). The module MSA_Sect within MASTAN2 [11] was used to compute the section properties. The section properties generated by CUFISM [12] using the rounding-edges model were employed as the benchmark solution. As shown in Table 1 which displays the results from Liu et al.'s study, the cross-section properties from the rounded corner model were almost identical to the cross-section properties determined from the rounded corner model in CUFISM [12], which is expected. The important comparison is between the sharp corner model and the 45-degree corner chamfer model. The sharp corner model resulted in several cross-section properties with greater than 5% percent error compared to the benchmark properties, whereas the 45-degree chamfers model had less than 4% percent difference for all section properties.

The authors recommend to consider the effects of rounded corners to determine accurate cross-section properties. This can be done using CUFISM [12] for the greatest accuracy, or with 45-degree corner chamfers for a minor reduction in accuracy.

Section 5.2 of EN 1993-1-3 [2] specifies the range of width-to-thickness ratios that apply for structural analysis. These limits represent the ranges that have sufficient experience and verification by testing. Cross sections outside the range of the width-to-thickness ratios may be used when their resistance at ultimate limit states and behavior at serviceability limit states are verified by physical testing and/or by analysis (calculations) with an appropriate number of tests, however, the appropriate number is not stated in the standard.

Table 1: Section properties of asymmetric cross section from Liu et al. [10]

Parameters	Percent difference (%) with the benchmark solution		
	The 5-lines model	The 45-degree chamfers model	The rounding-edges model
A	3.04	-1.01	0.00
I_y	5.84	-2.23	0.00
I_z	3.55	-1.42	0.00
J	2.95	-1.27	0.00
$C_w (I_w)$	8.05	-3.67	-0.15
y_c	-6.01	3.08	0.00
z_c	0.65	-0.48	0.00

Note: A is the cross-section area, I_y and I_z are the second moment of areas about the principal axes, J is the uniform torsional rigidity, $C_w (I_w)$ the uniform torsion warping constant, y_c and z_c are the coordinates of shear center

For the modeling of elements of a cross section, EN 1993-1-3 [2] suggests to follow Annex C of EN 1993-1-5 [8] or to use an approximate modeling of junctions and contribution of stiffeners where the restraining effect of the adjacent plates is simulated by elastic springs at intermediate stiffeners and edge stiffeners. i.e., the rotational and translational springs are used to simulate the stiffening effect of adjacent plates or stiffeners. However, there is no guidance on how to determine the numerical value of the springs.

The boundary conditions for supports, interfaces, and applied loads should be modeled so that obtained results are conservative [8].

2.3 Element Type and Size

The choice of FE-models (shell models or solid models) and the size of mesh determine the accuracy of the analysis results. According to Annex C.1 of EN 1993-1-5 [8], as shown in Table 2, the choice of FE methods depends on the assumptions of linearity/nonlinearity of material and geometric behaviors, and the presence of imperfections. Validation sensitivity checks with successive refinement may be performed.

Shell elements are utilized when the width-to-thickness ratio of elements is greater than 1.7 and solid elements shall have the ratio smaller than 4.0 [13]. Shell elements may be predominantly used for CFS structures because standard CFS cross-sections have the width-to-thickness ratios around 33.3. Multiple previous studies performed FE analysis on CFS members with convergence studies: Buchanan et al. [14] utilized a four-node shell element (S4R) and employed a mesh validation study with the element size varying from $10t$ to $\frac{t}{3}$, where $t = 1.34$ mm is the thickness of the circular hollow section. A size of $t \times t$ shell element was adopted as it yielded accurate failure load and deflection from the finest mesh, $\frac{t}{3}$, while maintaining computational efficiency. Pham [15] used the S4R element with a mesh

Table 2: Assumptions for FE methods from EN 1993-1-5 [8]

Material behavior	Geometric behavior	Imperfections	Example of use
linear	nonlinear	no	critical plate buckling load
linear	nonlinear	yes	elastic plate buckling resistance
nonlinear	nonlinear	yes	elastic-plastic resistance in ultimate limit state

size of 5 mm for 2 mm thick thin-walled channel sections with holes. Keerthan and Mahendran [16] conducted convergence studies and utilized element sizes of 5 mm × 5 mm for 1.5 mm or 1.9 mm thick lipped channel beams with web openings. Pham et al. [17] modeled a shear test of lipped channel beams that have thicknesses varying 1.2 mm to 3.0 mm with a 5 mm mesh. Different mesh sizes were used in the test set-up: 5 mm for the angle straps and 10 mm for other parts of the test set-up such as the stocky column, loading plates, and thick plates. As both the median and mean values of mesh size-to-thickness ratio are 2.6 from the studies covered in this section, the value of 2.6 can be used as the approximate mesh size-to-thickness ratio. Appropriate element sizes would be different based on the geometric properties such as cross-section type and thickness. The authors recommend to perform validation sensitivity checks to determine the mesh size that obtains accurate results or use the mesh size based on the approximate mesh size-to-thickness ratio.

2.4 Geometric Imperfection and Residual Stress

As the pattern and magnitude of geometric imperfections have a significant effect on the structural behavior, correct modeling of the geometric imperfections is necessary to accurately predict the response of the structure. Section C1.1 of AISI S100 [1] addresses that the effect of geometric imperfections shall be considered in the elastic design by using notional loads or directly using initial imperfections. The maximum displacement considered in the design shall be the magnitude of the initial displacements. The inclusion of imperfections is permissible to the analysis for gravity-only load combinations, not for load combinations including applied lateral loads.

Section 5.5 of EN 1993-1-3 [2] provides values of equivalent geometric imperfections, which reflect the possible effects of the imperfections, based on the type of imperfections or analysis. Design value of bow imperfections related to flexural buckling and torsional flexural buckling should be adopted from Table 3 with values based on analysis methods including elastic analysis and plastic analysis and five buckling curves illustrated in Figure 4. The selection of the appropriate buckling curve is based on the type of cross section, axis of buckling, and yield strength used. e.g., back-to-back lipped (or plain) channel sections for buckling about

Table 3: Design value of initial local bow imperfection e_0/L for members from EN 1993-1-1 [5]

Buckling curve	Elastic analysis (e_0/L)	Plastic analysis (e_0/L)
a_0	1/350	1/300
a	1/300	1/250
b	1/250	1/200
c	1/200	1/150
d	1/150	1/100

Note: e_0 is an initial bow imperfections; Buckling curves are illustrated in Figure 4

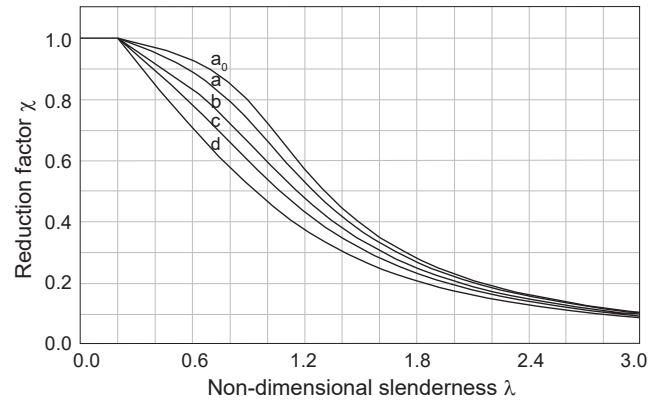


Figure 4: Buckling curves from EN 1993-1-1 [5]

the strong axis and the weak axis apply the buckling curves a and b, respectively. Closed built-up cross sections apply the buckling curve b when using nominal yield strength or the buckling curve c when the average yield strength is utilized. Lipped C and Z sections use the buckling curve b. Any other cross sections are applicable to the buckling curve c. Bow imperfections related to lateral-torsional buckling take $\frac{1}{600}$ for elastic analysis and $\frac{1}{500}$ for plastic analysis. The effects of cross-sectional imperfections should be taken into account when determining the resistance and stiffness of CFS members and sheeting. The effects of distortional buckling should be determined by performing linear or nonlinear buckling analysis using FE methods. Nonlinear buckling analysis is a static method which accounts for material and geometric nonlinearities. The examples of buckling analysis with FE methods are previously given in Table 2.

According to Annex C.5 of EN 1993-1-5 [8] provides equivalent geometric imperfections which may be used if there is an absence of a more refined analysis for the imperfections. Geometric imperfections may be based on the shape of the critical plate buckling modes. For cross-section imperfections, 80% of the geometric fabrication tolerances is recommended. The direction of the imperfection should be chosen which results in the lowest resistance. The equivalent geometric imperfections may be applied to the model with the values in Table 4 and Figure 5. When combining imperfections, a leading imperfection should be selected and

Table 4: Equivalent geometric imperfections from EN 1993-1-5 [8]

Type of imperfection	Component	Shape	Magnitude
global	member with length l	bow	See Table 3
local	panel or subpanel with short span a or b	buckling shape	$\min(a/200, b/200)$
local	stiffener or flange subject to twist	bow twist	$1/50$

Note: See Figure 5 for the notation of a , b and l

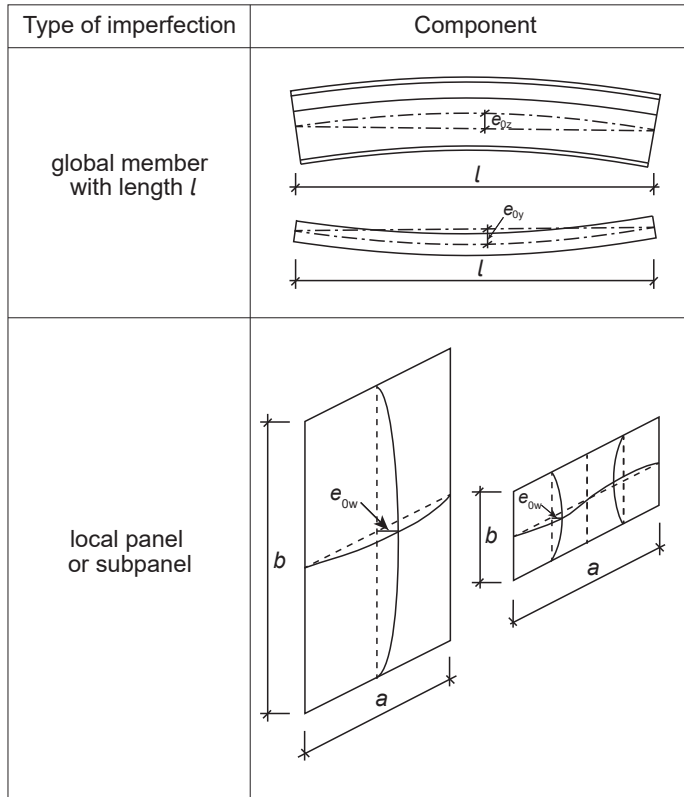


Figure 5: Modeling of equivalent geometric imperfections from EN 1993-1-5 [8]

the accompanying imperfections may have reduced values, 70% of their values. Any type of imperfections can be the leading imperfections or the accompanying imperfections.

Appendix B4 of AS/NZS 4600 [3] recommends including frame, member, and cross-sectional imperfections for the modeling of geometric imperfections. For frame imperfections, an out-of-plumbness ratio of $\frac{1}{500}$ is often adopted as the magnitude of frame imperfections in advanced analysis, or can be accounted for with notional horizontal forces for regular single or multi-story framing structures. For member imperfections, $\frac{1}{1000}$ of the member length shall be the maximum value, which is smaller than $\frac{1}{250}$ that EN 1993-1-3 [2] employs for elastic analysis of lipped C and Z sections. Local and distortional buckling imperfections shall be taken into account in the model by multiplying the local and distortional

buckling modes by a factor. Unit maximum deformation is assumed by imperfection multipliers: the imperfection multiplier for local buckling $s_{ol} (= 0.3t\sqrt{\frac{f_y}{f_{ol}}})$ and the imperfection multiplier for distortional buckling $s_{od} (= 0.3t\sqrt{\frac{f_y}{f_{od}}})$, where t is plate thickness, f_{ol} is elastic local buckling stress, and s_{od} is elastic distortional buckling stress. The scaled imperfections are superimposed onto the perfect geometry. The local and distortional buckling modes may be determined from a linear buckling analysis based on shell FE modeling or finite strip discretization of the member. However, for unbraced pitched roof cold-formed steel portal frames and unbraced cold-formed steel storage racks, local and distortional buckling imperfections are not required to be modeled.

Zeinoddini and Schafer [18] evaluated three methods for simulation of geometric imperfections in CFS members: (1) the Traditional Modal Approach that considers imperfections as a combination of buckling modes; the mode shapes are achieved from an eigenvalue buckling analysis of the member using five cross-sectional buckling mode shapes, (2) the 2D Spectra Approach that considers imperfections as a two-dimensional random field, and (3) 1D Modal Spectral Approach which is a combination of modal and spectral approaches; the spectral approach is used to generate the imperfection magnitudes in the longitudinal direction and the five mode shapes are considered in the transverse direction. A comparison of the simulation results obtained from the three methods shows that the Traditional Modal Approach is conservative for predicting the strength. The 2D Spectra Approach predicts the strength of models that have local and distortional failure with high accuracy, but it is less accurate when the global failure mode is dominant. The 1D Modal Spectral Approach accurately captures the imperfection distributions and the strength, axial flexibility, and failure mechanism of the member, it is thus the most appropriate method for simulation of imperfections in CFS members [18].

In summary, current standards mention three types of geometric imperfections including frame imperfection, member imperfection, and cross-sectional imperfections that should be considered in the analysis in directions that result in the worst case. Frame imperfections can be considered either directly in the structural model or applying notional loads for regular single or multi-story framing structures [1], [3]. Imperfections should be determined based either on actual (measured) imperfections, if known [4], or on equivalent geometric imperfections indicated in the standards. Cross-sectional imperfections can be determined by linear/nonlinear buckling analysis using FE models [2], [3]. The authors recommend that appropriate values for equivalent geometric imperfections for CFS members and structures be developed.

CFS design standards including AISI S100 [1] and EN 1993-

1-3 [2] recommend to consider stiffness reductions due to the effects of residual stresses and partial yielding. AS/NZS 4600 [3] includes stiffness reductions due to cross-section deformations or local and distortional deformations in addition to the effects of residual stresses and partial yielding. AISI S100 includes the influence of residual stresses and partial yielding by using the reduction factor 0.9 and the additional factor τ_b that considers the flexural stiffnesses, whereas AISC 360 [4] applies $0.8\tau_b$ to consider reduced stiffness. Residual stresses shall be modeled indirectly through the stress-strain curve [3] or based on a stress pattern produced by the fabrication process with amplitudes equivalent to the mean (expected) values [2]. As stiffness reductions may result in increased deflections and second-order bending moments, it is recommended to consider the effects that lead to reduced stiffness.

2.5 Second-order Effects

The standards for cold-formed steel (AISI S100 [1] and AS/NZS 4600 [3]) and hot-rolled steel (EN 1993-1-1 [5], AS 4100 [6], and AISC 360 [4]) require/suggest to consider second-order effects in the analysis. AISI S100 [1] considers second-order effects including $P - \Delta$ and $P - \delta$ only. AS 4100 [6] includes second-order effects in the analysis, while the type of second-order effects is not specified. EN 1993-1-1 [5] and AS/NZS 4600 [3] include second-order effects arising from deformed geometry not limited to $P - \Delta$ and $P - \delta$. Appendix 1 of AISC 360 [4] includes geometric nonlinearities such as $P - \Delta$, $P - \delta$, and twisting effects in analysis.

For non-doubly symmetric cross-section members, however, the consideration of only $P - \Delta$ and $P - \delta$ in a second-order analysis is not enough to fully reflect behaviors related to asymmetry [19]. Sippel et al. [19] analyzed the response of non-doubly symmetric cross-section beam members. The analysis results were used to evaluate that the methods can accurately capture behaviors related to asymmetry. The inclusion of only $P - \Delta$ and $P - \delta$ in a second-order analysis is not enough to fully reflect the behavior of non-doubly symmetric sections. The consideration of twisting effects including warping, the center of twist, and second-order twist effects are important to the analysis of non-doubly symmetric cross sections. Moreover, the inclusion of asymmetric cross-section properties such as nonconcentric shear center and centroid affects the analysis results. Sippel and Blum [20] examined the importance of the inclusion of the asymmetric section properties to structural systems with non-symmetric sections formed from cold-formed steel members. Thus, it is recommended to include not only the effects of $P - \Delta$ and $P - \delta$ but also the effects from twisting effects when non-doubly symmetric cross section is analyzed.

2.6 Connections

AISI S100 [1] and AS/NZS 4600 [3] provide requirements for modeling of connections. Connections shall have sufficient strength and ductility to avoid structural failure within the connections and instead ensure that the structure fails within the members. In addition, if connections show nonlinear behavior, it shall be included in the analysis [3]. Connection deformations and uncertainty in connection stiffness and strength shall be considered [1], [3].

Although the CFS design standards [1]–[3] have no classification of type of connection model, they could refer to the hot-rolled steel design standards. For connection modeling, for example, CSA S16 [7] provides three types of connections including simple, rigid, and semi-rigid. The design moment-rotation characteristic of a joint may adopt a simplified curve including a linearized approximation such as bilinear or tri-linear when the simplified curve lies entirely below the design moment-rotation characteristic [5].

Since connections of CFS portal frames, storage racks, and built-up sections used in framing display semi-rigid behavior [21]–[23], the inclusion of semi-rigidity is significant to the modeling of CFS structures. The type of connections can be decided by experimental results or previous experience in similar cases. However, the assumption of a pinned connection in racks or studs seated in track should be avoided because it leads to large displacement which decreases system stability [22], [23]. It is recommended to consider the effects of connection behavior including semi-rigid behavior in analysis.

2.7 Uncertainty

AISI S100 [1], AS/NZS 4600 [3], AISC 360 [4], and CSA S16 [7] include uncertainty in strength and stiffness which affect the behavior of structures in the analytical model. Consideration of uncertainty in the strength and stiffness properties must be modeled to be obtained the most adverse effects on the structure [7]. A reduction factor of 0.9 shall be applied to yield stress and stiffness of all steel members and connections to account for the uncertainty in system, member, and connection strength and stiffness [4]. In addition, AS/NZS 4600 [3] provides capacity reduction factors (ϕ) for the strength and stability limit states of prequalified frames. Values of ϕ are determined from reliability analyses [24], [25]. The frame should support the factored limit states actions multiplied by $\frac{1}{\phi}$, where ϕ is 0.85 for CFS portal frames and 0.9 for steel storage racks.

Test-based design provided by Chapter K of AISI S100 [1] requires structural performance to be established by tests or rational engineering analysis with confirmatory tests. The strength of the tested elements, assemblies, connections, or

members is determined based on the same procedures used to calibrate the LRFD design criteria, as given in Eq. 1. The resistance factor (ϕ) computed by Eq. 2 considers the uncertainty in material and geometric properties, failure mode, and prediction of the resistance,

$$\sum \gamma_i Q_i \leq \phi R_n \quad (1)$$

where γ_i is load factors; Q_i is load effects; and R_n is nominal resistance

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_Q^2}} \quad (2)$$

where C_ϕ is calibration coefficient, β_o is target reliability index, and V_Q is coefficient of variation of load effect. The values are given in AISI S100 [1]. M_m is mean value of material factor, F_m is mean value of fabrication factor, V_M is coefficient of variation of material factor, and V_F is coefficient of variation of fabrication factor. The values are listed in Table K2.1.1.1-1 of AISI S100 [1]. C_P is correction factor, $\frac{(n+1)(n-1)}{n(n-3)}$ for $n \geq 4$ and 5.7 for $n = 3$ in which n is the number of tests not fewer than three; P_m is mean value of professional factor for tested component, $\frac{1}{n} \sum_{i=1}^n \frac{R_{t,i}}{R_{n,i}}$ in which $R_{t,i}$ is tested strength and $R_{n,i}$ is calculated nominal strength; V_P is coefficient of variation of test results; and R_n is average value of all test results.

The correlation coefficient (C_c) between the tested strength and the nominal strength predicted from the rational engineering analysis model shall be greater than or equal to 0.8. The bias and variance between the measured and the nominally specified dimensions and material properties shall be reflected by including fabrication (F_m and V_F) and material (M_m and V_M) factors to the calculation of resistance factor. Uncertainties in material and geometric properties should be considered in analysis because they affect the strength of a member or a structure.

2.8 Benchmark Test

Annex C of EN 1993-1-5 [8], Appendix B of AS/NZS 4600 [3], and Chapter C and Appendix 1 of AISC 360 [4] require performing benchmark tests to prove the software is appropriate for the task. In Appendix 1 of AISC 360 [4], benchmark tests are used to check if the second-order effects resulting from the combination of axial force, flexure, and twist are being correctly performed in elastic analysis. Otherwise, according to Chapter C, benchmark problems are used to verify that $P - \delta$ and $P - \Delta$ second-order analysis used in the direct analysis method provide a confidence level of the task. Benchmark tests can be performed by well-documented experimental results or similar benchmark results [3].

Pham [15] used the finite-strip method as a benchmark test of FE method for elastic buckling analysis and the results from the two methods agree within 2% error. Ziemian et al. [26] performed benchmark problems to ensure that the non-linear analysis of an unbraced I-shaped member subjected to in-plane and out-of-plane loading effects that have significant spatial behavior such as warping and twisting effects achieves accurate results. The benchmark problems are crucial in validating the proper use of nonlinear analysis when the modeling of spatial behavior is important. Overall, it is recommended to perform benchmark tests to validate the accuracy of the software.

2.9 Dimension: 2D or 3D

Annex O of CSA S16 [7] and Appendix B of AS/NZS 4600 [6] include provisions for dimension of the model. CSA S16 [7] requires using a three-dimensional model, but a two-dimensional model can be employed providing that the use of model is validated for design. For the use of two-dimensional model, it is required to consider the out-of-plane response. AS/NZS 4600 [3] addresses the case of using a two-dimensional model without provisions for using three-dimensional analysis. A two-dimensional model can be used for analyzing regular building structures by considering them as a series of parallel two-dimensional substructures. The analysis should be carried out in two directions at right angles. However, the use of two-dimensional analysis is not applicable to structures that have significant load redistribution between the substructures. As it is important to consider the spatial behavior in analysis [19], [26], the authors recommend to employ a three-dimensional model to achieve correct structural responses.

2.10 Other Recommendations

EN 1993-1-5 [8] suggests to document details of the analytical model including the mesh size, loading, boundary conditions, and other input/output data to be reproduced by third parties. To implement design by analysis, the authors recommend the information of analytical model and analysis results to be documented.

3. Conclusion

Testing by analysis can compensate for the limitations of physical testing such as high cost and time. This study discussed the literature review of design standards for cold-formed steel structures and other industries that include testing by analysis requirements. In addition, a state-of-the-art review of selected research studies on testing by analysis is presented. Overall, recommendations on the use of testing by analysis to cold-formed steel design with regard to material, modeling of cross section, element type and size, imperfection, second-order effects, uncertainty, dimensions,

benchmark test, connection, and reproduction of physical testing are provided. The recommendations will be helpful for design and testing by analysis of cold-formed steel structures.

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