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Review of Testing by Analysis for Potential Implementation into AISI Standards

Committee on Specifications for the Design of Cold-Formed Steel Structural Members

research report

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Committee on Specifications
for the Design of Cold-Formed
Steel Structural Members

American Iron and Steel Institute

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PREFACE

Testing by analysis can compensate for the limitations of physical testing such as high cost and time. This project 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 and a survey for understanding the current commonly used software and software capabilities are 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, and connection are provided.

Review of Testing by Analysis for Potential Implementation into AISI Standards

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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. Although the computational capability for testing by analysis has been developed in recent years, the current US design code for cold-formed steel, AISI S100, lacks a standardized approach. This project 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, a survey for understanding the current commonly used software and software capabilities, and recent research relevant to testing by analysis are presented. The results of this report will assist with potential future codification of testing by analysis in the AISI standards.

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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 project aims to provide an overview of testing by analysis in existing cold-formed steel design standards, structural steel and concrete design standards, recent research in order to determine which test-based design procedures should be implemented into AISI standards. In addition, a survey was conducted to investigate which software and software capabilities are mostly considered for design of structures. The cold-formed steel design standards discussed herein include Chapter C and K of AISI S100-16 [5] 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 [6] 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 [7] that contain requirements for the design for stability and structural analysis by advanced methods, Chapter 5 of Eurocode 3 (EN 1993-1-1) [4] to describe modeling for structural analysis, Chapter 4 and Appendix D of Australian / New Zealand standard AS/NZS 4100 [8] which provide the requirements for the methods of structural analysis and advanced analysis, and Chapter 8 and Annex O of the Canadian standard CSA S16 [9] to cover structural analysis including advanced analysis. The discussed structural concrete standard includes Chapter 6 of ACI 318 [10]. 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 [1] 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. It was explored if timber design standards including AITC [11], ANSI/AWC [12], and ANSI/TPI [13] have design by analysis rules, but no specific requirements for testing by analysis was found. Recommendations for testing by analysis based on current design standards, research, and the survey is presented.

2. Survey

A survey was carried out to investigate which software programs widely used for design of CFS structures or structures of other materials. The survey was distributed to the Committee on Specifications (COS) and Committee on Framing Standards (COFS) mailing lists through an email from the AISI account and the AISI Steel Industry Code Forum members.

2.1. Survey Form

Software survey

This short survey is part of an AISI small project fellowship to evaluate the possibility of codifying testing by analysis in the AISI standards, which is supported by COS Subcommittee 06 – Test Based Design. This survey is beneficial to understanding the current commonly used software and software capabilities.

1. How do you identify professionally? (select all that apply)

- ☐ Structural engineer (Industry)
- ☐ Civil engineer (Industry)
- ☐ Structural engineer (Academia)
- ☐ Civil engineer (Academia)
- ☐ I have a SE license
- ☐ I have a PE license
- ☐ I have an EIT certification

The following question lists a series of structural analysis tools. For each program, please check the first column if you use the software for the design of cold-formed steel structures, check the second column if used for the design of structures of other materials, or check both columns if used for both purposes.

2. Select all of the software programs that you use for the design of structures from the following list. Please check the first column if you use the software for cold-formed steel structures, check the second column if used for structures of other materials, or check both columns if used for both purposes. (Sorted by developer's name A to Z)

   In this report, the list of the software programs is shown in Table 2.1 in Section 2.3

3. If you answered "Other" for the software program in the previous question, please provide additional information.

4. When working with the software indicated above which of the following analysis types and/or features do you utilize?

☐ Bi-axial bending
☐ Buckling (local-torsional buckling, post-buckling, global buckling, etc)
☐ Dynamic analysis
☐ Dynamic loading (wind, earthquake, vehicle, etc)
☐ Fatigue analysis
☐ Second-order effects ($P - \delta$, $P - \Delta$)
☐ Plastic analysis
☐ Rigid/semi-rigid link
☐ Shear center offset
☐ Static analysis
☐ Thermal effect
☐ Torsion
☐ Warping
☐ Web crippling
☐ Other

5. Please provide any details about how you select which software to use based on the capabilities

of the software.

6. Please list any relevant information below.

2.2. Survey Results: Respondents

Fifty-two responses were obtained from the survey. Respondents are structural engineers in the industry (71%) or academia (7.7%), or civil engineer (3.8%), or industry association manager (3.8%). 7.7% are a structural engineer in both industry and academia, 3.8% are both structural engineers and civil engineers in industry, and 2% are mechanical engineers in a structural engineering position.

2.3. Survey Results: Software

The survey responses and the list of software programs are summarized in Table 2.1. The percentage represents the number of responses for the software (n) divided by the total number of responses, $\frac{n}{52} \times 100(\%)$. It was allowed to select multiple software on the list. The survey responses show that in-house excel or Mathcad files are the most commonly used as a design program for both CFS structures (40%) and other materials structures (60%) except the software program CFS 12 (65%) for CFS design. Using in-house software composed 37% and 21% for CFS and other materials, respectively. According to the responses, in many companies, in-house software and computer code have been developed to have full automation (optimization) and customization required by design codes. Also, in-house Excel spreadsheets and Mathcad programs developed specifically for the products offered by the company on a regular basis. In the situation when a CFS section is not covered by the in-house programs or the results of the in-house programs must be validated, other software programs could be used. However, for commercial software, licensing is always a big issue and the software is typically expensive, thereby in-house software or code are commonly used.

Besides in-house software, the software widely used for CFS structures are CFS 12 (65%), CUFSM (35%), CFS Designer (33%), AISIWIN (21%), Revit (15%), RISA-2D (15%), and MASTAN2 (15%). For the design of structures of other materials, Revit (25%), RISA-2D (23%), RISA-3D (23%), RAM Structural Systems (21%), SAP2000 (19%), and MASTAN2 (19%) are commonly used. The use of software for the design of CFS structures are concentrated on the first three programs (CFS 12, CUFSM, and CFS Designer) due to their applicability to CFS members. For other materials, the top-ranked software programs are utilized with the almost same percentages which range from 19% to 25%. According to the responses, CFS 12 is used for basic CFS section calculation, CUFSM is used for research projects, and MASTAN2 is used for frame analysis. From the overall responses, software needs to be inexpensive, fast, accurate, and user friendly. It should be able to produce code-compliant results and concise reports and handle different shapes or custom CFS shapes.

Table 2.1: Design software used for cold-formed steel design or other materials

No.	Developer	Software	CFS	Other materials
1	ADINA	ADINA Structures	0%	2%
2	ANSYS	ANSYS	6%	8%
3	Applied Science International	SteelSmart System	10%	0%
4	ATIR Engineering Software	STRAP	2%	0%
5	Autodesk	Inventor Nastran	6%	8%
6	Autodesk	Revit	15%	25%
7	Bentley Systems	RAM Connection	4%	15%
8	Bentley Systems	RAM Elements	4%	13%
9	Bentley Systems	RAM Structural Systems	8%	21%
10	Bentley Systems	STAAD.pro	2%	8%
11	CSI	ETABS	2%	13%
12	CSI	Perform3D	0%	6%
13	CSI	SAP2000	8%	19%
14	Dassault Systemes	Abaqus	10%	10%

15	Dassault Systemes	SOLIDWORKS	6%	0%
16	Design Systems	Industry 4.0	0%	0%
17	Dlubal Software	RFEM	2%	0%
18	Dlubal Software	RSTAB	0%	2%
19	Dlubal Software	SHAPE-THIN	2%	0%
20	ENERCALC	Structural Engineering Library	4%	15%
21	FRAMECAD	FRAMECAD Structure	2%	0%
22	Georgia Tech	SABRE2	0%	0%
23	IES	ShapeBuilder	8%	8%
24	IES	Visual Analysis	12%	10%
25	JFBA	Truss Design & Estimating	2%	0%
26	JFBA	WallPanelPro	0%	0%
27	Johns Hopkins University	CUFSM	35%	6%
28	Keymark	Keymark Software Suite	0%	0%
29	RISA Technologies	RISA-3D	12%	23%
30	RISA Technologies	RISA -2D	15%	23%
31	RISA Technologies	RISAConnection	0%	10%
32	RISA Technologies	RISAFloor	0%	10%
33	RISA Technologies	RISASection	2%	4%
34	RSG Software	CFS 12	65%	0%
35	Simpson Strong Tie	AISIWIN	21%	0%
36	Simpson Strong Tie	CFS Designer	33%	0%
37	Simpson Strong Tie	Yield-Link	0%	0%
38	Strand7 Software Development	STRAND7	0%	0%
39	StructSoft Solutions	MWF pro metal	6%	0%
40	Trimble Solutions	Tekla Structural Designer	0%	4%
41	Trimble Solutions	Tekla Tedds	0%	4%
42	UC Berkeley	OpenSees	6%	8%
43	University of Lisbon	GBTUL	0%	0%
44	R. Ziemian and W. McGuire	MASTAN2	15%	19%
45	In-house Excel or Mathcad Files		40%	60%
46	In-house software		37%	21%

Note: The highly ranked software for CFS design are colored— blue: higher than 50%; green: higher than 30%; red: higher than 10%

2.4. Survey Results: Software Capabilities

The survey investigated which software capabilities are considered when using software. The survey responses are summarized in Table 2.2. The listed capabilities can be categorized into analysis types and features. For analysis types, static analysis (88%), dynamic analysis (35%), and plastic analysis (23%) are selected in descending order. The features related to geometric imperfections and deformations were highly selected — 90% for buckling, 65% for web crippling, and 58% for torsion. Features that influence internal forces of structures composed high rates with 69% for bi-axial bending and 65% for second-order effects. In addition to the listed capabilities, nonlinear analysis, time-dependent effects, and structural members with non-uniform elements are considered in the analysis. The responses indicate the importance of inclusion of geometric imperfections and second-order effects in analysis.

Table 2.2: Software capabilities (analysis types and features)

No.	Software capability (analysis types and features)	%
1	buckling	90%
2	static analysis	88%
3	bi-axial bending	69%
4	second-order effects ($P - \delta$, $P - \delta$)	65%
4	web crippling	65%
6	torsion	58%
7	dynamic loading	48%
8	connector effect (rigid/semi-rigid)	37%
8	shear center offset	37%
10	dynamic analysis	35%
11	warping	31%
12	plastic analysis	23%
13	thermal effect	19%

3. Recommendations

This chapter describes recommendations for testing by analysis that can be considered for adoption to AISI, based on existing design standards, recent research, and the results of the survey.

3.1. 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 [6], allow the use of nonlinear material stress-strain relationships for advanced analysis. Annex C.6 of EN 1993-1-5 [1] specifies that material properties should be taken as characteristic values and four types of material behavior may be used as illustrated in Figure 3.1: 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 [6, 14].

Gardner and Yun in 2018 [15] 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

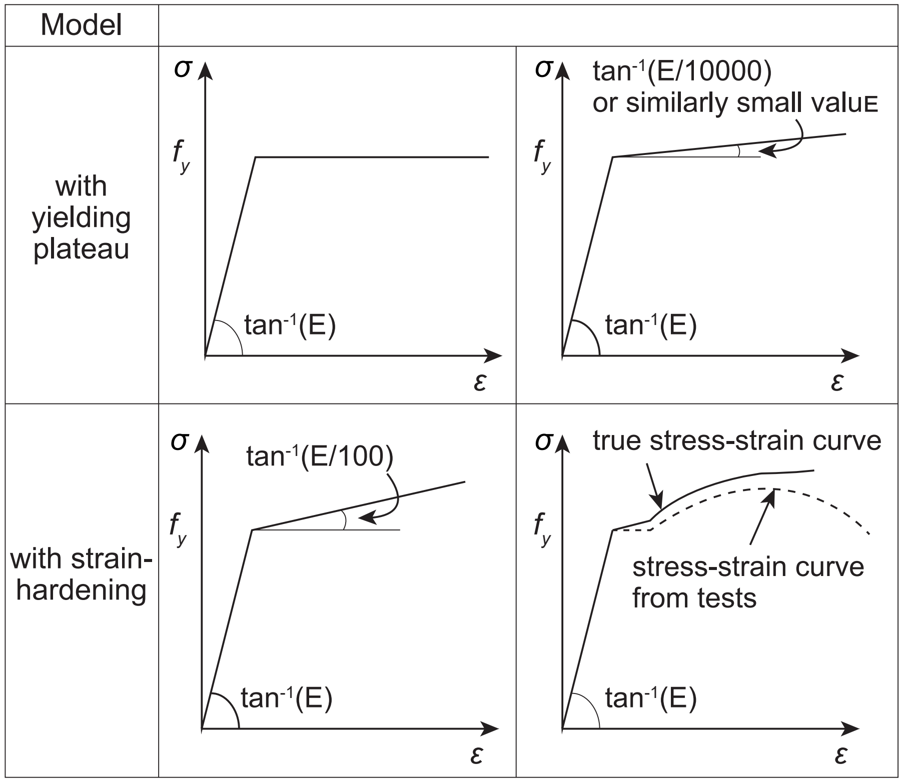


Figure 3.1: Modeling of material behavior from EN 1993-1-5 [1]

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 [15], which is a straight-forward approach to accurately model cold-formed steel materials.

3.2. 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

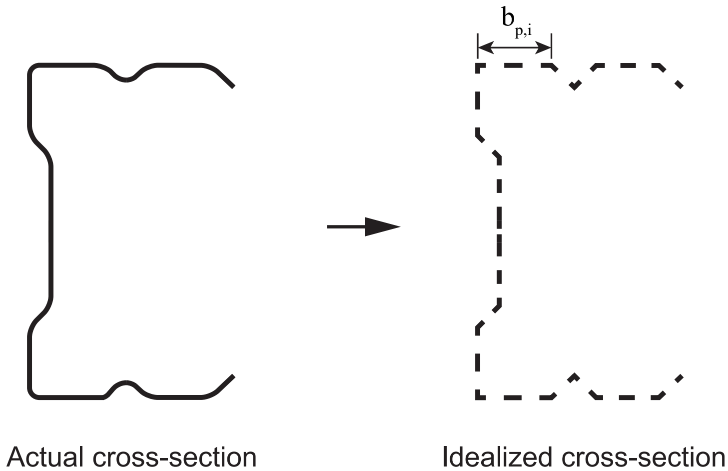
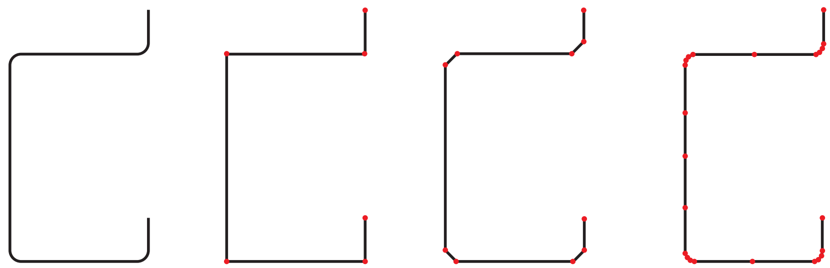


Figure 3.2: Approximate allowance for rounded corners from EN 1993-1-3 [2]

1993-1-3 [2] has provisions for considering the effect of rounded corners when determining section properties. If the internal radius $r \le 5t$ and $r \le 0.1b_p$, the rounded corners may be neglected and instead the cross-section can be assumed to consist of sharp corners as shown in Figure 3.2, 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. [3] investigated an improvement on an existing beam-column line element formulations for accurately simulating the axial buckling behavior of arbitrarily-shaped 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 3.3. The three cross-section models are established based on line-elements with (1) neglecting the rounded corners (Figure 3.3b), (2) considering the rounded corners as 45-degree line-elements (Figure 3.3c), and (3) full consideration of the rounded corners with three elements in a corner (Figure 3.3d). The module MSA_Sect within MASTAN2 [16] was used to compute the section properties. The section properties generated by CUFSM [17] using the rounding-edges model were employed as the benchmark solution. As shown in Table 3.1 which displays the results from Liu et al.'s study, the cross-section properties



(a) Cross-section (b) The 5-lines Model (c) The 45-degree Chamfers Model (d) The Rounding-Edges Model

Figure 3.3: Three cross-section models from Liu et al. [3]

Table 3.1: Section properties of asymmetric cross section from Liu et al. [3]

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

from the rounded corner model were almost identical to the cross-section properties determined from the rounded corner model in CUFSM [17], 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.

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.

For the modeling of elements of a cross section, EN 1993-1-3 [2] suggests to follow Annex C of EN 1993-1-5 [1] 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.

For modeling of rounded corners, the authors recommend to consider the effects of rounded corners to determine accurate cross-section properties. This can be done using CUFSM [17] for the greatest accuracy, or with 45-degree corner chamfers for a minor reduction in accuracy. The boundary conditions for supports, interfaces, and applied loads should be modeled so that obtained results are conservative [1].

3.3. 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. Chapter 6 of ACI 318 [10] requires using the element type that obtains the response required from the task and the mesh size capable of determining the full structural response in detail. Section 3 of AS 4084 [14] suggests to use shell finite elements or finite strips for modeling of storage racks. According to Annex C.1 of EN 1993-1-5 [1], as shown in Table 3.2, the choice of FE methods depends on the assumptions of linearity/nonlinearity of material and

Table 3.2: Assumptions for FE methods from EN 1993-1-5 [1]

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

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 [18]. 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 with convergence studies on CFS members using a four-node shell element (S4R): Theofanou et al. [19] modeled stainless steel oval hollow sections that have thicknesses between 1.9 mm and 3.2 mm, with the mesh size-to-thickness ratio varying from 4 to 10.3. As the thickness of the cross section increased, the mesh size decreased. Natario et al. [20] developed FE models for 4.73 mm thick plain channel section with the mesh size-to-thickness ratio of 1.8 for the flange and 3.2 for the web. Keerthan and Mahendran [21] utilized the element size of 5 mm $\times$ 5 mm for 1.5 mm or 1.9 mm thick lipped channel beams with web openings. Pham [22] used a mesh size of 5 mm for 2 mm thick channel sections. Buchanan et al. [23] employed FE analysis of 1.34 mm thick circular hollow sections. A size of $t \times t$ shell element was adopted which led to 1.0 as the mesh size-to-thickness ratio. Pham et al. [24] modeled a shear test of lipped channel beams that have thicknesses varying 1.2 mm to 3.0 mm with a mesh size of 5 mm. 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 the mean value of the mesh (four-node shell element) size-to-thickness ratio is 4.4 from the studies covered in this section, the value of 4.4 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.

3.4. 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 [5] states 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 3.3 with values based on analysis methods including elastic analysis and plastic analysis and five buckling curves illustrated in Figure 3.4. 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 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

Table 3.3: Design value of initial local bow imperfection e_0/L for members from EN 1993-1-1 [4]

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 3.4

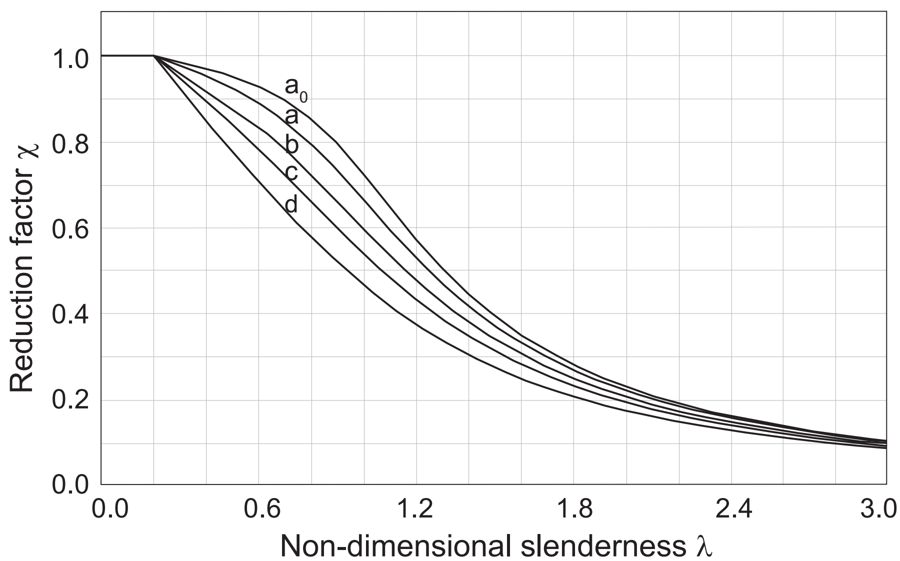


Figure 3.4: Buckling curves from EN 1993-1-1 [4]

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 3.2.

Table 3.4: Equivalent geometric imperfections from EN 1993-1-5 [1]

Type of imperfection	Component	Shape	Magnitude
global	member with length l	bow	See Table 3.3
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 3.5 for the notation of a, b and l

According to Annex C.5 of EN 1993-1-5 [1] 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 3.4 and Figure 3.5. When combining imperfections, a leading imperfection should be selected and 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 [6] and Section 3 of AS 4084 [14] recommend 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

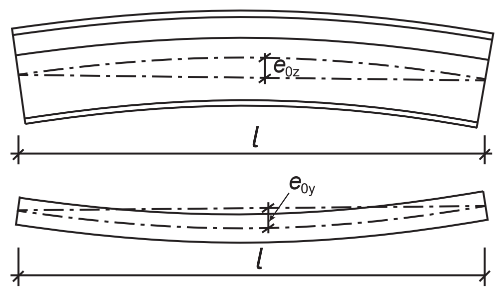
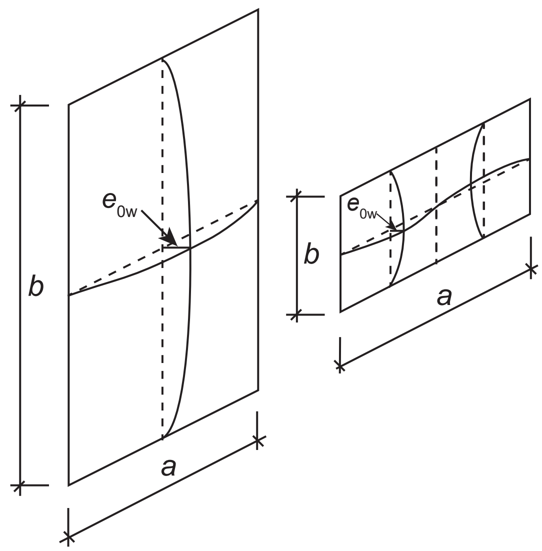
Type of imperfection	Component
global member with length l	
local panel or subpanel	

Figure 3.5: Modeling of equivalent geometric imperfections from EN 1993-1-5 [1]

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 [25] 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 [25].

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 [6, 5]. Imperfections should be determined based either on actual (measured) imperfections, if known [7], or on equivalent geometric imperfections indicated in the standards. Cross-sectional imperfections can be determined by linear/nonlinear buckling analysis using FE models [2, 6]. The authors recommend that appropriate values for equivalent geometric imperfections for CFS members and structures be developed.

CFS design standards including AISI S100 [5] and EN 1993-1-3 [2] recommend to consider stiffness reductions due to the effects of residual stresses and partial yielding. AS/NZS 4600 [6] 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 [5] 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 [7] applies $0.8\tau_b$ to consider reduced stiffness. Residual stresses shall be modeled indirectly through the stress-strain curve [6] or based on a stress pattern produced by the fabrication process with amplitudes equivalent to the mean (expected) values [2].

Moen et al. [26] provided a method for predicting initial residual stresses in cold-formed steel members. The proposed method considers residual stresses resulting from two manufacturing processes including (1) sheet coiling, uncoiling, and flattening, and (2) cross-section roll-forming. Equations for predicting the through-thickness residual stress in corner and flat regions regarding the manufacturing processes are derived based on experimental results. The experimental results showed that corners have larger residual stresses than the flats. The equations of residual stresses resulting from sheet coiling, uncoiling, and flattening includes longitudinal residual stresses only while the equations for cross-section roll-forming predict the transverse and longitudinal residual stresses. Residual stresses can be considered in FE models by directly applying suitable equations for the task, which are classified according to corner or flat regions and the manufacturing processes. 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.

3.5. Second-order Effects

The standards for cold-formed steel (AISI S100 [5] and AS/NZS 4600 [6]) and hot-rolled steel (EN 1993-1-1 [4], AS 4100 [8], and AISC 360 [7]) require/suggest to consider second-order effects in the analysis. AISI S100 [5] considers second-order effects $P-\Delta$ and $P-\delta$ only. AS 4100 [8] includes second-order effects in the analysis, while the type of second-order effects is not specified. EN 1993-1-1 [4] and AS/NZS 4600 [6] include second-order effects arising from deformed geometry not limited to $P-\Delta$ and $P-\delta$. Appendix 1 of AISC 360 [7] includes geometric nonlinearities such as $P-\Delta$, $P-\delta$, and twisting effects. Section 3 of AS 4084 [14] considers twist rotations and torsional internal actions including warping torsion in the 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 [27]. Sippel et al. [27] 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 [28] examined the importance of the inclusion of the asymmetric section properties to structural systems with non-symmetric sections formed from cold-formed steel members. 65% of the survey responses indicates that second-order effects should be performed in software. Additionally, torsion (58%), shear center offset (37%), and warping (31%) should be considered in analysis. 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.

3.6. Connections

AISI S100 [5] and AS/NZS 4600 [6] 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 [6]. Connection deformations and uncertainty in connection stiffness and strength shall be considered [6, 5].

Although the CFS design standards [2, 6, 5] have no classification of type of connection model, the authors recommend that the CFS design standards refer to the hot-rolled steel design standards. For connection modeling, for example, CSA S16 [9] 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 bi-linear or tri-linear when the simplified curve lies entirely below the design moment-rotation characteristic [4].

Since connections of CFS portal frames, storage racks, and built-up sections used in framing display semi-rigid behavior [29, 30, 31], the inclusion of semi-rigidity is significant to the modeling of CFS structures and AS 4084 [14] suggests to account for semi-rigidity of connections in storage racks. 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 [30, 31].

For the modeling of steel connections, Zhu et al. [32] proposed a generalized component model that predicts the full range behavior of the steel connections including the post-ultimate and post-fracture ranges. The method can be used to analyze multiple spring models and applicable to all types of steel connections. 37% of the survey responses addressed that connector effect including semi-rigidity is utilized in using the software. It is recommended to consider the effects of connection behavior including semi-rigid behavior in analysis.

3.7. Uncertainty

AISI S100 [5], AS/NZS 4600 [6], AISC 360 [7], and CSA S16 [9] 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 obtain the most adverse effects on the structure [9]. 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 [7]. In addition, AS/NZS 4600 [6] provides capacity reduction factors (ϕ) for the strength and stability limit states of prequalified frames. Values of ϕ are determined from reliability analyses [33, 34]. 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 [5] 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. 3.1. The resistance factor (ϕ) computed by Eq. 3.2 considers the uncertainty in material and geometric properties, failure mode, and prediction of the resistance,

$$\sum \gamma_i Q_i \leq \phi R_n \tag{3.1}$$

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

$$\phi = C_\phi (M_m F_m P_m) e^{-\beta_o \sqrt{{V_M}^2 + {V_F}^2 + C_P {V_P}^2 + {V_Q}^2}} \tag{3.2}$$

where

C_ϕ = calibration coefficient

β_o = target reliability index

V_Q = coefficient of variation of load effect, the values are given in AISI S100 [5].

M_m = mean value of material factor

F_m = mean value of fabrication factor

V_M = coefficient of variation of material factor

V_F = coefficient of variation of fabrication factor, the values are listed in Table K2.1.1.1-1 of AISI S100 [5].

C_P = 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 = 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 = coefficient of variation of test results

R_n = 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. The authors recommend to consider uncertainties in material and geometric properties because they affect the response of a member or a structure.

3.8. Benchmark Test

Annex C of EN 1993-1-5 [1], Appendix B of AS/NZS 4600 [6], and Chapter C and Appendix 1 of AISC 360 [7] require performing benchmark tests to prove the software is appropriate for the task. In Appendix 1 of AISC 360 [7], 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 [6].

Pham [22] 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. [35] performed benchmark problems to ensure that the nonlinear 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. The survey responses addressed that in-house excel or software is the most widely used software for structural design. The common use of in-house software emphasizes the necessity of benchmark tests for the validation of the software. Overall, it is recommended to perform benchmark tests to validate the accuracy of the software and the authors recommend AISI to develop requirements for employing benchmark tests.

3.9. Dimension: 2D or 3D

Annex O of CSA S16 [9] and Appendix B of AS/NZS 4600 [8] include provisions for dimension of the model. CSA S16 [9] 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 [6] 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 [35, 27], the authors recommend to employ a three-dimensional model to achieve correct structural responses.

3.10. Superposition Principle

ACI 318 [10] does not allow the use of the linear superposition principle, which considers the net response as the sum of the individual responses. e.g., when determining the ultimate inelastic response of a member, it is incorrect to analyze for service loads then combine the results linearly using load factors. A separate inelastic analysis will be performed for each factored load combination. The authors recommend not to use the superposition principle as it would result in different responses from the actual responses.

3.11. Documentation of Results

EN 1993-1-5 [1] 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 the analytical model and analysis results to be documented.

4. Selected Recent Research

This chapter introduces three studies regarded as good examples of FE modeling. While Abaqus [36] was used in the three studies, other finite element software packages with shell elements such as Ansys, SAP2000, RISA, Visual Analysis, etc. could be used. Mastan2 [16] with line elements can accurately model the behaviors related to non-symmetric cross-sections, however the line elements are not capable of directly considering local or distortional buckling [3, 27].

4.1. Buchanan et al. (2020)

Buchanan et al. [23] conducted a numerical investigation of experiments on ferritic stainless steel circular hollow section beam-columns subjected to combined axial loading and bending moment. More than 2,000 simulations employing Abaqus [36] were generated to carry out a parametric study covering austenitic, duplex, and ferritic grades of stainless steel and a wide range of cross section, member slenderness, and applied loading eccentricities, while only 26 beam-column tests were carried out.

The FE models utilized the 4-node doubly curved shell element (S4R). A mesh validation study was performed with the element size varying from $10t$ to $\frac{t}{3}$, where $t = 1.34$ mm is the thickness. 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. In addition, computational efficiency was increased by modeling half of the cross section and employing symmetrical boundary conditions. The FE models utilized the stress-strain relationships obtained from the measured tensile coupons and compressive stub column responses. The effect of membrane residual stresses was neglected while the through-thickness residual stresses are implicitly considered by using measured material properties. The modeling of boundary conditions followed the test conditions. The form of local and global geometric imperfections adopted the lowest local and global buckling mode shapes from an elastic buckling analysis. The amplitudes utilized were the measured mid-point global imperfections (ω_o) and $\frac{L}{1000}$ for global imperfections and $\frac{t}{10}$ and $\frac{t}{100}$ for local imperfections, where L is the effective length and t is the section thickness. In order to validate the numerical models from the experimental results, various amplitudes of imperfections and material properties including compressive and tensile properties were used.

The numerical models were validated by comparing the ultimate load and the mid-height lateral deformation at the ultimate load. The predicted values of ultimate load and lateral deflection were within 5% error against the measured values using the compressive material properties, whereas beyond 10% error occurred when the tensile coupon properties were utilized. This demonstrates the importance of using the proper material models for your analysis, and therefore the models with compressive material properties were adopted for the parametric study. The developed models were used to evaluate the existing beam-column design code, EN 1993-1-4 [37].

4.2. Pham et al. (2020)

Pham et al. [24] developed FE models using Abaqus [36] to validate against shear tests of cold-formed steel channel sections with both small and large web holes. A parametric study was performed to extend the experimental database. This study proposed a new Direct Strength Method of design of perforated channels in shear that can be applicable to a wider range of sectional dimensions and thicknesses.

In the FE models, the S4R element with a mesh size of 5 mm was used for the cold-formed channel sections while the 8-node linear solid element (C3D8R) was used for the test set-up with a mesh size of 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. The area surrounding the web openings adopted sweep meshing. The modeling of boundary conditions and connections followed the actual tests. The nonlinear behavior of the bolted connection was included in the analysis by using the nonlinear elastic properties obtained from the test results. For the material properties, the true stress-strain curve as previously shown in Figure 3.1 was adopted with measured stress and strain from the tensile coupon tests. The initial geometric imperfections were specified by the buckling modes with the lowest eigenvalue. Two scaling factors for the imperfection, $0.15t$ [38] and $0.64t$ [39], were employed as the imperfection amplitudes.

The ultimate shear strengths produced by the FE models and the actual tests were compared and the maximum percent difference was 5.37%. Moreover, the FE models produced similar shear failure modes with the tests. It was proved that the developed FE models properly simulate the actual tests.

4.3. Kyvelou et al. (2018)

Kyvelou et al. [40] developed FE models of composite flooring systems comprising cold-formed steel channel section beams with two stiffeners and wood-based particle boards using Abaqus [36]. One hundred simulations were generated and the simulation results were validated against twelve physical test results. A parametric study was conducted to investigate the effect of key parameters on the performance of the flooring systems including the depth and thickness, and the spacing of fasteners.

The material model adopted the two-stage Ramberg-Osgood model proposed by Gardner and Ashraf [41]. This study carried out corner coupon tests and it was revealed that the corner regions have 17% higher yield strength than the flat regions. The strength enhancements in the corner regions were considered by assigning different material properties. The effect of through-thickness residual stresses was implicitly included in the stress-strain curves. The S4R shell elements with a longitudinal size of 10 mm were chosen for the modeling of the CFS beams. The C3D8R solid elements with a longitudinal size of 10 mm were used to model the wood-based flooring panels. The self-drilling screws acting as the shear connection between the joists and the flooring panels were modeled with nonlinear spring elements that consider the load-slip response based on the push-out test results. For modeling of initial geometric imperfections, the pure local and distortional buckling mode shapes were obtained from CUFSM [17]. The obtained buckling modes were distributed longitudinally, through sinusoidal functions, and the deformed geometry was directly modeled in Abaqus [36] as the initial imperfections. The scaling factors for the local and distortional buckling mode shapes, $0.1t$ [42] and $0.3t$ [43], respectively, were employed as the imperfection magnitudes.

The ultimate moment capacities and flexural stiffnesses were compared to confirm if the developed FE models accurately predicted the test results. The mean ratios of predicted to tested results for moment capacities and flexural stiffnesses were 0.99 and 1.04, respectively. In addition, the FE models accurately predicted the exhibited failure modes, load-displacement responses, and strain distributions at the ultimate load. It was ensured that the FE models can be employed in the parametric study to examine the influence of the key parameters on the structural behavior of the flooring systems examined in the study.

5. Conclusion

Testing by analysis can compensate for the limitations of physical testing such as high cost and time. This project 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 and a survey for understanding the current commonly used software and software capabilities are 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, and connection are provided. The recommendations will be helpful for possible future codification of test-based design in the AISI standards which currently have no provisions for testing by analysis.

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