

**Review of Testing by
Analysis for Potential
Implementation into AISI
Standards**

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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.

1 **Review of Testing by Analysis for Potential**
2 **Implementation into AISI Standards**

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8 **Abstract**

9 New product development is crucial to allow innovation in the cold-formed steel structural
10 industry. However, the required physical testing of new components and assemblies are often a cost
11 barrier which prevents implementation and slows new product development. Testing by analysis
12 can be a good alternative to physical testing as it reduces the expense and time for performing
13 physical experiments, however, two considerations are necessary to ensure accurate results. First, it
14 requires a rational engineering analysis to calculate the capacities and deformations of the system,
15 and the requirements to produce accurate analyses must be explicitly stated. Second, it is necessary
16 to understand if the software used is capable of correctly modeling the behavior of standard thin-
17 walled and nonsymmetric structural members and systems. Although the computational capability
18 for testing by analysis has been developed in recent years, the current US design code for cold-
19 formed steel, AISI S100, lacks a standardized approach. This project aims to evaluate existing design
20 standards that include numerical test-based design for both cold-formed steel and other industries.
21 Recommendations for the use of testing by analysis based on the design standards, a survey for
22 understanding the current commonly used software and software capabilities, and recent research
23 relevant to testing by analysis are presented. The results of this report will assist with potential future
24 codification of testing by analysis in the AISI standards.

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62 **1. Introduction**

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

69 To reduce costs, virtual testing is beneficial in the initial design phase of new products. It is
70 important to determine the capacities of new shapes being developed, but also to understand how the
71 various elements in the cross-section move and interact. A new product is often designed for a specific
72 use or span, but it is necessary to understand how the new product will behave in other less common
73 loading and structural scenarios. Testing by analysis can be a good alternative to physical testing since
74 it allows researchers and engineers to reduce the expense and time in performing physical experiments.
75 In order to perform testing by analysis, a rational engineering judgement is required to determine
76 the capacities of the structures. Although the use of testing by analysis has been increased and
77 computational capability for modeling has been developed in recent years, most standards do not have
78 detailed requirements for design by analysis. Design by analysis must consider all relevant inputs,
79 such as material properties, imperfections, second-order effects, modeling selections, connection
80 effects, and uncertainties.

81 This project aims to provide an overview of testing by analysis in existing cold-formed steel
82 design standards, structural steel and concrete design standards, recent research in order to determine

83 which test-based design procedures should be implemented into AISI standards. In addition, a survey
84 was conducted to investigate which software and software capabilities are mostly considered for
85 design of structures. The cold-formed steel design standards discussed herein include Chapter C and
86 K of AISI S100-16 [5] which provide requirements for the design for stability and test-based design,
87 Chapter 5 and 9 of Eurocode 3 (EN 1993-1-3) [2] to cover provisions for structural analysis and design
88 by testing, and Appendix B of the Australia / New Zealand standard AS/NZS 4600 [6] that contains
89 provisions for the structural analysis. The discussed structural steel standards for hot-rolled members
90 include Chapter C and Appendix 1 of AISC 360-16 [7] that contain requirements for the design for
91 stability and structural analysis by advanced methods, Chapter 5 of Eurocode 3 (EN 1993-1-1) [4]
92 to describe modeling for structural analysis, Chapter 4 and Appendix D of Australian / New Zealand
93 standard AS/NZS 4100 [8] which provide the requirements for the methods of structural analysis
94 and advanced analysis, and Chapter 8 and Annex O of the Canadian standard CSA S16 [9] to cover
95 structural analysis including advanced analysis. The discussed structural concrete standard includes
96 Chapter 6 of ACI 318 [10]. Furthermore, EN 1993-1-3 states "*For a approach with FE-methods (or*
97 *others) see EN 1993-1-5, Annex C*", therefore Eurocode 3 Part 1-5: Plated Structural Elements [1] is
98 included. Plated structural elements can be applicable to cold-formed steel members in addition to
99 hot-rolled steel members such as plate girders or slender I-beams. It was explored if timber design
100 standards including AITC [11], ANSI/AWC [12], and ANSI/TPI [13] have design by analysis rules,
101 but no specific requirements for testing by analysis was found. Recommendations for testing by
102 analysis based on current design standards, research, and the survey is presented.

103 **2. Survey**

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

108 **2.1. Survey Form**

109 **Software survey**

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

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

- | | | | |
|-----|---|-----|--|
| 115 | <input type="checkbox"/> Structural engineer (Industry) | 119 | <input type="checkbox"/> I have a SE license |
| 116 | <input type="checkbox"/> Civil engineer (Industry) | 120 | <input type="checkbox"/> I have a PE license |
| 117 | <input type="checkbox"/> Structural engineer (Academia) | 121 | <input type="checkbox"/> I have an EIT certification |
| 118 | <input type="checkbox"/> Civil engineer (Academia) | 122 | |

123 The following question lists a series of structural analysis tools. For each program, please
124 check the first column if you use the software for the design of cold-formed steel structures, check the

125 second column if used for the design of structures of other materials, or check both columns if used
126 for both purposes.

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

131

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

133

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

136

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

- | | | | |
|-----|---|-----|--|
| 139 | <input type="checkbox"/> Bi-axial bending | 148 | <input type="checkbox"/> Rigid/semi-rigid link |
| 140 | <input type="checkbox"/> Buckling (local-torsional buckling, | 149 | <input type="checkbox"/> Shear center offset |
| 141 | post-buckling, global buckling, etc) | 150 | <input type="checkbox"/> Static analysis |
| 142 | <input type="checkbox"/> Dynamic analysis | 151 | <input type="checkbox"/> Thermal effect |
| 143 | <input type="checkbox"/> Dynamic loading (wind, earthquake, | 152 | <input type="checkbox"/> Torsion |
| 144 | vehicle, etc) | 153 | <input type="checkbox"/> Warping |
| 145 | <input type="checkbox"/> Fatigue analysis | 154 | <input type="checkbox"/> Web crippling |
| 146 | <input type="checkbox"/> Second-order effects ($P - \delta$, $P - \Delta$) | 155 | <input type="checkbox"/> Other |
| 147 | <input type="checkbox"/> Plastic analysis | | |

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

157 of the software.

158

159 6. Please list any relevant information below.

160 **2.2. Survey Results: Respondents**

161 Fifty-two responses were obtained from the survey. Respondents are structural engineers
162 in the industry (71%) or academia (7.7%), or civil engineer (3.8%), or industry association manager
163 (3.8%). 7.7% are a structural engineer in both industry and academia, 3.8% are both structural
164 engineers and civil engineers in industry, and 2% are mechanical engineers in a structural engineering
165 position.

166 **2.3. Survey Results: Software**

167 The survey responses and the list of software programs are summarized in Table 2.1. The
168 percentage represents the number of responses for the software (n) divided by the total number of
169 responses, $\frac{n}{52} \times 100(\%)$. It was allowed to select multiple software on the list. The survey responses
170 show that in-house excel or Mathcad files are the most commonly used as a design program for
171 both CFS structures (40%) and other materials structures (60%) except the software program CFS 12
172 (65%) for CFS design. Using in-house software composed 37% and 21% for CFS and other materials,
173 respectively. According to the responses, in many companies, in-house software and computer code
174 have been developed to have full automation (optimization) and customization required by design
175 codes. Also, in-house Excel spreadsheets and Mathcad programs developed specifically for the
176 products offered by the company on a regular basis. In the situation when a CFS section is not

177 covered by the in-house programs or the results of the in-house programs must be validated, other
 178 software programs could be used. However, for commercial software, licensing is always a big issue
 179 and the software is typically expensive, thereby in-house software or code are commonly used.

180 Besides in-house software, the software widely used for CFS structures are CFS 12 (65%),
 181 CUFSM (35%), CFS Designer (33%), AISIWIN (21%), Revit (15%), RISA-2D (15%), and MAS-
 182 TAN2 (15%). For the design of structures of other materials, Revit (25%), RISA-2D (23%), RISA-3D
 183 (23%), RAM Structural Systems (21%), SAP2000 (19%), and MASTAN2 (19%) are commonly used.
 184 The use of software for the design of CFS structures are concentrated on the first three programs (CFS
 185 12, CUFSM, and CFS Designer) due to their applicability to CFS members. For other materials, the
 186 top-ranked software programs are utilized with the almost same percentages which range from 19% to
 187 25%. According to the responses, CFS 12 is used for basic CFS section calculation, CUFSM is used
 188 for research projects, and MASTAN2 is used for frame analysis. From the overall responses, software
 189 needs to be inexpensive, fast, accurate, and user friendly. It should be able to produce code-compliant
 190 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%

191 **2.4. Survey Results: Software Capabilities**

192 The survey investigated which software capabilities are considered when using software.
 193 The survey responses are summarized in Table 2.2. The listed capabilities can be categorized
 194 into analysis types and features. For analysis types, static analysis (88%), dynamic analysis (35%),
 195 and plastic analysis (23%) are selected in descending order. The features related to geometric
 196 imperfections and deformations were highly selected — 90% for buckling, 65% for web crippling,
 197 and 58% for torsion. Features that influence internal forces of structures composed high rates with 69%
 198 for bi-axial bending and 65% for second-order effects. In addition to the listed capabilities, nonlinear
 199 analysis, time-dependent effects, and structural members with non-uniform elements are considered
 200 in the analysis. The responses indicate the importance of inclusion of geometric imperfections and
 201 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%

202 **3. Recommendations**

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

205 **3.1. Material**

206 Numerical modeling requires correct representation of the material stress-strain relationship
207 in order to obtain an accurate prediction of structural responses by considering the material stiffness
208 and effects due to yielding and plasticity. The standards for CFS design, EN 1993-1-3 [2] and AS/NZS
209 4600 [6], allow the use of nonlinear material stress-strain relationships for advanced analysis. Annex
210 C.6 of EN 1993-1-5 [1] specifies that material properties should be taken as characteristic values and
211 four types of material behavior may be used as illustrated in Figure 3.1: elastic-plastic without strain
212 hardening, elastic-plastic with a nominal plateau slope, elastic-plastic with linear strain hardening,
213 and true stress-strain curve modified from the test results. True stress and strain are approximated by
214 $\sigma_{true} = \sigma(1 + \epsilon)$ and $\epsilon_{true} = \ln(1 + \epsilon)$, respectively, where σ is stress and ϵ is strain. In addition to
215 these material behaviors, material models recognized for CFS can be adopted [6, 14].

216 Gardner and Yun in 2018 [15] developed an accurate stress-strain model of CFS described
217 by a two-stage Ramberg-Osgood model. Predictive expressions to model the stress-strain curve were
218 developed based on 700 experimental stress-strain curves, covering a wide range of steel grades,
219 thicknesses, and cross-section types. The accuracy of the proposed model is demonstrated even if only
220 the value of the yield strength is known. As such, this model can be considered as appropriate for use

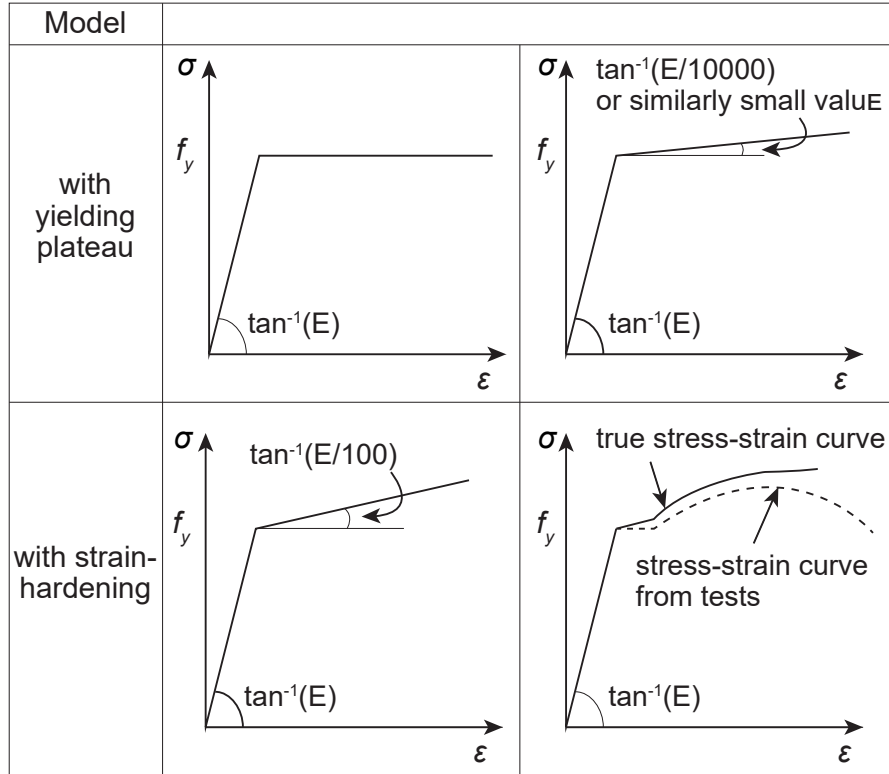


Figure 3.1: Modeling of material behavior from EN 1993-1-5 [1]

221 in design by advanced computational analysis.

222 For design by analysis, it is recommended to consider the nonlinear stress-strain relationships
 223 to capture inelastic behavior of structural components or structures. The authors recommend to use
 224 the Ramberg-Osgood model proposed by Gardner and Yun [15], which is a straight-forward approach
 225 to accurately model cold-formed steel materials.

226 3.2. Modeling of Cross Section

227 The cross-section properties affect the analysis of structural members and systems, espe-
 228 cially for nonsymmetric cross-sections, and must be correctly accounted for. Section 5.1 of EN

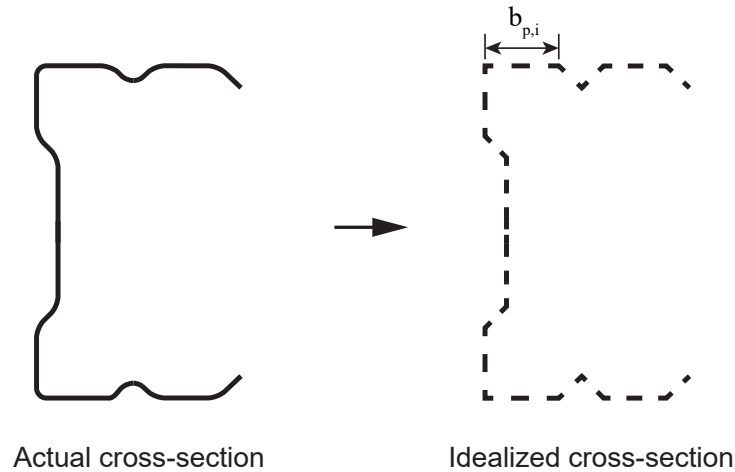


Figure 3.2: Approximate allowance for rounded corners from EN 1993-1-3 [2]

229 1993-1-3 [2] has provisions for considering the effect of rounded corners when determining section
 230 properties. If the internal radius $r \leq 5t$ and $r \leq 0.1b_p$, the rounded corners may be neglected and
 231 instead the cross-section can be assumed to consist of sharp corners as shown in Figure 3.2, where
 232 b_p is the notional flat widths measured from the midpoints of the adjacent corner elements. For
 233 cross-section stiffness properties, the effect of rounded corners should always be considered.

234 Liu et al. [3] investigated an improvement on an existing beam-column line element
 235 formulations for accurately simulating the axial buckling behavior of arbitrarily-shaped open-sections.
 236 One of the asymmetric sections studied was a lipped-C shape consisting of one lip that is turned
 237 outward and one inward. To study the effects of the rounded corners on the section properties, three
 238 different modeling methods to consider the corners were created as shown in Figure 3.3. The three
 239 cross-section models are established based on line-elements with (1) neglecting the rounded corners
 240 (Figure 3.3b), (2) considering the rounded corners as 45-degree line-elements (Figure 3.3c), and (3)
 241 full consideration of the rounded corners with three elements in a corner (Figure 3.3d). The module
 242 MSA_Sect within MASTAN2 [16] was used to compute the section properties. The section properties
 243 generated by CUFSM [17] using the rounding-edges model were employed as the benchmark solution.
 244 As shown in Table 3.1 which displays the results from Liu et al.'s study, the cross-section properties

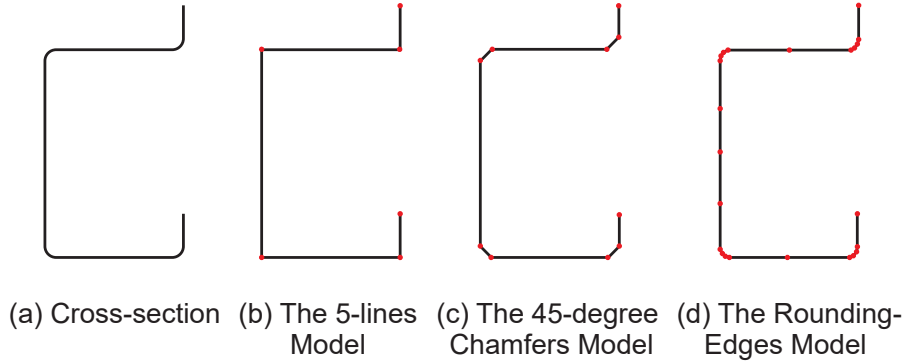


Figure 3.3: Three cross-section models from Liu et al. [3]

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

245 from the rounded corner model were almost identical to the cross-section properties determined from
 246 the rounded corner model in CUFSM [17], which is expected. The important comparison is between
 247 the sharp corner model and the 45-degree corner chamfer model. The sharp corner model resulted
 248 in several cross-section properties with greater than 5% percent error compared to the benchmark
 249 properties, whereas the 45-degree chamfers model had less than 4% percent difference for all section
 250 properties.

251 Section 5.2 of EN 1993-1-3 [2] specifies the range of width-to-thickness ratios that apply for
 252 structural analysis. These limits represent the ranges that have sufficient experience and verification

253 by testing. Cross sections outside the range of the width-to-thickness ratios may be used when their
254 resistance at ultimate limit states and behavior at serviceability limit states are verified by physical
255 testing and/or by analysis (calculations) with an appropriate number of tests, however, the appropriate
256 number is not stated in the standard.

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

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

268 **3.3. Element Type and Size**

269 The choice of FE-models (shell models or solid models) and the size of mesh determine
270 the accuracy of the analysis results. Chapter 6 of ACI 318 [10] requires using the element type
271 that obtains the response required from the task and the mesh size capable of determining the full
272 structural response in detail. Section 3 of AS 4084 [14] suggests to use shell finite elements or finite
273 strips for modeling of storage racks. According to Annex C.1 of EN 1993-1-5 [1], as shown in Table
274 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

275 geometric behaviors, and the presence of imperfections. Validation sensitivity checks with successive
 276 refinement may be performed.

277 Shell elements are utilized when the width-to-thickness ratio of elements is greater than 1.7
 278 and solid elements shall have the ratio smaller than 4.0 [18]. Shell elements may be predominantly
 279 used for CFS structures because standard CFS cross-sections have the width-to-thickness ratios around
 280 33.3.

281 Multiple previous studies performed FE analysis with convergence studies on CFS members
 282 using a four-node shell element (S4R): Theofanou et al. [19] modeled stainless steel oval hollow
 283 sections that have thicknesses between 1.9 mm and 3.2 mm, with the mesh size-to-thickness ratio
 284 varying from 4 to 10.3. As the thickness of the cross section increased, the mesh size decreased.
 285 Natario et al. [20] developed FE models for 4.73 mm thick plain channel section with the mesh
 286 size-to-thickness ratio of 1.8 for the flange and 3.2 for the web. Keerthan and Mahendran [21] utilized
 287 the element size of 5 mm × 5 mm for 1.5 mm or 1.9 mm thick lipped channel beams with web
 288 openings. Pham [22] used a mesh size of 5 mm for 2 mm thick channel sections. Buchanan et al.
 289 [23] employed FE analysis of 1.34 mm thick circular hollow sections. A size of $t \times t$ shell element
 290 was adopted which led to 1.0 as the mesh size-to-thickness ratio. Pham et al. [24] modeled a shear
 291 test of lipped channel beams that have thicknesses varying 1.2 mm to 3.0 mm with a mesh size of 5
 292 mm. Different mesh sizes were used in the test set-up: 5 mm for the angle straps and 10 mm for other
 293 parts of the test set-up such as the stocky column, loading plates, and thick plates.

294 As the mean value of the mesh (four-node shell element) size-to-thickness ratio is 4.4 from

295 the studies covered in this section, the value of 4.4 can be used as the approximate mesh size-to-
296 thickness ratio. Appropriate element sizes would be different based on the geometric properties such
297 as cross-section type and thickness. The authors recommend to perform validation sensitivity checks
298 to determine the mesh size that obtains accurate results or use the mesh size based on the approximate
299 mesh size-to-thickness ratio.

300 **3.4. Geometric Imperfection and Residual Stress**

301 As the pattern and magnitude of geometric imperfections have a significant effect on
302 the structural behavior, correct modeling of the geometric imperfections is necessary to accurately
303 predict the response of the structure. Section C1.1 of AISI S100 [5] states that the effect of geometric
304 imperfections shall be considered in the elastic design by using notional loads or directly using initial
305 imperfections. The maximum displacement considered in the design shall be the magnitude of the
306 initial displacements. The inclusion of imperfections is permissible to the analysis for gravity-only
307 load combinations, not for load combinations including applied lateral loads.

308 Section 5.5 of EN 1993-1-3 [2] provides values of equivalent geometric imperfections,
309 which reflect the possible effects of the imperfections, based on the type of imperfections or analysis.
310 Design value of bow imperfections related to flexural buckling and torsional flexural buckling should
311 be adopted from Table 3.3 with values based on analysis methods including elastic analysis and
312 plastic analysis and five buckling curves illustrated in Figure 3.4. The selection of the appropriate
313 buckling curve is based on the type of cross section, axis of buckling, and yield strength used. e.g.,
314 back-to-back lipped (or plain) channel sections for buckling about the strong axis and the weak axis
315 apply the buckling curves a and b, respectively. Closed built-up cross sections apply the buckling
316 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

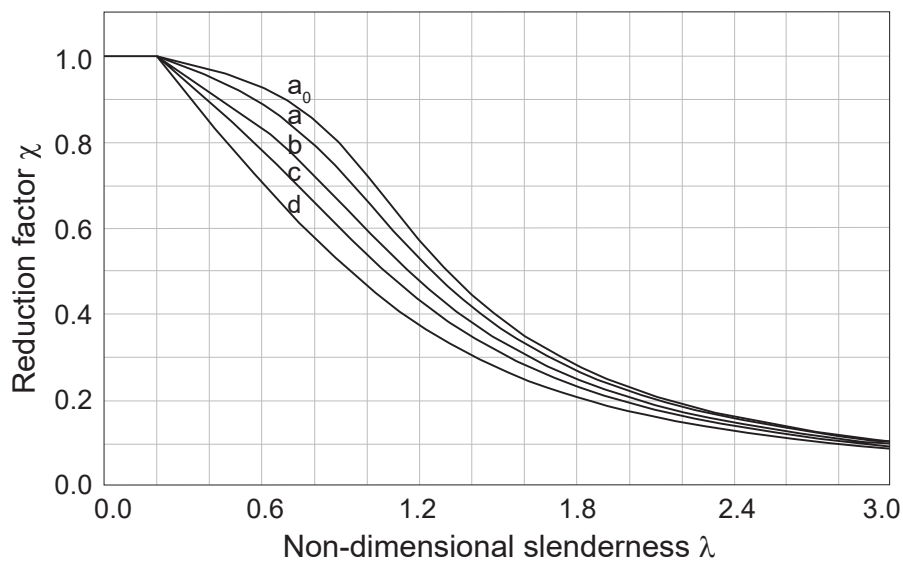


Figure 3.4: Buckling curves from EN 1993-1-1 [4]

317 utilized. Lipped C and Z sections use the buckling curve b. Any other cross sections are applicable
 318 to the buckling curve c. Bow imperfections related to lateral-torsional buckling take $\frac{1}{600}$ for elastic
 319 analysis and $\frac{1}{500}$ for plastic analysis. The effects of cross-sectional imperfections should be taken into
 320 account when determining the resistance and stiffness of CFS members and sheeting. The effects
 321 of distortional buckling should be determined by performing linear or nonlinear buckling analysis
 322 using FE methods. Nonlinear buckling analysis is a static method which accounts for material and
 323 geometric nonlinearities. The examples of buckling analysis with FE methods are previously given
 324 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

325 According to Annex C.5 of EN 1993-1-5 [1] provides equivalent geometric imperfections
326 which may be used if there is an absence of a more refined analysis for the imperfections. Geometric
327 imperfections may be based on the shape of the critical plate buckling modes. For cross-section
328 imperfections, 80% of the geometric fabrication tolerances is recommended. The direction of the
329 imperfection should be chosen which results in the lowest resistance. The equivalent geometric imper-
330 fections may be applied to the model with the values in Table 3.4 and Figure 3.5. When combining
331 imperfections, a leading imperfection should be selected and the accompanying imperfections may
332 have reduced values, 70% of their values. Any type of imperfections can be the leading imperfections
333 or the accompanying imperfections.

334 Appendix B4 of AS/NZS 4600 [6] and Section 3 of AS 4084 [14] recommend including
335 frame, member, and cross-sectional imperfections for the modeling of geometric imperfections. For
336 frame imperfections, an out-of-plumbness ratio of $\frac{1}{500}$ is often adopted as the magnitude of frame
337 imperfections in advanced analysis, or can be accounted for with notional horizontal forces for regular
338 single or multi-story framing structures. For member imperfections, $\frac{1}{1000}$ of the member length shall
339 be the maximum value, which is smaller than $\frac{1}{250}$ that EN 1993-1-3 [2] employs for elastic analysis
340 of lipped C and Z sections. Local and distortional buckling imperfections shall be taken into account
341 in the model by multiplying the local and distortional buckling modes by a factor. Unit maximum
342 deformation is assumed by imperfection multipliers: the imperfection multiplier for local buckling
343 $s_{ol} (= 0.3t\sqrt{\frac{f_y}{f_{ol}}})$ 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]

344 is plate thickness, f_{ol} is elastic local buckling stress, and s_{od} is elastic distortional buckling stress.
345 The scaled imperfections are superimposed onto the perfect geometry. The local and distortional
346 buckling modes may be determined from a linear buckling analysis based on shell FE modeling or
347 finite strip discretization of the member. However, for unbraced pitched roof cold-formed steel portal
348 frames and unbraced cold-formed steel storage racks, local and distortional buckling imperfections
349 are not required to be modeled.

350 Zeinoddini and Schafer [25] evaluated three methods for simulation of geometric imper-
351 fections in CFS members: (1) the Traditional Modal Approach that considers imperfections as a
352 combination of buckling modes; the mode shapes are achieved from an eigenvalue buckling analysis
353 of the member using five cross-sectional buckling mode shapes, (2) the 2D Spectra Approach that
354 considers imperfections as a two-dimensional random field, and (3) 1D Modal Spectral Approach
355 which is a combination of modal and spectral approaches; the spectral approach is used to generate the
356 imperfection magnitudes in the longitudinal direction and the five mode shapes are considered in the
357 transverse direction. A comparison of the simulation results obtained from the three methods shows
358 that the Traditional Modal Approach is conservative for predicting the strength. The 2D Spectra
359 Approach predicts the strength of models that have local and distortional failure with high accuracy,
360 but it is less accurate when the global failure mode is dominant. The 1D Modal Spectral Approach
361 accurately captures the imperfection distributions and the strength, axial flexibility, and failure mech-
362 anism of the member, it is thus the most appropriate method for simulation of imperfections in CFS
363 members [25].

364 In summary, current standards mention three types of geometric imperfections including
365 frame imperfection, member imperfection, and cross-sectional imperfections that should be con-
366 sidered in the analysis in directions that result in the worst case. Frame imperfections can be
367 considered either directly in the structural model or applying notional loads for regular single or
368 multi-story framing structures [6, 5]. Imperfections should be determined based either on actual

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

373 CFS design standards including AISI S100 [5] and EN 1993-1-3 [2] recommend to consider
374 stiffness reductions due to the effects of residual stresses and partial yielding. AS/NZS 4600 [6]
375 includes stiffness reductions due to cross-section deformations or local and distortional deformations
376 in addition to the effects of residual stresses and partial yielding. AISI S100 [5] includes the influence
377 of residual stresses and partial yielding by using the reduction factor 0.9 and the additional factor
378 τ_b that considers the flexural stiffnesses, whereas AISC 360 [7] applies $0.8\tau_b$ to consider reduced
379 stiffness. Residual stresses shall be modeled indirectly through the stress-strain curve [6] or based on
380 a stress pattern produced by the fabrication process with amplitudes equivalent to the mean (expected)
381 values [2].

382 Moen et al. [26] provided a method for predicting initial residual stresses in cold-formed
383 steel members. The proposed method considers residual stresses resulting from two manufacturing
384 processes including (1) sheet coiling, uncoiling, and flattening, and (2) cross-section roll-forming.
385 Equations for predicting the through-thickness residual stress in corner and flat regions regarding
386 the manufacturing processes are derived based on experimental results. The experimental results
387 showed that corners have larger residual stresses than the flats. The equations of residual stresses
388 resulting from sheet coiling, uncoiling, and flattening includes longitudinal residual stresses only
389 while the equations for cross-section roll-forming predict the transverse and longitudinal residual
390 stresses. Residual stresses can be considered in FE models by directly applying suitable equations
391 for the task, which are classified according to corner or flat regions and the manufacturing processes.
392 As stiffness reductions may result in increased deflections and second-order bending moments, it is
393 recommended to consider the effects that lead to reduced stiffness.

394 3.5. Second-order Effects

395 The standards for cold-formed steel (AISI S100 [5] and AS/NZS 4600 [6]) and hot-rolled
396 steel (EN 1993-1-1 [4], AS 4100 [8], and AISC 360 [7]) require/suggest to consider second-order
397 effects in the analysis. AISI S100 [5] considers second-order effects $P-\Delta$ and $P-\delta$ only. AS 4100 [8]
398 includes second-order effects in the analysis, while the type of second-order effects is not specified.
399 EN 1993-1-1 [4] and AS/NZS 4600 [6] include second-order effects arising from deformed geometry
400 not limited to $P-\Delta$ and $P-\delta$. Appendix 1 of AISC 360 [7] includes geometric nonlinearities such as
401 $P-\Delta$, $P-\delta$, and twisting effects. Section 3 of AS 4084 [14] considers twist rotations and torsional
402 internal actions including warping torsion in the analysis.

403 For non-doubly symmetric cross-section members, however, the consideration of only $P-\Delta$
404 and $P-\delta$ in a second-order analysis is not enough to fully reflect behaviors related to asymmetry
405 [27]. Sippel et al. [27] analyzed the response of non-doubly symmetric cross-section beam members.
406 The analysis results were used to evaluate that the methods can accurately capture behaviors related
407 to asymmetry. The inclusion of only $P-\Delta$ and $P-\delta$ in a second-order analysis is not enough to fully
408 reflect the behavior of non-doubly symmetric sections. The consideration of twisting effects including
409 warping, the center of twist, and second-order twist effects are important to the analysis of non-doubly
410 symmetric cross sections. Moreover, the inclusion of asymmetric cross-section properties such as
411 nonconcentric shear center and centroid affects the analysis results. Sippel and Blum [28] examined
412 the importance of the inclusion of the asymmetric section properties to structural systems with non-
413 symmetric sections formed from cold-formed steel members. 65% of the survey responses indicates
414 that second-order effects should be performed in software. Additionally, torsion (58%), shear center
415 offset (37%), and warping (31%) should be considered in analysis. Thus, it is recommended to include
416 not only the effects of $P-\Delta$ and $P-\delta$ but also the effects from twisting effects when non-doubly

417 symmetric cross section is analyzed.

418 **3.6. Connections**

419 AISI S100 [5] and AS/NZS 4600 [6] provide requirements for modeling of connections.
420 Connections shall have sufficient strength and ductility to avoid structural failure within the connec-
421 tions and instead ensure that the structure fails within the members. In addition, if connections show
422 nonlinear behavior, it shall be included in the analysis [6]. Connection deformations and uncertainty
423 in connection stiffness and strength shall be considered [6, 5].

424 Although the CFS design standards [2, 6, 5] have no classification of type of connection
425 model, the authors recommend that the CFS design standards refer to the hot-rolled steel design
426 standards. For connection modeling, for example, CSA S16 [9] provides three types of connections
427 including simple, rigid, and semi-rigid. The design moment-rotation characteristic of a joint may
428 adopt a simplified curve including a linearized approximation such as bi-linear or tri-linear when the
429 simplified curve lies entirely below the design moment-rotation characteristic [4].

430 Since connections of CFS portal frames, storage racks, and built-up sections used in framing
431 display semi-rigid behavior [29, 30, 31], the inclusion of semi-rigidity is significant to the modeling
432 of CFS structures and AS 4084 [14] suggests to account for semi-rigidity of connections in storage
433 racks. The type of connections can be decided by experimental results or previous experience in
434 similar cases. However, the assumption of a pinned connection in racks or studs seated in track
435 should be avoided because it leads to large displacement which decreases system stability [30, 31].

436 For the modeling of steel connections, Zhu et al. [32] proposed a generalized component
437 model that predicts the full range behavior of the steel connections including the post-ultimate and
438 post-fracture ranges. The method can be used to analyze multiple spring models and applicable to

439 all types of steel connections. 37% of the survey responses addressed that connector effect including
440 semi-rigidity is utilized in using the software. It is recommended to consider the effects of connection
441 behavior including semi-rigid behavior in analysis.

442 **3.7. Uncertainty**

443 AISI S100 [5], AS/NZS 4600 [6], AISC 360 [7], and CSA S16 [9] include uncertainty in
444 strength and stiffness which affect the behavior of structures in the analytical model. Consideration
445 of uncertainty in the strength and stiffness properties must be modeled to obtain the most adverse
446 effects on the structure [9]. A reduction factor of 0.9 shall be applied to yield stress and stiffness of
447 all steel members and connections to account for the uncertainty in system, member, and connection
448 strength and stiffness [7]. In addition, AS/NZS 4600 [6] provides capacity reduction factors (ϕ)
449 for the strength and stability limit states of prequalified frames. Values of ϕ are determined from
450 reliability analyses [33, 34]. The frame should support the factored limit states actions multiplied by
451 $\frac{1}{\phi}$, where ϕ is 0.85 for CFS portal frames and 0.9 for steel storage racks.

452 Test-based design provided by Chapter K of AISI S100 [5] requires structural performance
453 to be established by tests or rational engineering analysis with confirmatory tests. The strength of the
454 tested elements, assemblies, connections, or members is determined based on the same procedures
455 used to calibrate the LRFD design criteria, as given in Eq. 3.1. The resistance factor (ϕ) computed by
456 Eq. 3.2 considers the uncertainty in material and geometric properties, failure mode, and prediction
457 of the resistance,

$$\sum \gamma_i Q_i \leq \phi R_n \quad (3.1)$$

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

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

459 where

460 C_ϕ = calibration coefficient

461 β_o = target reliability index

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

463 M_m = mean value of material factor

464 F_m = mean value of fabrication factor

465 V_M = coefficient of variation of material factor

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

468 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
469 tests not fewer than three.

470 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
471 tested strength and $R_{n,i}$ is calculated nominal strength.

472 V_P = coefficient of variation of test results

473 R_n = average value of all test results

474

475 The correlation coefficient (C_c) between the tested strength and the nominal strength pre-
476 dicted from the rational engineering analysis model shall be greater than or equal to 0.8. The bias and
477 variance between the measured and the nominally specified dimensions and material properties shall
478 be reflected by including fabrication (F_m and V_F) and material (M_m and V_M) factors to the calculation
479 of resistance factor. The authors recommend to consider uncertainties in material and geometric

480 properties because they affect the response of a member or a structure.

481 **3.8. Benchmark Test**

482 Annex C of EN 1993-1-5 [1], Appendix B of AS/NZS 4600 [6], and Chapter C and
483 Appendix 1 of AISC 360 [7] require performing benchmark tests to prove the software is appropriate
484 for the task. In Appendix 1 of AISC 360 [7], benchmark tests are used to check if the second-order
485 effects resulting from the combination of axial force, flexure, and twist are being correctly performed
486 in elastic analysis. Otherwise, according to Chapter C, benchmark problems are used to verify that
487 $P - \delta$ and $P - \Delta$ second-order analysis used in the direct analysis method provide a confidence level
488 of the task. Benchmark tests can be performed by well-documented experimental results or similar
489 benchmark results [6].

490 Pham [22] used the finite-strip method as a benchmark test of FE method for elastic buckling
491 analysis and the results from the two methods agree within 2% error. Ziemian et al. [35] performed
492 benchmark problems to ensure that the nonlinear analysis of an unbraced I-shaped member subjected
493 to in-plane and out-of-plane loading effects that have significant spatial behavior such as warping
494 and twisting effects achieves accurate results. The benchmark problems are crucial in validating the
495 proper use of nonlinear analysis when the modeling of spatial behavior is important. The survey
496 responses addressed that in-house excel or software is the most widely used software for structural
497 design. The common use of in-house software emphasizes the necessity of benchmark tests for the
498 validation of the software. Overall, it is recommended to perform benchmark tests to validate the
499 accuracy of the software and the authors recommend AISI to develop requirements for employing
500 benchmark tests.

501 **3.9. Dimension: 2D or 3D**

502 Annex O of CSA S16 [9] and Appendix B of AS/NZS 4600 [8] include provisions for di-
503 mension of the model. CSA S16 [9] requires using a three-dimensional model, but a two-dimensional
504 model can be employed providing that the use of model is validated for design. For the use of two-
505 dimensional model, it is required to consider the out-of-plane response. AS/NZS 4600 [6] addresses
506 the case of using a two-dimensional model without provisions for using three-dimensional analysis. A
507 two-dimensional model can be used for analyzing regular building structures by considering them as a
508 series of parallel two-dimensional substructures. The analysis should be carried out in two directions
509 at right angles. However, the use of two-dimensional analysis is not applicable to structures that have
510 significant load redistribution between the substructures. As it is important to consider the spatial
511 behavior in analysis [35, 27], the authors recommend to employ a three-dimensional model to achieve
512 correct structural responses.

513 **3.10. Superposition Principle**

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

520 **3.11. Documentation of Results**

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

525 **4. Selected Recent Research**

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

531 **4.1. Buchanan et al. (2020)**

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

538 The FE models utilized the 4-node doubly curved shell element (S4R). A mesh validation
539 study was performed with the element size varying from $10t$ to $\frac{t}{3}$, where $t = 1.34$ mm is the thickness.
540 A size of $t \times t$ shell element was adopted as it yielded accurate failure load and deflection from the
541 finest mesh, $\frac{t}{3}$, while maintaining computational efficiency. In addition, computational efficiency was
542 increased by modeling half of the cross section and employing symmetrical boundary conditions.
543 The FE models utilized the stress-strain relationships obtained from the measured tensile coupons

544 and compressive stub column responses. The effect of membrane residual stresses was neglected
545 while the through-thickness residual stresses are implicitly considered by using measured material
546 properties. The modeling of boundary conditions followed the test conditions. The form of local and
547 global geometric imperfections adopted the lowest local and global buckling mode shapes from an
548 elastic buckling analysis. The amplitudes utilized were the measured mid-point global imperfections
549 (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
550 length and t is the section thickness. In order to validate the numerical models from the experimental
551 results, various amplitudes of imperfections and material properties including compressive and tensile
552 properties were used.

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

560 **4.2. Pham et al. (2020)**

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

566 In the FE models, the S4R element with a mesh size of 5 mm was used for the cold-formed
567 channel sections while the 8-node linear solid element (C3D8R) was used for the test set-up with
568 a mesh size of 5 mm for the angle straps and 10 mm for other parts of the test set-up such as the
569 stocky column, loading plates, and thick plates. The area surrounding the web openings adopted
570 sweep meshing. The modeling of boundary conditions and connections followed the actual tests.
571 The nonlinear behavior of the bolted connection was included in the analysis by using the nonlinear
572 elastic properties obtained from the test results. For the material properties, the true stress-strain
573 curve as previously shown in Figure 3.1 was adopted with measured stress and strain from the tensile
574 coupon tests. The initial geometric imperfections were specified by the buckling modes with the
575 lowest eigenvalue. Two scaling factors for the imperfection, $0.15t$ [38] and $0.64t$ [39], were employed
576 as the imperfection amplitudes.

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

581 **4.3. Kyvelou et al. (2018)**

582 Kyvelou et al. [40] developed FE models of composite flooring systems comprising cold-
583 formed steel channel section beams with two stiffeners and wood-based particle boards using Abaqus
584 [36]. One hundred simulations were generated and the simulation results were validated against twelve
585 physical test results. A parametric study was conducted to investigate the effect of key parameters
586 on the performance of the flooring systems including the depth and thickness, and the spacing of
587 fasteners.

588 The material model adopted the two-stage Ramberg-Osgood model proposed by Gardner
589 and Ashraf [41]. This study carried out corner coupon tests and it was revealed that the corner regions
590 have 17% higher yield strength than the flat regions. The strength enhancements in the corner regions
591 were considered by assigning different material properties. The effect of through-thickness residual
592 stresses was implicitly included in the stress-strain curves. The S4R shell elements with a longitudinal
593 size of 10 mm were chosen for the modeling of the CFS beams. The C3D8R solid elements with
594 a longitudinal size of 10 mm were used to model the wood-based flooring panels. The self-drilling
595 screws acting as the shear connection between the joists and the flooring panels were modeled with
596 nonlinear spring elements that consider the load-slip response based on the push-out test results. For
597 modeling of initial geometric imperfections, the pure local and distortional buckling mode shapes
598 were obtained from CUFSM [17]. The obtained buckling modes were distributed longitudinally,
599 through sinusoidal functions, and the deformed geometry was directly modeled in Abaqus [36] as the
600 initial imperfections. The scaling factors for the local and distortional buckling mode shapes, $0.1t$
601 [42] and $0.3t$ [43], respectively, were employed as the imperfection magnitudes.

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

609 **5. Conclusion**

610 Testing by analysis can compensate for the limitations of physical testing such as high
611 cost and time. This project discussed the literature review of design standards for cold-formed steel
612 structures and other industries that include testing by analysis requirements. In addition, a state-of-
613 the-art review of selected research studies on testing by analysis and a survey for understanding the
614 current commonly used software and software capabilities are presented. Overall, recommendations
615 on the use of testing by analysis to cold-formed steel design with regard to material, modeling of
616 cross section, element type and size, imperfection, second-order effects, uncertainty, dimensions,
617 benchmark test, and connection are provided. The recommendations will be helpful for possible
618 future codification of test-based design in the AISI standards which currently have no provisions for
619 testing by analysis.

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