CAN/CSA-S6-06
A National Standard of Canada

Canadian Highway Bridge Design Code

CANADIAN STANDARDS ASSOCIATION

Standards Council of Canada
Conseil canadien des normes

S6S1-10
Supplement No. 1 to CAN/CSA-S6-06, Canadian Highway Bridge Design Code
May 2010

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Title: *Canadian Highway Bridge Design Code* — originally published November 2006

The following revisions have been formally approved and are marked by the symbol delta (Δ) in the margin on the attached replacement pages:

Revised	Table of Contents, Clauses 1.2, 3.10.2.2, 3.10.5.2, A3.3.3.3.1, A3.3.3.3.3, 4.4.7.1, 4.5.3.2, 4.7.4.3, Table of Contents (Section 5), 5.3.2, 5.7.1.6.1.1, 5.7.1.7.1, 5.12.3, 8.4.4.6, 8.8.4.6, 8.9.3.18, 8.10.5.1, 8.14.6, 8.16.7.6.4.2, 8.16.7.6.5, Table of Contents (Section 9), 9.6.1, 9.6.2, 9.8.1, 9.8.3.3, Table of Contents (Section 10), 10.3.2, 10.4.2, 10.4.5–10.4.7, 10.4.9, 10.4.11.2, 10.5.3.2, 10.5.7, 10.5.9.2.4, 10.6.4.2, 10.6.6, 10.7, 10.7.1, 10.7.4.3, 10.8.1.3.1, 10.8.1.3.2, 10.8.2, 10.9.3.1, 10.9.5.2, 10.9.5.4, 10.10.2.3, 10.10.8.1, 10.10.8.2, 10.11.5.3.2, 10.11.6.3.1.2, 10.11.8.2, 10.11.8.3, 10.12.5.3.2, 10.12.5.4, 10.13.6.1.2, 10.13.6.1.3, 10.13.7.3, 10.17.2.2, 10.17.2.3.2, 10.17.2.7, 10.18.1.5, 10.18.2.3.1–10.18.2.3.3, 10.18.3.2.2, 10.19, 10.20.2.4, 10.22, 10.22.1–10.22.4, 10.23.3.5, 10.23.4.2–10.23.4.5, 10.24.3.2, 10.24.4.3.2, 10.24.4.8, 10.24.5.1, 10.24.5.4–10.24.5.6, 10.24.7.2, 10.24.8.2, 10.24.9, Table of Contents (Section 14), 14.3, 14.5.2.1, 14.5.2.3, 14.5.4, 14.5.4.1, 14.6.2, 14.6.4, 14.7.1, 14.7.4.3, 14.8.2.1, 14.9.3, 14.14.1.3.1, 14.14.1.3.2, 14.17.1, 15.6.2.4, 16.3.2, 16.5.3, 16.6.2, 16.7.1–16.7.3, 16.8.4.1, 16.8.6.3, 16.8.7, 16.8.8.1, 16.8.8.2, 16.10, 16.11.2.5.6, and 16.11.3.1–16.11.3.3, Tables 3.1, A3.1.1, 4.6, 5.4, 5.10, A5.2.2, 8.5, 9.15, 10.3, 10.9, 10.18, 13.3–13.5, 14.14, 16.2, and 16.3, and Figures 5.1, 5.2, 5.3, 5.6, 10.1, 10.3, and 10.5
New	Technical Committee (membership at the time of the formal approval of CSA S6S1-10), Subcommittees on Sections 3–6, 8, 10, 14, and 16 (membership at the time of the formal approval of CSA S6S1-10), Code Calibration Task Force (membership at the time of the formal approval of CSA S6S1-10), French Translation Task Force (membership at the time of the formal approval of CSA S6S1-10), Regulatory Authority Committee (membership at the time of the formal approval of CSA S6S1-10), Clauses 10.4.13, 10.5.3.6, 10.8.1.3.2.1–10.8.1.3.2.5, 10.11.8.3.1–10.11.8.3.3, 10.22.5–10.22.8, and 14.4.4, Tables 10.14A and 10.14B, and Figure 5.1A
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- Update your copy by inserting these revised pages.
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W.V. Anderson	Delcan Corporation, Markham, Ontario	
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N. Banthia	University of British Columbia, Vancouver, British Columbia	*Associate*
F.M. Bartlett	University of Western Ontario, London, Ontario	
D. Beaulieu	Centre de recherche industrielle du Québec, Ste-Foy, Québec	
G.P. Carlin	Les Ponts Jacques Cartier et Champlain Incorporée, Longueuil, Québec	*Associate*
M.S. Cheung	University of Ottawa, Ottawa, Ontario	
C. Clarke	Alberta Infrastructure and Transportation, Edmonton, Alberta	
D. Cogswell	New Brunswick Department of Transportation, Fredericton, New Brunswick	*Associate*

D. Dundas	Ontario Ministry of Transportation, Toronto, Ontario	*Associate*
D.J. Evans	Prince Edward Island Department of Transportation and Public Works, Charlottetown, Prince Edward Island	
P. Gauvreau	University of Toronto, Toronto, Ontario	*Associate*
A. Ho	Ontario Ministry of Transportation, Toronto, Ontario	
K.C. Johns	Université de Sherbrooke, Sherbrooke, Québec	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
P. Lepper	Canadian Wood Council, Ottawa, Ontario	
D. Macleod	Public Works and Government Services Canada, Gatineau, Québec	
B. Massicotte	École Polytechnique de Montréal, Montréal, Québec	
R. Mathieson	British Columbia Ministry of Transportation, Victoria, British Columbia	
R.J. McGrath	Cement Association of Canada, Ottawa, Ontario	
D. Mitchell	McGill University, Montréal, Québec	
A.A. Mufti	University of Manitoba, Winnipeg, Manitoba	
A. Nelson	Manitoba Department of Highways and Transportation, Winnipeg, Manitoba	
R.J. Ramsay	UMA Engineering Limited, Edmonton, Alberta	*Associate*
J.A. Skeet	Dillon Consulting Limited, Calgary, Alberta	*Associate*
A.F. Wong	Canadian Institute of Steel Construction, Toronto, Ontario	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Technical Committee on the Canadian Highway Bridge Design Code (CSA S6S1-10)

Note: *This list reflects the Technical Committee membership when CSA S6S1-10 was formally approved.*

H. Yea	Saskatchewan Highways and Infrastructure, Regina, Saskatchewan	*Chair*
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W.V. Anderson	Delcan Corporation, Markham, Ontario	
L. Atkin	Alberta Transportation, Edmonton, Alberta	
D. Bagnariol	Ontario Ministry of Transportation, St. Catharines, Ontario	*Associate*
N. Banthia	University of British Columbia, Vancouver, British Columbia	*Associate*
F.M. Bartlett	University of Western Ontario, London, Ontario	
D. Beaulieu	Université Laval, Québec, Québec	
M.S. Cheung	University of Ottawa, Ottawa, Ontario	
D. Cogswell	New Brunswick Department of Transportation, Fredericton, New Brunswick	*Associate*
D. Dundas	Ontario Ministry of Transportation, Toronto, Ontario	*Associate*

P. Gauvreau	University of Toronto, Toronto, Ontario	*Associate*
G. Grondin	Unversity of Alberta, Edmonton, Alberta	*Associate*
A. Ho	Ontario Ministry of Transportation, Toronto, Ontario	
K.C. Johns	Université de Sherbrooke, Sherbrooke, Québec	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
P. Lepper	Canadian Wood Council, Ottawa, Ontario	
B. Massicotte	École Polytechnique de Montréal, Montréal, Québec	
R. Mathieson	British Columbia Ministry of Transportation and Infrastructure, Victoria, British Columbia	
R.J. McGrath	Cement Association of Canada, Ottawa, Ontario	
D. Mitchell	McGill University, Montréal, Québec	
A.A. Mufti	University of Manitoba, Winnipeg, Manitoba	
R.J. Ramsay	AECOM, Edmonton, Alberta	*Associate*
G. Richard	Dessau Inc., Québec, Québec	
J. Saweczko	Byrne Engineering Inc., Burlington, Ontario	
J.A. Skeet	Dillon Consulting Limited, Calgary, Alberta	*Associate*
A.F. Wong	Canadian Institute of Steel Construction, Toronto, Ontario	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 1 — General

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R. Richardson	Manitoba Department of Highways and Transportation, Winnipeg, Manitoba	
D.M. Tran	Ministère des transports du Québec, Québec, Québec	
R. Walters	Stantec Consulting Limited, Edmonton, Alberta	
E. Waschuk	Alberta Infrastructure and Transportation, Edmonton, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

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J. Kroman	City of Calgary, Calgary, Alberta	
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S. Mindess	University of British Columbia, Vancouver, British Columbia	
K. Sakai	Kagawa University, Takamatsu, Japan	
G. Tadros	STECO Engineering Ltd., Calgary, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

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J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
P. King	University of Western Ontario, London, Ontario	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
D. Mitchell	McGill University, Montréal, Québec	
R.H. Pion	Public Works and Government Services Canada, Gatineau, Québec	
G. Van Der Vinne	Northwest Hydraulic Consultants Ltd., Edmonton, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Subcommittee on Section 3 — Loads (CSA S6S1-10)

Note: *This list reflects the Subcommittee membership when CSA S6S1-10 was formally approved.*

A.C. Agarwal	Brampton, Ontario	*Chair*
A. Au	Ontario Ministry of Transportation, St. Catharines, Ontario	*Secretary*
D.P. Gagnon	Buckland & Taylor Ltd., North Vancouver, British Columbia	
J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
P. King	University of Western Ontario, London, Ontario	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
D. Mitchell	McGill University, Montréal, Québec	
G. Van Der Vinne	Northwest Hydraulic Consultants Ltd., Edmonton, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 4 — Seismic design (CAN/CSA-S6-06)

Note: *This list reflects the Subcommittee membership when CAN/CSA-S6-06 was formally approved.*

D. Mitchell	McGill University, Montréal, Québec	*Chair*
U.D. Atukorala	Golder Associates Ltd., Burnaby, British Columbia	
D. Bagnariol	Ontario Ministry of Transportation, St. Catharines, Ontario	
M. Bruneau	University at Buffalo, Buffalo, New York, USA	
A. Heidebrecht	McMaster University, Hamilton, Ontario	
D. Kennedy	Associated Engineering Ltd., Burnaby, British Columbia	
N. Theodor	Ontario Ministry of Transportation, St. Catharines, Ontario	
R. Tremblay	École Polytechnique de Montréal, Montréal, Québec	
S. Zhu	Buckland & Taylor Ltd., North Vancouver, British Columbia	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Subcommittee on Section 4 — Seismic design (CSA S6S1-10)

Note: *This list reflects the Subcommittee membership when CSA S6S1-10 was formally approved.*

D. Mitchell	McGill University, Montréal, Québec	*Chair*
U.D. Atukorala	Golder Associates Ltd., Burnaby, British Columbia	
D. Bagnariol	Ontario Ministry of Transportation, St. Catharines, Ontario	
M. Bruneau	University at Buffalo, Buffalo, New York, USA	
A. Heidebrecht	McMaster University, Hamilton, Ontario	
D. Kennedy	Associated Engineering Ltd., Burnaby, British Columbia	
N. Theodor	Ontario Ministry of Transportation, St. Catharines, Ontario	
R. Tremblay	École Polytechnique de Montréal, Montréal, Québec	
S. Zhu	Buckland & Taylor Ltd., North Vancouver, British Columbia	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 5 — Methods of analysis (CAN/CSA-S6-06)

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B. Massicotte	École Polytechnique de Montréal, Montréal, Québec	*Chair*
J. Au	Ontario Ministry of Transportation, St. Catharines, Ontario	
T. Chicoine	SNC-Lavalin Inc., Montréal, Québec	
J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
J. Newhook	Dalhousie University, Halifax, Nova Scotia	
S. Sabanathan	Ontario Ministry of Transportation, St. Catharines, Ontario	
M. Talbot	Ministère des transports du Québec, Québec, Québec	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Subcommittee on Section 5 — Methods of analysis (CSA S6S1-10)

Note: *This list reflects the Subcommittee membership when CSA S6S1-10 was formally approved.*

B. Massicotte	École Polytechnique de Montréal, Montréal, Québec	*Chair*
J. Au	Ontario Ministry of Transportation, St. Catharines, Ontario	
T. Chicoine	Buckland & Taylor Ltd., North Vancouver, British Columbia	
J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
R. Hasan	Ontario Ministry of Transportation, St. Catharines, Ontario	
M. Talbot	Ministère des transports du Québec, Québec, Québec	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 6 — Foundations (CAN/CSA-S6-06)

Note: *This list reflects the Subcommittee membership when CAN/CSA-S6-06 was formally approved.*

D. Dundas	Ontario Ministry of Transportation, Toronto, Ontario	*Chair*
A. Altaee	Urkkada Technology, Ottawa, Ontario	
U.D. Atukorala	Golder Associates Ltd., Burnaby, British Columbia	
P. Branco	Thurber Engineering Limited, Oakville, Ontario	
G. Fenton	Dalhousie University, Halifax, Nova Scotia	
R. Green	Waterloo, Ontario	
I. Husain	Ontario Ministry of Transportation, St. Catharines, Ontario	
I. Leclerc	Ministère des transports du Québec, Québec, Québec	
P. Ojala	Lea Consulting Limited, Markham, Ontario	
D. Woeller	ConeTec Investigations Ltd., Vancouver, British Columbia	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Subcommittee on Section 6 — Foundations (CSA S6S1-10)

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D. Dundas	Ontario Ministry of Transportation, Toronto, Ontario	*Chair*
A. Altaee	Urkkada Technology, Ottawa, Ontario	
U.D. Atukorala	Golder Associates Ltd., Burnaby, British Columbia	
P. Branco	Thurber Engineering Limited, Oakville, Ontario	
G. Fenton	Dalhousie University, Halifax, Nova Scotia	
R. Green	Waterloo, Ontario	
I. Husain	Ontario Ministry of Transportation, St. Catharines, Ontario	
I. Leclerc	Ministère des transports du Québec, Québec, Québec	
P. Ojala	Lea Consulting Limited, Markham, Ontario	
D. Woeller	ConeTec Investigations Ltd., Vancouver, British Columbia	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 7 — Buried structures

B. Bakht	JMBT Structures Research Inc., Toronto, Ontario	*Chair*
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K. Bontius	Hatch, Mott & MacDonald, Mississauga, Ontario	
W. Brockbank	Reinforced Earth Company Ltd., Mississauga, Ontario	
M. Gergely	Ontario Ministry of Transportation, St. Catharines, Ontario	
W. Kenedi	Ontario Ministry of Transportation, St. Catharines, Ontario	
E. Kling	Centennial Concrete Pipe & Products Inc., Cambridge, Ontario	
I. Leclerc	Ministère des transports du Québec, Québec, Québec	
J. Meyboom	Delcan Corporation, New Westminster, British Columbia	
C. Mirza	Toronto, Ontario	
I. Moore	Queen's University, Kingston, Ontario	
T. Morrison	Atlantic Industries Limited, Cambridge, Ontario	
J. Newhook	Dalhousie University, Halifax, Nova Scotia	
P. Sheehan	Armtec Limited Partnership, Guelph, Ontario	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 8 — Concrete structures (CAN/CSA-S6-06)

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D. Bernard	Ministère des transports du Québec, Québec, Québec	
M.P. Collins	University of Toronto, Toronto, Ontario	
H. Ibrahim	Buckland & Taylor Ltd., North Vancouver, British Columbia	
W. Leblanc	Con-Force Structures Limited, Calgary, Alberta	
R.J. McGrath	Cement Association of Canada, Ottawa, Ontario	
M. Meleka	Ontario Ministry of Transportation, St. Catharines, Ontario	
D.M. Rogowsky	UMA Engineering Ltd., Edmonton, Alberta	
R. Stofco	McCormick Rankin Corporation, Mississauga, Ontario	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Subcommittee on Section 8 — Concrete structures (CSA S6S1-10)

Note: *This list reflects the Subcommittee membership when CSA S6S1-10 was formally approved.*

P. Gauvreau	University of Toronto, Toronto, Ontario	*Chair*
D. Bernard	Ministère des transports du Québec, Québec, Québec	
M.P. Collins	University of Toronto, Toronto, Ontario	
S. Goulet	Ministère des transports du Québec, Québec, Québec	
H. Ibrahim	Buckland & Taylor Ltd., North Vancouver, British Columbia	
W. Leblanc	Con-Force Structures Limited, Calgary, Alberta	
R.J. McGrath	Cement Association of Canada, Ottawa, Ontario	
M. Meleka	Ontario Ministry of Transportation, St. Catharines, Ontario	
D.M. Rogowsky	UMA Engineering Ltd., Edmonton, Alberta	
R. Stofco	McCormick Rankin Corporation, Mississauga, Ontario	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 9 — Wood structures

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L.M. Bélanger	Ministère des transports du Québec, Québec, Québec	
R.M.G. Britton	University of Manitoba, Winnipeg, Manitoba	
G.E. Brudermann	Frido Consulting, Halfmoon Bay, British Columbia	
R.J. Eden	Manitoba Floodway Authority, Winnipeg, Manitoba	
M. Erki	Royal Military College of Canada, Kingston, Ontario	
R.O. Foschi	University of British Columbia, Vancouver, British Columbia	
R. Krisciunas	Ontario Ministry of Transportation, Thunder Bay, Ontario	
P. Lepper	Canadian Wood Council, Ottawa, Ontario	
I. Sturrock	British Columbia Ministry of Transportation, Victoria, British Columbia	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 10 — Steel structures (CAN/CSA-S6-06)

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D. Beaulieu	Centre de recherche industrielle du Québec, Ste-Foy, Québec	*Chair*
D. Francis	Supreme Steel Ltd., Edmonton, Alberta	
G. Grondin	University of Alberta, Edmonton, Alberta	
H. Hawk	Delcan Corporation, Calgary, Alberta	
J. Labbé	Ministère des transports du Québec, Québec, Québec	
N. Theodor	Ontario Ministry of Transportation, St. Catharines, Ontario	
R. Vincent	Canam Group Inc., Boucherville, Québec	
A.F. Wong	Canadian Institute of Steel Construction, Toronto, Ontario	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Subcommittee on Section 10 — Steel structures (CSA S6S1-10)

Note: *This list reflects the Subcommittee membership when CSA S6S1-10 was formally approved.*

G. Grondin	University of Alberta, Edmonton, Alberta	*Chair*
R. Vincent	Canam Group Inc., Boucherville, Québec	*Vice-Chair*
H. Hawk	Delcan Corporation, Calgary, Alberta	
P. King	Rapid-Span Structures, Armstrong, British Columbia	
J. Labbé	Ministère des transports du Québec, Québec, Québec	
N. Theodor	Ontario Ministry of Transportation, St. Catharines, Ontario	
E. Whalen	Canadian Institute of Steel Construction, Markham, Ontario	
A.F. Wong	Canadian Institute of Steel Construction, Markham, Ontario	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 11 — Joints and bearings

J.A. Skeet	Dillon Consulting Limited, Calgary, Alberta	*Chair*
J. Labbé	Ministère des transports du Québec, Québec, Québec	
R. Mihaljevic	Ontario Ministry of Transportation, St. Catharines, Ontario	
N. Patel	Ontario Ministry of Transportation, St. Catharines, Ontario	
J.F. Reysset	Goodco Ltée, Laval, Québec	
R. Yu	Alberta Infrastructure and Transportation, Edmonton, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 12 — Barriers and highway accessory supports

R.J. Ramsay	UMA Engineering Limited, Edmonton, Alberta	*Chair*
D. Beaulieu	Centre de recherche industrielle du Québec, Ste-Foy, Québec	
M. Blouin	Ministère des transports du Québec, Québec, Québec	
R. Haynes	Ontario Ministry of Transportation, St. Catharines, Ontario	
M. Vallières	Ministère des transports du Québec, Québec, Québec	
R. Yu	Alberta Infrastructure and Transportation, Edmonton, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 13 — Movable bridges

T.B. Tharmabala	Ontario Ministry of Transportation, St. Catharines, Ontario	*Chair*
D. Campbell	Klohn-Krippen Consultants Ltd., Vancouver, British Columbia	
R. Clayton	Byrne Engineering Inc., Burlington, Ontario	
J. Crabb	Delcan Corporation, Markham, Ontario	
D. Dixon	McCormick Rankin Corporation, Mississauga, Ontario	
L. Huang	Modjeski and Masters, Mechanicsburg, Pennsylvania, USA	
Q. Islam	Ontario Ministry of Transportation, Kingston, Ontario	
W. McCraken	Byrne Engineering Inc., Burlington, Ontario	
J. Saweczko	Byrne Engineering Inc., Burlington, Ontario	
P.M. Skelton	Hardesty & Hanover, New York, New York, USA	
A. Zaki	SNC-Lavalin Inc., Montréal, Québec	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 14 — Evaluation (CAN/CSA-S6-06)

Note: *This list reflects the Subcommittee membership when CAN/CSA-S6-06 was formally approved.*

D. Bagnariol	Ontario Ministry of Transportation, St. Catharines, Ontario	*Chair*
F.M. Bartlett	University of Western Ontario, London, Ontario	
D.P. Gagnon	Buckland & Taylor Ltd., North Vancouver, British Columbia	
J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
B. Higgins	CBCL Limited, Halifax, Nova Scotia	
W. Kenedi	Ontario Ministry of Transportation, St. Catharines, Ontario	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
J. Meyboom	Delcan Corporation, New Westminster, British Columbia	
P. Ojala	Lea Consulting Limited, Markham, Ontario	
R.J. Ramsay	UMA Engineering Limited, Edmonton, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

∆ Subcommittee on Section 14 — Evaluation (CSA S6S1-10)

Note: *This list reflects the Subcommittee membership when CSA S6S1-10 was formally approved.*

D. Bagnariol	Ontario Ministry of Transportation, St. Catharines, Ontario	*Chair*
F.M. Bartlett	University of Western Ontario, London, Ontario	
D.P. Gagnon	Buckland & Taylor Ltd., North Vancouver, British Columbia	
J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
B. Higgins	CBCL Limited, Halifax, Nova Scotia	
W. Kenedi	Ontario Ministry of Transportation, St. Catharines, Ontario	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
R.J. Ramsay	AECOM, Edmonton, Alberta	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 15 — Rehabilitation and repair

F.M. Bartlett	University of Western Ontario, London, Ontario	*Chair*
D. Dixon	McCormick Rankin Corporation, Mississauga, Ontario	
L. Feldman	University of Saskatchewan, Saskatoon, Saskatchewan	
D.P. Gagnon	Buckland & Taylor Ltd., North Vancouver, British Columbia	
D. Lai	Ontario Ministry of Transportation, St. Catharines, Ontario	
K.W. Neale	Université de Sherbrooke, Sherbrooke, Québec	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Subcommittee on Section 16 — Fibre-reinforced structures (CAN/CSA-S6-06)

Note: *This list reflects the Subcommittee membership when CAN/CSA-S6-06 was formally approved.*

A.A. Mufti	University of Manitoba, Winnipeg, Manitoba	*Chair*
B. Bakht	JMBT Structures Research Inc., Toronto, Ontario	
N. Banthia	University of British Columbia, Vancouver, British Columbia	
B. Benmokrane	Université de Sherbrooke, Sherbrooke, Québec	
G. Desgagné	Ministère des transports du Québec, Québec, Québec	
R.J. Eden	Manitoba Floodway Authority, Winnipeg, Manitoba	
M. Erki	Royal Military College of Canada, Kingston, Ontario	
V. Karbhari	University of California at San Diego, La Jolla, California, USA	
J. Kroman	City of Calgary, Calgary, Alberta	
D. Lai	Ontario Ministry of Transportation, St. Catharines, Ontario	
A. Machida	Saitama University, Urawa, Japan	
K.W. Neale	Université de Sherbrooke, Sherbrooke, Québec	
G. Tadros	STECO Engineering Ltd., Calgary, Alberta	
B. Taljsten	BYG-DTU, Lyngby, Denmark	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Subcommittee on Section 16 — Fibre-reinforced structures (CSA S6S1-10)

Note: *This list reflects the Subcommittee membership when CSA S6S1-10 was formally approved.*

A.A. Mufti	University of Manitoba, Winnipeg, Manitoba	*Chair*
B. Bakht	JMBT Structures Research Inc., Toronto, Ontario	*Secretary*
N. Banthia	University of British Columbia, Vancouver, British Columbia	
B. Benmokrane	Université de Sherbrooke, Sherbrooke, Québec	
G. Desgagné	Ministère des transports du Québec, Québec, Québec	
R.J. Eden	Manitoba Floodway Authority, Winnipeg, Manitoba	
M. Erki	Royal Military College of Canada, Kingston, Ontario	
V. Karbhari	University of California at San Diego, La Jolla, California, USA	
J. Kroman	City of Calgary, Calgary, Alberta	
D. Lai	Ontario Ministry of Transportation, St. Catharines, Ontario	
A. Machida	Saitama University, Urawa, Japan	
K.W. Neale	Université de Sherbrooke, Sherbrooke, Québec	
J. Newhook	Dalhousie University, Halifax, Nova Scotia	
S. Shamim	University of Toronto, Toronto, Ontario	
G. Tadros	STECO Engineering Ltd., Calgary, Alberta	

B. Taljsten	BYG-DTU, Lyngby, Denmark	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Code Calibration Task Force (CAN/CSA-S6-06)

Note: *This list reflects the Code Calibration Task Force membership when CAN/CSA-S6-06 was formally approved.*

A.C. Agarwal	Brampton, Ontario	*Chair*
A. Au	Ontario Ministry of Transportation, St. Catharines, Ontario	
D. Becker	Golder Associates Ltd., Calgary, Alberta	
R.O. Foschi	University of British Columbia, Vancouver, British Columbia	
D.P. Gagnon	Buckland & Taylor Ltd., North Vancouver, British Columbia	
H. Hong	University of Western Ontario, London, Ontario	
P. King	University of Western Ontario, London, Ontario	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
D. Mitchell	McGill University, Montréal, Québec	
A.S. Nowak	University of Nebraska, Lincoln, Nebraska, USA	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

∆ Code Calibration Task Force (CSA S6S1-10)

Note: *This list reflects the Code Calibration Task Force membership when CSA S6S1-10 was formally approved.*

A.C. Agarwal	Brampton, Ontario	*Chair*
A. Au	Ontario Ministry of Transportation, St. Catharines, Ontario	*Secretary*
D. Becker	Golder Associates Ltd., Calgary, Alberta	
D.P. Gagnon	Buckland & Taylor Ltd., North Vancouver, British Columbia	
H. Hong	University of Western Ontario, London, Ontario	
P. King	University of Western Ontario, London, Ontario	
C. Lam	Ontario Ministry of Transportation, St. Catharines, Ontario	
D. Mitchell	McGill University, Montréal, Québec	
A.S. Nowak	University of Nebraska, Lincoln, Nebraska, USA	
I. Smith	University of New Brunswick, Fredericton, New Brunswick	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Editorial Task Force

M. Erki	Royal Military College of Canada, Kingston, Ontario	*Chair*
B. Bakht	JMBT Structures Research Inc., Toronto, Ontario	
L.G. Jaeger	Halifax, Nova Scotia	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

French Translation Task Force (CAN/CSA-S6-06)

Note: *This list reflects the French Translation Task Force membership when CAN/CSA-S6-06 was formally approved.*

G. Desgagné	Ministère des transports du Québec, Québec, Québec	*Chair*
L.M. Bélanger	Ministère des transports du Québec, Québec, Québec	
D. Bernard	Ministère des transports du Québec, Québec, Québec	
M. Blouin	Ministère des transports du Québec, Québec, Québec	
J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
J. Labbé	Ministère des transports du Québec, Québec, Québec	
M. Lacroix	Ministère des transports du Québec, Québec, Québec	
J. Prévost	Ministère des transports du Québec, Québec, Québec	
M. Savard	Ministère des transports du Québec, Québec, Québec	
M. Vallières	Ministère des transports du Québec, Québec, Québec	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ French Translation Task Force (CSA S6S1-10)

Note: *This list reflects the French Translation Task Force membership when CSA S6S1-10 was formally approved.*

G. Desgagné	Ministère des transports du Québec, Québec, Québec	*Chair*
L.M. Bélanger	Ministère des transports du Québec, Québec, Québec	
D. Bernard	Ministère des transports du Québec, Québec, Québec	
M. Blouin	Ministère des transports du Québec, Québec, Québec	
D. Boulet	Ministère des transports du Québec, Québec, Québec	
J.P. Grenier	Ministère des transports du Québec, Québec, Québec	
J. Labbé	Ministère des transports du Québec, Québec, Québec	
M. Lacroix	Ministère des transports du Québec, Québec, Québec	
J. Prévost	Ministère des transports du Québec, Québec, Québec	
M. Savard	Ministère des transports du Québec, Québec, Québec	
M. Talbot	Ministère des transports du Québec, Québec, Québec	
M. Vallières	Ministère des transports du Québec, Québec, Québec	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Regulatory Authority Committee (CAN/CSA-S6-06)

Note: *This list reflects the Regulatory Authority Committee membership when CAN/CSA-S6-06 was formally approved.*

G. Richard	Ministère des transports du Québec, Québec, Québec	*Chair*
J. Francis	Nova Scotia Department of Transportation and Public Works, Halifax, Nova Scotia	*Vice-Chair*
T.B. Tharmabala	Ontario Ministry of Transportation, St. Catharines, Ontario	*Vice-Chair*
B. Bakht	JMBT Structures Research Inc., Toronto, Ontario	*Associate*
G.P. Carlin	Les Ponts Jacques Cartier et Champlain Incorporée, Longueuil, Québec	
C. Clarke	Alberta Infrastructure and Transportation, Edmonton, Alberta	
D. Cogswell	New Brunswick Department of Transportation, Fredericton, New Brunswick	
D.J. Evans	Prince Edward Island Department of Transportation and Public Works, Charlottetown, Prince Edward Island	
A. Ikpong	Yukon Department of Highways and Public Works, Whitehorse, Yukon	
D. Macleod	Public Works and Government Services Canada, Gatineau, Québec	
R. Mathieson	British Columbia Ministry of Transportation, Victoria, British Columbia	
D. Power	Newfoundland and Labrador Department of Transportation and Works, St. John's, Newfoundland and Labrador	
R. Richardson	Manitoba Department of Highways and Transportation, Winnipeg, Manitoba	
D.R. Sitland	Government of Nunavut, Iqaluit, Nunavut	
T. Williams	Government of the Northwest Territories, Yellowknife, Northwest Territories	

H. Yea	Saskatchewan Highways and Transportation, Regina, Saskatchewan	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Δ Regulatory Authority Committee (CSA S6S1-10)

Note: *This list reflects the Regulatory Authority Committee membership when CSA S6S1-10 was formally approved.*

T.B. Tharmabala	Ontario Ministry of Transportation, St. Catharines, Ontario	*Chair*
G. Desgagné	Ministère des transports du Québec, Québec, Québec	*Vice-Chair*
R. Mathieson	British Columbia Ministry of Transportation and Infrastructure, Victoria, British Columbia	*Vice-Chair*
L. Atkin	Alberta Transportation, Edmonton, Alberta	
D. Cogswell	New Brunswick Department of Transportation, Fredericton, New Brunswick	
R. Eden	Manitoba Department of Infrastructure and Transportation, Winnipeg, Manitoba	
S. Eisan	Nova Scotia Department of Transportation and Infrastructure Renewal, Halifax, Nova Scotia	
D.J. Evans	Prince Edward Island Department of Transportation and Infrastructure Renewal, Charlottetown, Prince Edward Island	
G. Hewas	The Federal Bridge Corporation, Ottawa, Ontario	
A. Ikpong	Yukon Department of Highways and Public Works, Whitehorse, Yukon	
A. Lanteigne	Government of the Northwest Territories, Yellowknife, Northwest Territories	
D. Power	Newfoundland and Labrador Department of Transportation and Works, St. John's, Newfoundland and Labrador	
H. Yea	Saskatchewan Highways and Transportation, Regina, Saskatchewan	
M. Braiter	CSA, Mississauga, Ontario	*Project Manager*

Preface

This is the tenth edition of CAN/CSA-S6, *Canadian Highway Bridge Design Code*. It supersedes the previous edition published in 2000, which amalgamated and superseded CAN/CSA-S6-88, *Design of Highway Bridges*, and the Ontario Ministry of Transportation's OHBDC-91-01, *Ontario Highway Bridge Design Code*, 3rd ed. Earlier editions of the CSA Standard were published in 1978, 1974, 1966, 1952, 1938, 1929, and 1922. Earlier editions of the *Ontario Highway Bridge Design Code* were published in 1983 and 1979 by the Ontario Ministry of Transportation.

This Code uses the limit states design approach and reflects current design conditions across Canada as well as research activity since the publication of the previous edition. Several design aspects are addressed for the first time in this edition and a more detailed treatment of many areas is provided. This Code has been written to be applicable in all provinces and territories.

As in the previous edition, there are 16 Sections in this Code.

Section 1 ("General") specifies general requirements and includes definitions and a reference publications clause applicable throughout this Code. It also specifies geometric requirements, based in part on the Transportation Association of Canada's *Geometric Design Guide for Canadian Roads* (1999), and hydraulic design requirements, based in part on the Transportation Association of Canada's *Guide to Bridge Hydraulics*, 2nd ed. (2001). There are also provisions covering durability, economics, environmental considerations, aesthetics, safety, maintenance, and maintenance inspection access.

Section 2 ("Durability") addresses durability aspects of materials used in the construction of highway bridges, culverts, and other structures located in transportation corridors. The durability requirements for all of the materials are based on common principles applicable to the deterioration mechanisms for each material, the environmental conditions to which the materials are subjected, and the protective measures and detailing requirements needed to limit deterioration to acceptable levels.

Section 3 ("Loads") specifies loading requirements for the design of new bridges, including requirements for permanent loads, live loads, and miscellaneous transitory and exceptional loads (but excluding seismic loads). The 625 kN truck load model and corresponding lane load model are specified as the minima for interprovincial transportation and are based on current Canadian legal loads. Ship collision provisions are also included in Section 3.

Section 3 no longer specifies limits on the span lengths for application of the truck and lane loads. Accordingly, long-span requirements have been developed and appear in Section 3 and elsewhere in this Code (these requirements, however, should not be considered comprehensive). Section 3 covers long-span live loading and addresses wind tunnel testing for aerodynamic effects.

Section 4 ("Seismic design") specifies seismic design requirements for new bridges. These are based primarily on AASHTO (American Association of State Highway and Transportation Officials) LRFDEM-3-M, *AASHTO LRFD Bridge Design Specifications*, 3rd ed. (2005). Section 4 differs from AASHTO LRFDEM-3-M, however, by providing a more extensive treatment of the importance and response modification factors, new design and detailing requirements for structural steel ductile substructure elements, and design provisions for seismic base isolation. Section 4 also includes design provisions for the seismic evaluation of existing bridges and provisions pertaining to the seismic rehabilitation of existing bridges.

Section 5 ("Methods of analysis") specifies requirements for analyzing the basic superstructure of a bridge. In its methods for simplified analysis of bridge superstructures, a bridge is treated as a single beam and force effects are averaged over the width of the bridge and subsequently amplified to calculate the true intensity. Distribution factors are based on research conducted up to the late 1990s. Simplified elastic methods are included for the analysis of transverse effects. Refined methods of analysis for short-, medium-, and long-span bridges are also addressed.

Section 6 ("Foundations") is based, as in the previous edition, on the use of global resistance factors. Section 6 employs the limit states design approach, in which the term "resistance" is applied to the strength or capacity of the soil or rock at the ultimate limit state and the term "reaction" is associated with the serviceability limit state and is indicative of a particular deformation. Section 6 also emphasizes the importance of communication between the bridge structural engineer and the geotechnical engineer at all stages of a project.

Section 7 (“Buried structures”) deals with soil-metal structures with shallow corrugated plates in which thrust is the dominant force in the metal plates as well as soil-metal structures with deep corrugated plates and metal box structures in which flexural effects are also considered in the design of the metal plates. Provisions have been added for reinforced concrete precast and cast-in-place structures, including pipes, box sections, and segmental structures. Section 7 also specifies requirements for determining the properties and dimensions of the engineered soil and non-soil components and addresses construction supervision and construction procedures for soil components.

Section 8 (“Concrete structures”) covers reinforced and partially and fully prestressed concrete components (including deck slabs) made of normal-density, semi-low-density, and high-density concrete of a strength varying from 30 to 80 MPa. Compression field theory is used for proportioning for shear and for torsion combined with flexure. The strut-and-tie approach is used for proportioning regions where the plane sections assumption is not applicable.

Section 9 (“Wood structures”) specifies properties for materials and fastenings that are consistent with CAN/CSA-O86-01, *Engineering Design in Wood*, and includes data for structural composite lumber. Its provisions related to shear and compression, load distribution, design factors (in many cases), and laminated wood decks are essentially unchanged from those of the previous edition. The shear force concept has been reintroduced for the shear design of sawn wood members and the specified strength values in shear have been increased, in accordance with CAN/CSA-O86-01. In addition, the factors for load-sharing systems have changed.

Section 10 (“Steel structures”) specifies the majority of this Code’s design requirements for steel structures (with the exception of some seismic requirements specified in Section 4). Construction requirements that can have an impact on the resistance factors used in Section 10 are specified in Clause 10.24. Because this Code has been expanded to include long-span bridges, cables and arches are now dealt with. In addition, durability is now addressed much more fully and clauses dealing with beams and girders, composite beams and girders, horizontally curved girders, orthotropic decks, fatigue, and construction have been revised.

Section 11 (“Joints and bearings”) covers the deck joints and bearings most commonly used in Canada.

Section 12 (“Barriers and highway accessory supports”) specifies crash test requirements for barriers and breakaway highway accessory supports. Crash testing may be waived for barrier and accessory support designs that have a successful in-service performance record. Performance levels alternative to those specified in Section 12 are permitted if approved by the regulatory authority.

Section 13 (“Movable bridges”) specifies requirements for the design, construction, and operation of conventional movable bridges. Although the structural design aspects are based on the limit states design approach, the mechanical systems design aspects follow the working stress principle used in North American industry.

Section 14 (“Evaluation”) includes new provisions concerning the three-level evaluation system, evaluation of deck slabs, and detailed evaluation from bridge testing. The provisions on the strength of wood members and the shear resistance of concrete have been improved. Another category of permit vehicle (Permit — Annual or project [PA]) has been added. An optional probability-based mean load method that uses site-specific load and resistance information for more accurate evaluation is also provided. As in the previous edition, a more conventional approach to determining material grades from small samples is used in place of the Baye’s theorem approach in CAN/CSA-S6-88.

Section 15 (“Rehabilitation and repair”) specifies rehabilitation design requirements and provides guidance on the selection of loads and load factors for rehabilitation that is based on the intended use of the bridge following rehabilitation.

Section 16 (“Fibre-reinforced structures”) specifies design requirements for a limited number of structural components containing either high- or low-modulus fibres. The high-modulus fibres (aramid, carbon, and glass) are employed in fibre-reinforced polymers (FRPs), which are used as replacements for steel bars and tendons. The low-modulus fibres are used for controlling cracks in concrete. Section 16 covers concrete beams and slabs, concrete deck slabs, and stressed wood decks. In this edition, Section 16 includes new design provisions that permit glass-fibre-reinforced polymer to be used as primary reinforcement and as tendons in concrete.

Other new provisions in Section 16 permit the rehabilitation of concrete and timber structures using externally bonded FRP systems or near-surface-mounted reinforcement. The provisions concerning fibre-reinforced concrete deck slabs in the previous edition have been reorganized for this edition to include deck slabs of both cast-in-place and precast construction, which are now referred to as "externally restrained deck slabs", whereas deck slabs containing internal FRP reinforcement are referred to as "internally restrained deck slabs". The resistance factors of the previous edition have been revised and depend on conditions of use, with a further distinction made between factory- and field-produced FRPs. The previous edition's deformability requirements for FRP-reinforced and FRP-prestressed concrete beams and slabs are now dealt with in separate clauses that cover design for deformability, minimum flexural resistance, and crack control reinforcement. The effect of the sustained loads on the strength of FRPs is accounted for in this edition by limits on stresses in FRPs induced at the serviceability limit state. In addition, new stress limits for tendons have been introduced. The design for shear is now an adaptation of this Code's method for concrete structures and accounts for the decrease in shear carried by the concrete in FRP-reinforced beams. There are also modified provisions for barrier walls.

Funding for developing and publishing this Code was provided by the governments of Alberta, British Columbia, Manitoba, New Brunswick, Newfoundland and Labrador, the Northwest Territories, Nova Scotia, Nunavut, Ontario, Prince Edward Island, Québec, Saskatchewan, and Yukon, Public Works and Government Services Canada, and the Federal Bridge Corporation Limited. This Code could not have been developed without the cooperation of all of these sponsors.

This Code was prepared by the Technical Committee on the Canadian Highway Bridge Design Code, under the jurisdiction of the Strategic Steering Committee on Structures (Design), and has been formally approved by the Technical Committee. It has been approved as a National Standard of Canada by the Standards Council of Canada.

November 2006

Notes:

(1) *Use of the singular does not exclude the plural (and vice versa) when the sense allows.*

(2) *Although the intended primary application of this Code is stated in Clause 1.1.1, it is important to note that it remains the responsibility of the users of the Code to judge its suitability for their particular purpose.*

(3) *This publication was developed by consensus, which is defined by* CSA Policy governing standardization — Code of good practice for standardization *as "substantial agreement. Consensus implies much more than a simple majority, but not necessarily unanimity". It is consistent with this definition that a member may be included in the Technical Committee list and yet not be in full agreement with all clauses of this publication.*

(4) *CSA Codes and Standards are subject to periodic review, and suggestions for their improvement will be referred to the appropriate committee.*

(5) *All enquiries regarding this Code, including requests for interpretation, should be addressed to Canadian Standards Association, 5060 Spectrum Way, Suite 100, Mississauga, Ontario, Canada L4W 5N6.*

Requests for interpretation should

(a) define the problem, making reference to the specific clause, and, where appropriate, include an illustrative sketch;

(b) provide an explanation of circumstances surrounding the actual field condition; and

(c) be phrased where possible to permit a specific "yes" or "no" answer.

Committee interpretations are processed in accordance with the CSA Directives and guidelines governing standardization *and are published in CSA's periodical* Info Update*, which is available on the CSA Web site at www.csa.ca.*

Foreword

In Canada, the legal mandate for establishing design and construction requirements for highways, including highway bridges, lies with the provincial and territorial governments. Before the publication of the previous edition of this Code, Ontario regulated the design and construction of highway bridges through the *Ontario Highway Bridge Design Code*. All of the other provinces and territories used CAN/CSA-S6, *Design of Highway Bridges,* with the exception of Manitoba, which adopted the bridge code published by the American Association of State Highway and Transportation Officials (AASHTO). The previous edition of the *Canadian Highway Bridge Design Code* was developed to provide a state-of-the-art model design code that could be adopted by all provinces and territories. This new edition is intended to serve the same purpose.

Among the benefits associated with undertaking the development of this Code is the opportunity to establish safety and reliability levels for highway bridges that are consistent across all Canadian jurisdictions. Adoption of a single code makes it easier for the consulting and producer industries to respond to calls for proposals and eliminates the need for familiarity with the details of several codes. The adoption of a single code also supports the implementation of a national highway transportation system with agreed minimum standards and loadings for bridges on interprovincial highways, thereby encouraging consistency of vehicle weights across jurisdictions and supporting the objective of more cost-effective transportation of goods.

Designers need to be aware, however, that although this Code establishes CL-625 loading as the minimum for bridges that are part of the national highway system, it is within the mandate of the provinces and territories to adopt a heavier or lighter live loading based on local traffic conditions. For example, Ontario requires (as specified in Annex A3.4) the use of a CL-625-ONT loading in the design of new bridges; this reflects the higher average regulatory and observed loads for trucks operating in the province. All of the requirements of this Code applicable to CL-W loading also apply to CL-625-ONT loading. Designers should always obtain approval from the regulatory authority when a live loading other than the CL-625 loading is to be used for design, and should check whether any variations from the requirements of this Code are in effect in the jurisdiction, e.g., for evaluation of existing bridges or issuance of overload permits.

This Code was developed by taking into account the different regulatory structures and standards of Canada's provinces and territories. Overall priorities and objectives were established by the Regulatory Authority Committee (RAC), which also monitored the progress of the Code's development. In accordance with CSA procedural requirements, however, responsibility for the technical content of this Code was assigned to the Technical Committee (TC), as were decisions on how to deal with the priorities and objectives identified by the RAC. Because of the breadth and complexity of this Code, subcommittees (which were required to operate and report on a consensus basis) were established to oversee each section. In addition, task forces were established to handle specific aspects of this Code. The subcommittees and task forces reported to the TC through their Chairs. The extensive use of subcommittees permitted the recruitment of experts with the knowledge needed to address the sometimes highly specialized subjects covered by this Code.

The developers of this Code wish to acknowledge the contributions of the following individuals, who were unable to complete their terms on the RAC and TC: Ismail Elkholy (RAC and former Vice-Chair of TC) (Manitoba Department of Highways and Transportation) and Peter Lester (RAC and TC) (Newfoundland and Labrador Department of Transportation and Works).

This Code is complemented by CSA S6.1-06, *Commentary on CAN/CSA-S6-06,* Canadian Highway Bridge Design Code, which provides rationale statements and explanatory material for many of the clauses of this Code. Although this Code is being published in both English and French, CSA S6.1 is available only in English.

A 490M-04a
Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints [Metric]

A 510-03
Standard Specification for General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel

A 586-04a
Standard Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand

Δ A 588/A 588M-05
Standard Specification for High-Strength Low-Alloy Structural Steel, up to 50 ksi [345 MPa] Minimum Yield Point, with Atmospheric Corrosion Resistance

A 603-98 (R2003)
Standard Specification for Zinc-Coated Steel Structural Wire Rope

A 641/A 641M-03
Standard Specification for Zinc-Coated (Galvanized) Carbon Steel Wire

A 653/A 653M-05
Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Alloy-Coated (Galvannealed) by the Hot-Dip Process

A 668/A 668M-04
Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use

A 675/A 675M-03 e1
Standard Specification for Steel Bars, Carbon, Hot-Wrought, Special Quality, Mechanical Properties

A 709/A 709M-05
Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges

Δ A 722/A 722M-07
Standard Specification for Uncoated High-Strength Steel Bars for Prestressing Concrete

B 22-02
Standard Specification for Bronze Castings for Bridges and Turntables

B 36/B 36M-01
Standard Specification for Brass Plate, Sheet, Strip, and Rolled Bar

B 121/B 121M-01
Standard Specification for Leaded Brass Plate, Sheet, Strip, and Rolled Bar

B 746/B 746M-02
Standard Specification for Corrugated Aluminum Alloy Structural Plate for Field-Bolted Pipe, Pipe-Arches, and Arches

Δ C 78-09
Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)

C 506M-05
Standard Specification for Reinforced Concrete Arch Culvert, Storm Drain, and Sewer Pipe [Metric]

C 507M-05
Standard Specification for Reinforced Concrete Elliptical Culvert, Storm Drain, and Sewer Pipe [Metric]

C 567-05
Standard Test Method for Determining Density of Structural Lightweight Concrete

Δ C 1399-07
Standard Test Method for Obtaining Average Residual-Strength of Fiber-Reinforced Concrete

C 1433-04 e1
Standard Specification for Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers [Metric]

D 395-03
Standard Test Methods for Rubber Property — Compression Set

D 412-98a (2002) e1
Standard Test Methods for Vulcanized Rubber and Thermoplastic Elastomers — Tension

D 429-03
Standard Test Methods for Rubber Property — Adhesion to Rigid Substrates

D 573-04
Standard Test Method for Rubber — Deterioration in an Air Oven

D 698-00 ae1
Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))

D 746-04
Standard Test Method for Brittleness Temperature of Plastics and Elastomers by Impact

D 1149-99
Standard Test Method for Rubber Deterioration-Surface Ozone Cracking in a Chamber

D 1557-02 e1
Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort (56,000 ft-lbf/ft^3 (2,700 kN-m/m^3))

D 2239-03
Standard Specification for Polyethylene (PE) Plastic Pipe (SIDR-PR) Based on Controlled Inside Diameter

D 2240-04 e1
Standard Test Method for Rubber Property — Durometer Hardness

D 2487-00
Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)

D 3350-04
Standard Specification for Polyethylene Plastics Pipe and Fittings Materials

D 4541-02
Standard Test Method for Pull-Off Strength of Coatings Using Portable Adhesion Testers

D 4894-04
Standard Specification for Polytetrafluoroethylene (PTFE) Granular Molding and Ram Extrusion Materials

D 5456-05
Standard Specification for Evaluation of Structural Composite Lumber Products

F 436-04
Standard Specification for Hardened Steel Washers

F 568M-04
Standard Specification for Carbon and Alloy Steel Externally Threaded Metric Fasteners

Δ F 1852-08
Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength

Δ F 2280-08
Standard Specification for "Twist Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated 150 ksi Minimum Tensile Strength

PS 62-97 (withdrawn Provisional Standard)
Provisional Standard Specification for Precast Reinforced Concrete Box Sections for Culverts, Storm Drains, and Sewers

AWPA (American Wood-Preservers' Association)
C33-03
Standard for Preservative Treatment of Structural Composite Lumber by Pressure Processes

Government of Canada
Navigable Waters Protection Act, RSC 1985, c. N-22

NCHRP (National Cooperative Highway Research Program)
Report 230 (1980)
Recommended Procedures for the Safety Performance Evaluation of Highway Appurtenances

Report 350 (1993)
Recommended Procedures for the Safety Performance Evaluation of Highway Features

NEMA (National Electrical Manufacturers Association)
ICS9-1993 (withdrawn Standard)
Industrial Control and Systems: Power Circuit Accessories

NLGA (National Lumber Grades Authority)
Standard Grading Rules for Canadian Lumber (2003)

NRCC (National Research Council Canada)
National Building Code of Canada, 2005

National Fire Code of Canada, 2005

Research Council on Structural Connections
Specification for Structural Joints Using ASTM A325 or A490 Bolts (2004)

Transportation Association of Canada
Geometric Design Guide for Canadian Roads (1999)

Guide to Bridge Hydraulics, 2nd ed. (2001)

Manual of Uniform Traffic Control Devices for Canada (1998)

UL (Underwriters Laboratories Inc.)
845 (1995)
Standard for Motor Control Centers

U.S. Department of Defense
MIL-S-8660C (1999) (cancelled Specification)
Silicone Compound

1.3 Definitions

1.3.1 General

The definitions in Clauses 1.3.2 to 1.3.4 apply in this Code.

Note: *Additional definitions are found in Sections 2 to 16. In the case of a conflict between a definition in Sections 2 to 16 and a definition in Clauses 1.3.2 to 1.3.4, the definition in Sections 2 to 16 takes precedence.*

1.3.2 General administrative definitions

Note: *The general administrative terms defined in this Clause are capitalized wherever they are used in this Code in their defined sense.*

Approval or **Approved** — approval, or approved, in writing by the Regulatory Authority.

Checker — a member or licensee of the Engineering Association who carries out the design check, rehabilitation design check, or evaluation check of a bridge or structure.

Construction — the construction, reconstruction, rehabilitation, repair, or demolition of a structure.

Constructor —
(a) a Person that contracts to perform all of the Construction work on a project;
(b) an Owner that contracts with two or more Persons for such Persons to perform part of the Construction work on a project; or
(c) an Owner that performs all or part of the Construction work on a project.

Engineer — a member or licensee of the Engineering Association who carries out the design, rehabilitation design, or evaluation of a bridge or structure.

Engineering Association — an organization authorized by charter to regulate the profession of engineering in a province or territory.

Owner — the Person having responsibility for and control of a bridge or structure.

Person — an individual, board, commission, partnership, or corporation (including a municipal corporation) and his, her, or its agents, successors, and assignees.

Plans — the drawings, documents, and specifications that define a Construction project, form part of the contract documents, or are included in the contract documents by reference; all Approved drawings and descriptions produced by a Constructor for the Construction of a bridge or other structure; and all revisions to the items described in this definition.

Regulatory Authority — the federal, provincial, or territorial Minister having governmental jurisdiction and control, his or her nominee, or the local authority to whom this authority is delegated.

1.3.3 General technical definitions

Abutment — a substructure that supports the end of a superstructure and retains some or all of the bridge approach fill.

Arterial road — an arterial road as defined in the Transportation Association of Canada's *Geometric Design Guide for Canadian Roads.*

Auxiliary component — a component of a structural system that does not constitute part of the intended load-sharing system. Auxiliary components include expansion joints, approach slabs, railings and barriers, and deck drains.

Average annual daily traffic — the total volume of traffic during a year divided by the number of days in the year.

Δ

Table 3.1
Load factors and load combinations

(See Clauses 3.5.1, 3.10.1.1, 3.10.5.2, 3.13, 3.16.3, 4.10.7, 4.10.10.1, 7.6.3.1.1, 7.7.3.1.1, 9.4.2, and 15.6.2.4.)

	Permanent loads			Transitory loads					Exceptional loads			
Loads	***D***	***E***	***P***	***L******	***K***	***W***	***V***	***S***	***EQ***	***F***	***A***	***H***
Fatigue limit state												
FLS Combination 1	1.00	1.00	1.00	1.00	0	0	0	0	0	0	0	0
Serviceability limit states												
SLS Combination 1	1.00	1.00	1.00	0.90	0.80	0	0	1.00	0	0	0	0
SLS Combination 2†	0	0	0	0.90	0	0	0	0	0	0	0	0
Ultimate limit states‡												
ULS Combination 1	α_D	α_E	α_P	1.70††	0	0	0	0	0	0	0	0
ULS Combination 2	α_D	α_E	α_P	1.60	1.15	0	0	0	0	0	0	0
ULS Combination 3	α_D	α_E	α_P	1.40	1.00	0.45§	0.45	0	0	0	0	0
ULS Combination 4	α_D	α_E	α_P	0	1.25	1.50§	0	0	0	0	0	0
ULS Combination 5	α_D	α_E	α_P	0	0	0	0	0	1.00	0	0	0
ULS Combination 6**	α_D	α_E	α_P	0	0	0	0	0	0	1.30	0	0
ULS Combination 7	α_D	α_E	α_P	0	0	0.80§	0	0	0	0	1.30	0
ULS Combination 8	α_D	α_E	α_P	0	0	0	0	0	0	0	0	1.00
ULS Combination 9	1.35	α_E	α_P	0	0	0	0	0	0	0	0	0

**For the construction live load factor, see Clause 3.16.3.*

†*For superstructure vibration only.*

‡*For ultimate limit states, the maximum or minimum values of* α_D, α_E, *and* α_P *specified in Table 3.2 shall be used.*

§*For wind loads determined from wind tunnel tests, the load factors shall be as specified in Clause 3.10.5.2.*

***For long spans, it is possible that a combination of ice load F and wind load W will require investigation.*

††*Also to be applied to the barrier loads.*

Legend:

A = ice accretion load
D = dead load
E = loads due to earth pressure and hydrostatic pressure, including surcharges but excluding dead load
EQ = earthquake load
F = loads due to stream pressure and ice forces or to debris torrents
H = collision load arising from highway vehicles or vessels, excluding barrier loads
K = all strains, deformations, and displacements and their effects, including the effects of their restraint and the effects of friction or stiffness in bearings. Strains and deformations include strains and deformations due to temperature change and temperature differential, concrete shrinkage, differential shrinkage, and creep, but not elastic strains
L = live load (including the dynamic load allowance, when applicable), including barrier loads
P = secondary prestress effects
S = load due to differential settlement and/or movement of the foundation
V = wind load on traffic
W = wind load on structure

Table 3.2
Permanent loads — Maximum and minimum values of load factors for ULS
(See Clauses 3.5.1, 3.5.2.1, 4.4.1, 4.4.9.3, and 7.8.7.1 and Table 3.1.)

Dead load	**Maximum α_D**	**Minimum α_D**
Factory-produced components, excluding wood	1.10	0.95
Cast-in-place concrete, wood, and all non-structural components	1.20	0.90
Wearing surfaces, based on nominal or specified thickness	1.50	0.65
Earth fill, negative skin friction on piles	1.25	0.80
Water	1.10	0.90
Dead load in combination with earthquakes	**Maximum α_D**	**Minimum α_D**
All dead loads for ULS Combination 5 (see Table 3.1)	1.25	0.80
Earth pressure and hydrostatic pressure	**Maximum α_E**	**Minimum α_E**
Passive earth pressure, considered as a load*	1.25	0.50
At-rest earth pressure	1.25	0.80
Active earth pressure	1.25	0.80
Backfill pressure	1.25	0.80
Hydrostatic pressure	1.10	0.90
Prestress	**Maximum α_P**	**Minimum α_P**
Secondary prestress effects	1.05	0.95

**When passive earth pressure is considered as a resistance, it is factored in accordance with Section 6.*

3.5.2 Permanent loads

3.5.2.1 General

Total factored load effects shall include those effects due to all permanent loads acting on the structure.

For ULS loading combinations, the maximum or minimum value specified in Table 3.2 for each load factor shall be used to maximize each total factored load effect. However, it is not normally necessary to consider load factors having different values in different spans.

Except as required by Clause 3.5.2.2, the minimum values for load factors specified in Table 3.2 shall not be used for some loads together with maximum values for other loads when the possibility of these loads having minimum and maximum values simultaneously can safely be excluded.

3.5.2.2 Overturning and sliding effects

When the maximum value of α_E for active pressure is used in calculating overturning for cantilever earth-retaining structures and for horizontal sliding, a value of 1.00 shall be used for α_D.

In calculating backfill pressures that oppose one another or reduce load effects within a structure, all combinations of maximum and minimum earth pressure load factors shall be considered.

In calculating overturning moments occurring during balanced cantilever construction of segmental concrete bridges, maximum and minimum values of α_D equal to 1.05 and 1.0, respectively, may be used for the erected segments provided that construction controls are specified to ensure that the difference in weight between any two segments forming a balancing pair does not exceed 5%, and that all differences in weight are corrected before the addition of further segments.

3.10 Wind loads

3.10.1 General

3.10.1.1 General

Clause 3.10 specifies design wind loads for all highway structures and provides specific requirements for bridge substructures and superstructures. Guidance for determining the tendency toward aeroelastic instability in wind-sensitive bridges is also provided. All wind loads based on the reference wind pressure, q, shall be treated as equivalent static loads.

Special requirements for wind tunnel testing are specified in Clause 3.10.5, with reference to the determination of specific load factors to replace those specified in Table 3.1 for wind load effects.

Annex A3.2 specifies requirements for wind loads on highway accessory supports, barriers, and slender structural elements, including the effects of vortex shedding.

3.10.1.2 Reference wind pressure

The hourly mean reference wind pressure, q, shall be as specified in Table A3.1.1 for a return period of
(a) 100 years for bridge structures with any span 125 m long or longer;
(b) 50 years for bridge structures with a maximum span shorter than 125 m, luminaire support structures higher than 16 m, and overhead sign structures;
(c) 25 years for luminaire and traffic signal support structures 16 m high or shorter, and for barriers; and
(d) 10 years for roadside sign structures where a long life expectancy is not required, or for any of the structures specified in Items (a) to (c) during construction.

If the topography at the structure site can cause funnelling of the wind, the reference wind pressure shall be increased by 20%.

3.10.1.3 Gust effect coefficient

For highway bridges that are not sensitive to wind action (which includes most bridges of spans less than 125 m except those that are cable supported), the gust effect coefficient, C_g, shall be taken as 2.0.

For slender, lighter structures, e.g., pedestrian bridges, luminaire, sign, and traffic signal supports, barriers, and slender structural elements, C_g shall be taken as 2.5.

For structures that are sensitive to wind action, the gust factor approach shall not be used and the wind loads shall be determined on the basis of a detailed analysis of dynamic wind action, using an Approved method that includes the effects of buffeting.

3.10.1.4 Wind exposure coefficient

The wind exposure coefficient, C_e, shall not be less than 1.0 and shall be taken from Table 3.8 or calculated as $(0.10H)^{0.2}$, where H is the height above ground of the top of the superstructure. For luminaire, sign, and traffic signal supports, and for barriers, H shall be taken to the top of the standard, support, or structure considered. The height above ground shall be measured from the foot of cliffs, hills, or escarpments when the structure is located in uneven terrain, or from the low water level for structures over water.

Table 3.8
Wind exposure coefficient, C_e
(See Clause 3.10.1.4.)

Height above ground of the top of the superstructure, H, m	Wind exposure coefficient, C_e
0 to 10	1.0
Over 10 to 16	1.1
Over 16 to 25	1.2
Over 25 to 37	1.3
Over 37 to 54	1.4
Over 54 to 76	1.5
Over 76 to 105	1.6

3.10.1.5 Non-uniform loading
Wind loads shall be applied uniformly or non-uniformly over the entire structure, whichever produces the more critical effects. Unless an analysis of non-uniform wind loads specific to the structure is undertaken, the non-uniform loading shall be 0.75 times the effective uniformly distributed load over any portion of the structure and the full effective uniformly distributed load applied over the remaining portion.

3.10.1.6 Overturning and overall stability
When the prescribed loads in the design of members are being applied, overturning, uplift, and lateral displacement shall be considered.

3.10.1.7 Alternative methods
When Approved, representative wind tunnel tests or more detailed methods of analysis may be used to establish load coefficients or design criteria different from those specified in this Section.

Wind loads derived from the results of wind tunnel tests shall be used with wind load factors determined in accordance with Clause 3.10.5.2.

3.10.2 Design of the superstructure

3.10.2.1 General
The superstructure shall be designed for wind-induced vertical and horizontal drag loads acting simultaneously. The assumed wind direction shall be perpendicular to the longitudinal axis for a straight structure or to an axis chosen to maximize wind-induced effects for a structure curved in plan.

Δ 3.10.2.2 Horizontal drag load
The following wind load per unit exposed frontal area of the superstructure shall be applied horizontally:

$$F_h = qC_eC_gC_h$$

where q, C_e, and C_g are as specified in Clauses 3.10.1.2, 3.10.1.4, and 3.10.1.3, respectively, and $C_h = 2.0$.

In the case of truss spans, this load shall be taken to act on the windward truss simultaneously with a load on the leeward truss equal to the load on the windward truss in the through-trusses and 75% of the load on the windward truss in other trusses unless a recognized method is used to calculate the shielding effect of the windward truss.

3.10.2.3 Vertical load
The following wind load per unit exposed plan area of the superstructure shall be applied vertically:

$$F_v = qC_eC_gC_v$$

3.10.5 Wind tunnel tests

3.10.5.1 General

Structures that are sensitive to wind include those that are flexible, slender, lightweight, long span, or of unusual geometry. For such structures, supplementary studies by an expert in the field should be conducted, and it is possible that wind tunnel tests will be required. Representative wind tunnel tests may be used to satisfy the requirements of Clauses 3.10.4.1 and 3.10.4.2. These tests may also be used to establish the components of the overall structural loads specified in Clauses 3.10.2 and 3.10.3.

Δ 3.10.5.2 Load factors

If the overall structural loads due to wind are determined using wind tunnel tests, the load factor for wind, α_w, in ULS Combination 4 shall be calculated as

$$\alpha_w = 0.80\delta_w \exp(3.5kV_w)$$

where

$$k = \sqrt{\frac{V_w^2}{0.15^2 + V_w^2}}$$

The bias coefficient, δ_w, and coefficient of variation, V_w, of the wind load effect shall be determined by the persons responsible for the wind tunnel tests and shall account for the bias and uncertainty of the reference wind pressure, the gust, the pressure and exposure coefficients, and the uncertainty of the modelling.

Wind load factors for design in the ULS Combination 3 and ULS Combination 7 shall be the product of the factor specified in Table 3.1 and the ratio ($\alpha_w/1.50$).

3.11 Water loads

3.11.1 General

Local conditions at the site shall be considered in all cases.

3.11.2 Static pressure

Static water pressure shall be assumed to act perpendicular to the surface that is retaining the water. The pressure of water at a specific point shall be calculated as the product of the height of water above that point and the density of water.

3.11.3 Buoyancy

The effects of immersion in water or exposure to water pressure shall be considered. The beneficial effects of buoyancy shall be included, provided that they are always in existence. The non-beneficial effects of buoyancy shall be included unless the possibility of their occurrence can be excluded with certainty.

Buoyancy shall be taken as the vertical components of the static forces as calculated in accordance with Clause 3.11.2. Buoyancy shall be considered as an uplift force equivalent to the volume of water displaced.

3.11.4 Stream pressure

3.11.4.1 Longitudinal effects

The load due to flowing water acting longitudinally on a substructure element, P, shall be taken as $C_D \rho A v^2/2$, where the longitudinal drag coefficient, C_D, is as specified in Table 3.10.

Table 3.10
Longitudinal drag coefficient, C_D
(See Clause 3.11.4.1.)

Upstream shape of pier	Longitudinal drag coefficient, C_D
Semi-circular nosed	0.7
Square ended	1.4
Wedge nosed at ≤ 90°	0.8
Pier with debris lodged	1.4

3.11.4.2 Lateral effects

The lateral load due to water flowing at angle, θ, against a substructure element, P_p, shall be taken as $C_L \rho H L v^2/2$, where the lateral load coefficient, C_L, is as specified in Table 3.11.

Table 3.11
Lateral load coefficient, C_L
(See Clause 3.11.4.2.)

Angle, θ, between direction of flow and longitudinal pier axis, degrees	Lateral load coefficient, C_L
0	0.0
5	0.5
10	0.7
20	0.9
≥ 30	1.0

3.11.5 Wave action

Force effects due to wave action on bridge substructure elements exposed to environments where significant wave action can occur shall be evaluated in accordance with site-specific conditions. In the absence of such evaluations, the force against a flat surface substructure element, F_w, due to wave action, as a function of the wave height, H_w, shall be taken as $10H_w^2$.

F_w shall be considered to act at mid-height of the wave, $H_w/2$, above the still water elevation. For aerodynamically curved frontal surfaces, a value of $F_w/2$ shall be used.

3.11.6 Scour action

Local conditions and past records of floods shall be consulted in designing foundation elements when scour is expected to occur. The requirements of Sections 1 and 6 shall be applied. Changes in foundation conditions resulting from the design flood shall be considered at serviceability and ultimate limit states.

3.11.7 Debris torrents

Debris torrent loads shall be considered on exposed superstructures and substructures in accordance with site-specific conditions. Sites subject to heavy rainfall of short duration, earthquakes, landslides, and rockfalls shall be investigated for debris torrents when the following conditions exist:
(a) the creek channel gradient is greater than 25° for an extended length along the channel profile;
(b) boulders and debris exist in the channel;
(c) there is a history of such events.

An expert in the field shall be consulted to determine debris torrent loads.

Annex A3.1 (normative) **Climatic and environmental data**

Notes:

(1) *This Annex is a mandatory part of this Code.*

(2) *See Annex CA3.1 of CSA S6.1 for the sources and derivation of the data presented in this Annex.*

Table A3.1.1 Reference wind pressure and seismic zoning

(See Clauses 3.10.1.2, 3.13, and 4.4.3.)

Location	Hourly mean wind pressure, Pa, for return periods of				Velocity-related seismic zone, Z_v	Zonal velocity ratio, V	Acceleration-related seismic zone, Z_a	Zonal acceleration ratio, A
	10 yr	25 yr	50 yr	100 yr				
British Columbia								
Abbotsford	415	530	620	710	4	0.20	4	0.20
Agassiz	570	675	755	840	3	0.15	3	0.15
Alberni	470	560	630	700	5	0.30	5	0.30
Ashcroft	280	340	385	430	2	0.10	1	0.05
Beatton River	220	265	300	340	1	0.05	0	0.00
Burns Lake	305	355	390	430	3	0.15	1	0.05
Cache Creek	285	340	380	430	2	0.10	1	0.05
Campbell River	455	560	640	720	6	0.40	6	0.40
Carmi	245	315	375	440	1	0.05	1	0.05
Castlegar	225	285	335	390	1	0.05	1	0.05
Chetwynd	320	370	405	440	1	0.05	0	0.00
Chilliwack	475	605	715	830	4	0.20	4	0.20
Cloverdale	360	420	470	520	4	0.20	—	—
Comox	445	555	645	740	6	0.40	6	0.40
Courtenay	445	555	645	740	6	0.40	6	0.40
Cranbrook	225	280	325	370	1	0.05	1	0.05
Crescent Valley	225	280	325	370	1	0.05	1	0.05
Crofton	485	565	625	690	5	0.30	5	0.30
Dawson Creek	310	365	400	440	1	0.05	0	0.00
Dog Creek	310	365	400	440	2	0.10	1	0.05
Duncan	485	565	625	690	5	0.30	5	0.30
Elko	270	355	425	500	1	0.05	1	0.05
Fernie	325	410	480	550	1	0.05	1	0.05
Fort Nelson	210	255	285	310	1	0.05	0	0.00
Fort St. John	305	350	385	420	1	0.05	0	0.00
Glacier	240	285	315	350	1	0.05	1	0.05
Golden	270	310	345	380	1	0.05	1	0.05
Grand Forks	265	345	415	480	1	0.05	1	0.05
Greenwood	285	375	445	520	1	0.05	1	0.05
Haney	360	420	470	520	4	0.20	—	—
Hope	405	525	625	730	3	0.15	3	0.15
Kamloops	305	360	405	450	1	0.05	1	0.05
Kaslo	225	275	320	360	1	0.05	1	0.05
Kelowna	340	410	470	530	1	0.05	1	0.05
Kimberley	225	280	325	370	1	0.05	1	0.05
Kitimat Plant	380	440	485	520	4	0.20	2	0.10
Kitimat Townsite	380	440	485	520	4	0.20	2	0.10
Δ Langley	360	420	470	520	4	0.20	—	—

(Continued)

Table A3.1.1 (Continued)

Location	Hourly mean wind pressure, Pa, for return periods of 10 yr	25 yr	50 yr	100 yr	Velocity-related seismic zone, Z_v	Zonal velocity ratio, V	Acceleration-related seismic zone, Z_a	Zonal acceleration ratio, A
British Columbia *(continued)*								
Lillooet	315	380	430	490	2	0.10	1	0.05
Lytton	310	380	435	490	2	0.10	2	0.10
Mackenzie	245	285	315	350	2	0.10	0	0.00
Masset	490	565	625	690	6	0.40	6	0.40
McBride	275	315	350	380	1	0.05	0	0.00
McLeod Lake	245	285	315	350	2	0.10	0	0.00
Merritt	315	380	430	490	2	0.10	1	0.05
Mission City	465	580	675	770	4	0.20	4	0.20
Montrose	215	285	340	410	1	0.05	1	0.05
Nakusp	235	285	330	370	1	0.05	1	0.05
Nanaimo	470	560	635	710	4	0.20	4	0.20
Nelson	225	280	325	370	1	0.05	1	0.05
New Westminster	360	420	470	520	4	0.20	—	—
North Vancouver	360	430	480	530	4	0.20	—	—
Ocean Falls	465	535	595	650	4	0.20	2	0.10
100 Mile House	300	355	390	430	1	0.05	1	0.05
Osoyoos	300	405	495	590	1	0.05	1	0.05
Penticton	395	505	590	680	1	0.05	1	0.05
Port Alberni	470	560	630	700	5	0.30	5	0.30
Port Hardy	485	565	625	660	6	0.40	6	0.40
Port McNeill	485	565	625	680	6	0.40	6	0.40
Powell River	420	530	620	710	5	0.30	5	0.30
Prince George	280	335	370	410	2	0.10	0	0.00
Prince Rupert	420	485	535	590	5	0.30	3	0.15
Princeton	240	310	365	420	2	0.10	2	0.10
Qualicum Beach	460	560	640	720	4	0.20	4	0.20
Quesnel	250	285	310	340	2	0.10	0	0.00
Revelstoke	240	285	315	350	1	0.05	1	0.05
Richmond	360	430	480	530	4	0.20	—	—
Salmon Arm	285	340	380	430	1	0.05	1	0.05
Sandspit	535	615	680	740	6	0.40	6	0.40
Sidney	460	535	595	660	5	0.30	5	0.30
Smithers	315	365	400	440	3	0.15	1	0.05
Smith River	210	255	285	310	2	0.10	1	0.05
Squamish	380	480	560	650	3	0.15	3	0.15
Stewart	325	380	425	480	4	0.20	2	0.10
Taylor	315	365	400	440	1	0.05	0	0.00
Terrace	270	330	360	400	4	0.20	2	0.10
Tofino	540	615	675	740	5	0.30	5	0.30
Trail	260	315	350	390	1	0.05	1	0.05
Ucluelet	540	315	675	740	5	0.30	5	0.30
Vancouver	360	430	480	530	4	0.20	4	0.20
Vernon	315	380	430	490	1	0.05	1	0.05
Victoria	475	560	630	690	5	0.30	6	0.40
Williams Lake	295	340	375	410	2	0.10	1	0.05
Youbou	460	535	595	660	4	0.20	4	0.20
Alberta								
Athabasca	305	360	405	450	1	0.05	0	0.00
Banff	390	440	485	520	1	0.05	0	0.00
Barrhead	315	380	430	490	1	0.05	0	0.00

(Continued)

The design vessel for each pier or span element shall be chosen so that the annual frequency of collapse due to vessels that are equal in size to or larger than the design vessel is less than the acceptable risk criterion specified in Item (a) or (b), as applicable.

A3.3.3 Annual frequency of collapse

A3.3.3.1 General

The annual frequency of bridge component collapse due to vessel collision, *AF*, shall be taken as

$$AF = (N)(PA)(PG)(PC)$$

where

N = the annual number of vessels, classified by type, size, and loading condition, that use the channel and can hit the bridge component

PA = the probability of vessel aberrancy

PG = the geometric probability of a collision between an aberrant vessel and a bridge pier or span

PC = the probability of bridge collapse due to a collision with an aberrant vessel

AF shall be calculated for each bridge element and vessel classification. The summation of all element *AF*s equals the annual frequency of collapse for the entire bridge structure.

A3.3.3.2 Probability of aberrancy

The probability of vessel aberrancy, *PA* (the probability that a vessel will stray off course and threaten a bridge) may be determined either by statistical analysis of historical data on vessels transiting the waterway or by the following approximate method:

$$PA = (BR)(R_B)(R_C)(R_{XC})(R_D)$$

where

BR = aberrancy base rate (usually taken as 0.6×10^{-4} for ships)

R_B = correction factor for bridge location

R_C = correction factor for current acting parallel to vessel transit path

R_{XC} = correction factor for cross-currents acting perpendicular to vessel transit path

R_D = correction factor for vessel traffic density

A3.3.3.3 Correction factors

Δ A3.3.3.3.1 Factor for bridge location

Based on the relative location of the bridge in one of three waterway regions, as shown in Figure A3.3.1, the correction factor, R_B, shall be as follows:

(a) for straight regions: 1.0

(b) for transition regions:

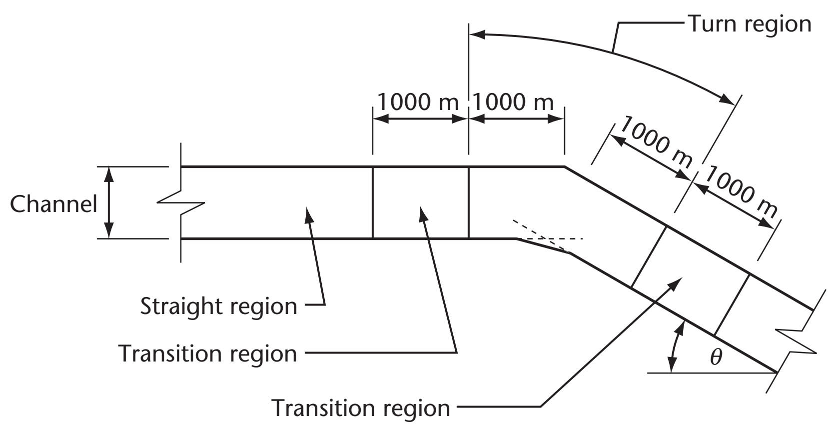
$$1.0 + \frac{\theta}{90°}$$

(c) for turn/bend regions:

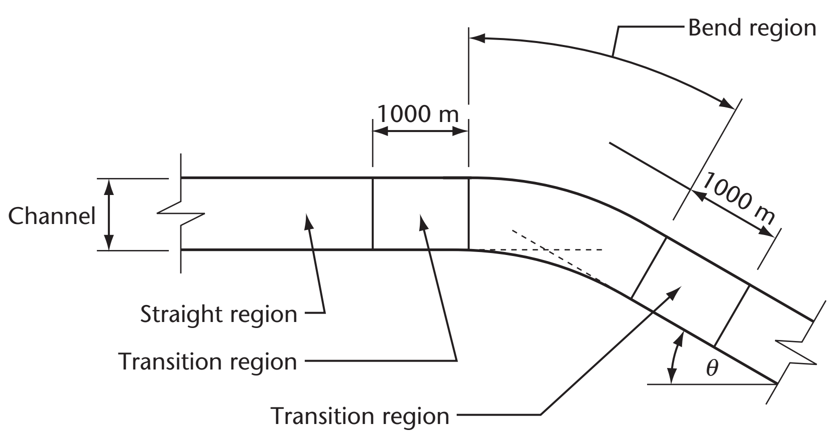
$$1.0 + \frac{\theta}{45°}$$

where

θ = angle of the turn or bend specified in Figure A3.3.1, degrees



(a) Turn in channel



(b) Bend in channel

Figure A3.3.1
Waterway regions for bridge location
(See Clause A3.3.3.1.)

A3.3.3.3.2 Factor for parallel currents

For currents acting parallel to the vessel transit path in the waterway, the correction factor shall be

$$R_C = 1.0 + \frac{V_C}{5}$$

where

V_C = current velocity component parallel to vessel transit path, m/sec

Δ A3.3.3.3.3 Factor for cross-currents

For currents acting perpendicular to the vessel transit path in the waterway, the correction factor shall be

$$R_{XC} = 1.0 + \frac{V_{XC}}{2}$$

where

V_{XC} = current velocity component perpendicular to vessel transit path, m/sec

A3.3.3.3.4 Factor for vessel traffic density

The correction factor, R_D, selected on the basis of the ship traffic density in the waterway in the immediate vicinity of the bridge, shall be

(a) 1.0 for low-density traffic, in which vessels rarely meet, pass, or overtake each other in the immediate vicinity of the bridge;
(b) 1.3 for average-density traffic, in which vessels occasionally meet, pass, or overtake each other in the immediate vicinity of the bridge; or
(c) 1.6 for high-density traffic, in which vessels routinely meet, pass, or overtake each other in the immediate vicinity of the bridge.

A3.3.3.3.5 Geometric probability

The geometric probability, PG, being the conditional probability that an aberrant vessel in the zone will hit a pier, may be determined by a normal distribution to model the aberrant vessel sailing path near the bridge or by the following approximate method:

$$PG = 1 - \frac{L_C - B}{Z_W}$$

where

L_C = width of horizontal clearance from the pier(s), m

B = width of ship, m

Z_W = width of zone, m

For the same zone and pier widths, PG shall be calculated for each ship category. A similar equation based on vertical clearance may be used to calculate PG values for an impact between bridge spans and vessel superstructures.

A3.3.3.3.6 Probability of collapse

The probability of bridge collapse, PC, once a bridge component has been hit by an aberrant vessel, being based on the ratio of the ultimate lateral resistance of the pier, H_P, and span, H_S, to the vessel impact force, P, as shown in Figure A3.3.2, shall be calculated as follows:

$$PC = 0.1 + 9(0.1 - H/P) \quad \text{if } 0.0 \le H/P < 0.1$$

$$PC = \frac{(1 - H/P)}{9} \quad \text{if } 0.1 \leq H/P \leq 1.0$$

$$PC = 0.0 \quad \text{if } H/P > 1.0$$

where

H = ultimate bridge element strength, H_P or H_S, MN

P = vessel impact forces, P_S, P_{BH}, P_{DH}, or P_{MT}, MN, as specified in Clauses A3.3.5 and A3.3.7

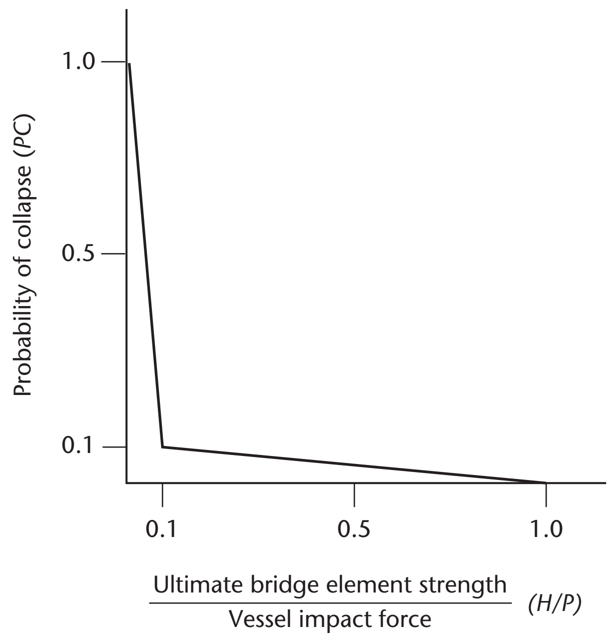


Figure A3.3.2
Probability of collapse distribution
(See Clause A3.3.3.3.6.)

A3.3.4 Design collision velocity

A3.3.4.1 Transit velocity in channel

The vessel transit velocity, V_T, shall represent the velocity at which the design vessel is transiting the channel or waterway under normal environmental conditions.

A3.3.4.2 Minimum collision velocity

The minimum collision velocity, V_{min}, shall not be less than the yearly mean current velocity at the bridge location. In waterways subject to seasonal flooding, flood flow velocity shall be considered in determining the collision velocity.

A3.3.4.3 Distribution

When an aberrant vessel wanders away from the navigation channel, its velocity shall be considered to reduce linearly to the minimum velocity over a distance equal to three times the overall length of the design vessel, as shown in Figure A3.3.3.

Table 4.3
Regular bridge requirements
(See Clause 4.4.5.3.2.)

	Number of spans				
	2	3	4	5	6
Maximum subtended angle (curved bridge)	90°	90°	90°	90°	90°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent or pier stiffness ratio from span to span (excluding abutments)					
Continuous superstructure or multiple simple spans with longitudinal restrainers and transverse restraint at each support or a continuous deck slab	—	4	4	3	2
Multiple simple spans without restrainers or a continuous deck slab	—	1.25	1.25	1.25	1.25

Note: *All ratios are expressed in terms of the smaller value.*

4.4.6 Site effects

4.4.6.1 General

The effects of site conditions on bridge response shall be included in the determination of seismic loads for bridges.

The site coefficient, *S*, specified in Table 4.4, shall be based on the soil profile types specified in Clauses 4.4.6.2 to 4.4.6.5.

Subject to the requirements of Clause 4.4.6.6, a site coefficient need not be explicitly identified if a site-specific seismic response coefficient is developed by a qualified specialist.

Table 4.4
Site coefficient, *S*
(See Clauses 4.4.6.1 and 4.4.6.6.)

Soil profile type	Site coefficient, *S*
I	1.0
II	1.2
III	1.5
IV	2.0

4.4.6.2 Soil Profile Type I

Soil Profile Type I is a profile with
(a) rock of any characteristic, shale-like or crystalline in nature (such material can be characterized by a shear wave velocity greater than 750 m/s or by another appropriate means of classification); or
(b) stiff soil conditions where the soil depth is less than 60 m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

4.4.6.3 Soil Profile Type II
Soil Profile Type II is a profile with stiff clay or deep cohesionless soils where the soil depth exceeds 60 m and the soil types overlying rock are stable deposits of sands, gravels, or stiff clays.

4.4.6.4 Soil Profile Type III
Soil Profile Type III is a profile with soft to medium-stiff clays and sands, characterized by 9 m or more of soft to medium-stiff clays with or without intervening layers of sand or other cohesionless soils.

4.4.6.5 Soil Profile Type IV
Soil Profile Type IV is a profile with soft clays or silts greater than 12 m in depth. These materials can be characterized by a shear wave velocity less than 150 m/s and can include loose natural deposits or non-engineered fill.

4.4.6.6 Other soil profile types
For other soil profile types, or where the soil properties are not known in sufficient detail to determine the soil profile type with confidence, the Engineer shall use his or her judgment to select a site coefficient from Table 4.4 that conservatively represents the amplification effects at the site. The soil profile coefficients shall apply to all foundation types, including pile-supported and spread footings.

4.4.7 Elastic seismic response coefficient

Δ 4.4.7.1 General
Unless otherwise specified in Clause 4.4.7.2, the elastic seismic response coefficient, C_{sm}, for the *m*th mode of vibration shall be

$$C_{sm} = \frac{1.2AIS}{T_m^{2/3}} \leq 2.5AI$$

where

A = zonal acceleration ratio specified in Clause 4.4.3

S = site coefficient specified in Clause 4.4.6

T_m = period of vibration of the *m*th mode, s

I = importance factor based on the importance category specified in Clause 4.4.2
= 3.0 for lifeline bridges, but need not be taken greater than the value of R for the ductile substructure elements specified in Table 4.5
= 1.5 for emergency-route bridges
= 1.0 for other bridges

4.4.7.2 Exceptions
The following exceptions shall apply:

(a) For Soil Profile Type III or Type IV soils in areas where the zonal acceleration ratio is equal to or greater than 0.30, C_{sm} need not exceed $2.0AI$.

(b) For Soil Profile Type III or Type IV soils, C_{sm} for modes other than the fundamental mode that have periods less than 0.3 s shall be taken as

$$C_{sm} = AI(0.8 + 4.0T_m)$$

(c) For structures in which the period of vibration of any mode exceeds 4.0 s, the value of C_{sm} for that mode shall be taken as

$$C_{sm} = \frac{3AIS}{T_m^{4/3}}$$

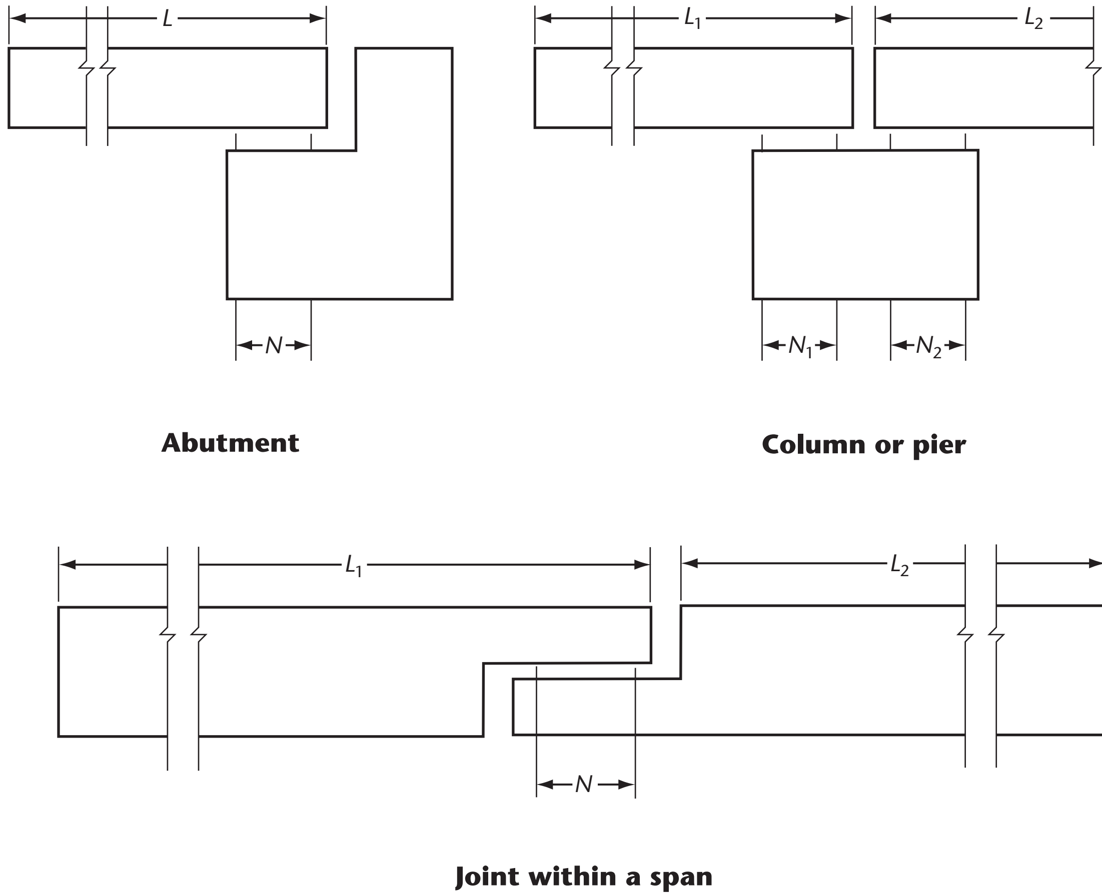


Figure 4.1
Dimensions for minimum support lengths
(See Clause 4.4.10.5.)

Δ

Table 4.6
Modification factor, *K*
(See Clause 4.4.10.5.)

Seismic performance zone	Zonal acceleration ratio	Soil profile type	Modification factor, *K*
1	0	I or II	0.5
1	0	III or IV	1.0
1	0.05	All	1.0
2	All applicable	All	1.0
3	All applicable	All	1.5
4	All applicable	All	1.5

4.4.10.6 Longitudinal restrainers

Restrainers shall be designed to ensure integrity under excessive forces or movements without experiencing brittle failures. Friction shall not be considered to be an effective restrainer.

Restrainers shall be designed for a force calculated as three times the zonal acceleration ratio, A, multiplied by the dead load of the lighter of the two adjoining spans or parts of the structure, but the factor $3A$ shall not be less than 0.2.

If the restrainer is at a point where relative displacement of sections of the superstructure will occur due to effects such as temperature change and shrinkage, sufficient slack shall be provided in the restrainer so that the restrainer does not restrict such movements.

Where restrainers are to be provided at columns or piers, the restrainer of each span may be attached to the column or pier rather than interconnecting adjacent spans. Where a column or pier could be subject to instability due to ground liquefaction or excessive ground movements, the restrainer shall be attached to the column or pier.

The connections of a restrainer to the superstructure or substructure shall be designed to resist 125% of the ultimate restrainer capacity. Restrainers shall be designed to remain elastic under the design seismic forces specified in this Section.

4.4.10.7 Hold-down devices

For bridges in Seismic Performance Zones 2, 3, and 4, hold-down devices shall be provided at supports and at hinges where the vertical force effect resulting from the seismic load cases specified in Clause 4.4.9.2 or 4.4.9.3 opposes and exceeds 50% but is less than 100% of the reaction due to dead loads. In this case, the net upward force for the design of the hold-down device shall be taken as 10% of the reaction due to dead loads that would be exerted if the span were simply supported.

If the vertical seismic forces result in net uplift, the hold-down device shall be designed to resist the larger of

(a) 120% of the difference between the vertical seismic force and the reaction due to dead loads; and
(b) 10% of the reaction due to dead loads that would be exerted if the span were simply supported.

4.5 Analysis

4.5.1 General

The minimum analysis requirements for seismic effects are specified in Clauses 4.4.5.2 and 4.4.5.3. The four types of analysis are described in Clauses 4.5.3.1 to 4.5.3.4.

In the analysis methods specified in Clause 4.5.3, the actual weight shall be taken as the effective weight.

In the modelling of reinforced concrete sections, either uncracked or cracked cross-sectional properties shall be used when the periods and force effects are calculated. The effects of cracking shall be taken into account in calculating deflections.

For the modal methods specified in analysis specified in Clauses 4.5.3.2 and 4.5.3.3, the elastic design spectrum shall be in accordance with Clause 4.4.7.

Bridges in Seismic Performance Zone 1 need not be analyzed for seismic loads, regardless of their importance and geometry. However, the minimum requirements specified in Clauses 4.4.10.2 and 4.4.10.5 shall apply.

4.5.2 Single-span bridges

Seismic analysis of single-span bridges shall not be required, regardless of seismic zone, except as required by Clause 4.4.5.2.1.

4.5.3 Multi-span bridges

4.5.3.1 Uniform-load method

The uniform load method is applicable to both transverse and longitudinal earthquake motions. The equivalent uniformly distributed static seismic loading, P_e, shall be taken as

$$P_e = \frac{C_{sm}W}{L}$$

where

W = effective weight of the bridge

L = total length of the bridge

In determining C_{sm} from Clause 4.4.7.1, the period of vibration of the bridge, T, shall be taken as

$$T = 2\pi\sqrt{\frac{W}{gK}}$$

where

g = acceleration due to gravity, m/s^2

K = lateral stiffness of the bridge

$$= \frac{p_o L}{V_{s,max}}$$

where

$V_{s,max}$ = maximum static displacement of the bridge due to an arbitrary uniform lateral load, p_o

Δ 4.5.3.2 Single-mode spectral method

The single-mode spectral method of analysis shall be based on the fundamental mode of vibration in either the longitudinal or transverse direction, as appropriate. This mode shape may be found by applying a uniform horizontal load to the structure and calculating the corresponding deformed shape. The natural period may be calculated by equating the maximum potential and kinetic energies associated with the fundamental mode shape or by a more rigorous dynamic analysis. The amplitude of the displaced shape may be found from the elastic seismic response coefficient, C_{sm}, specified in Clause 4.4.7.1 and the corresponding spectral displacement. This amplitude shall be used to determine force effects.

The intensity of the equivalent static seismic loading, $p_e(x)$, shall be taken as

$$p_e(x) = \frac{\beta C_{sm}}{\gamma} W(x) V_s(x)$$

where

β = $\int W(x)V_s(x)\,dx$

C_{sm} = elastic seismic response coefficient in Clause 4.4.7.1

V_s = deformation corresponding to p_o

where

p_o = an arbitrary uniform lateral load

γ = $\int W(x)V_s^2(x)\,dx$

$W(x)$ = effective weight of the bridge

In determining C_{sm}, the period of vibration of the bridge, T, shall be taken as

$$T = 2\pi\sqrt{\frac{\gamma}{p_o g \alpha}}$$

where

α = $\int V_s(x)\,dx$

4.5.3.3 Multi-mode spectral method

The multi-mode spectral method of analysis shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. A three-dimensional model shall be used to represent the structure.

The number of modes used in the analysis shall be such that 90% mass participation of the superstructure in the direction under consideration is accounted for. The elastic seismic response spectrum specified in Clause 4.4.7 shall be used for each mode.

The member forces and displacements shall be estimated using an accepted modal combination procedure. For bridges with closely spaced modes (within 10% of each other in terms of natural frequency), the complete quadratic combination (CQC) method or the absolute sum of the modal quantities shall be used.

4.5.3.4 Time-history method

The time-histories of input acceleration used to describe the earthquake loads shall be selected in consultation with the Regulatory Authority. Unless the Regulatory Authority otherwise directs, five spectrum-compatible time histories shall be used when site-specific time-histories are not available. The spectrum used to generate these five time-histories shall be the seismic response spectrum specified in Clause 4.4.7. If site-specific time-histories are used, they shall include the site soil profile effects and be modified by the importance factor, I.

Every step-by-step time-history method of analysis used for elastic or inelastic analysis shall satisfy the requirements of Clause 5.11.

If an inelastic time-history method of analysis is used, the R-factors shall be taken as 1.0.

The sensitivity of the numerical solution to the size of the time step used for the analysis shall be determined. A sensitivity study shall also be carried out to investigate the effects of variations in assumed material hysteretic properties.

4.5.3.5 Static pushover analysis

The static pushover analysis shall be a step-by-step force deformation response analysis that takes account of inelastic response and the structural detailing specified in the design, e.g., anchorage of reinforcement for reinforced concrete members and connection details for steel members. Possible local and global instability and brittle failure modes shall be considered. The analysis results may be used to determine the deformation capacity of the structure.

4.6 Foundations

4.6.1 General

In addition to satisfying the requirements of Section 6, the requirements specified in Clauses 4.6.2 to 4.6.6 shall also be satisfied in Seismic Performance Zones 2, 3, and 4.

4.6.2 Liquefaction of foundation soils

An evaluation shall be made of the potential for liquefaction of foundation soils and the impact of liquefaction on bridge foundations.

4.7.4.2.5 Transverse reinforcement for confinement at plastic hinge regions

The cores of columns and concrete pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The transverse reinforcement for confinement shall have a yield strength not more than that of the longitudinal reinforcement and the spacing shall be in accordance with Clause 4.7.4.2.6.

For a circular column, the ratio of spiral reinforcement, ρ_s, shall not be less than the greater of that determined in accordance with Clause 8.14.4.2 or

$$\rho_s = 0.12\frac{f'_c}{f_y}\left[0.5 + \frac{1.25P_f}{\phi_c f'_c A_g}\right]$$

where

$$\left[0.5 + \frac{1.25P_f}{\phi_c f'_c A_g}\right] \geq 1.0$$

Within plastic hinge regions, splices in spiral reinforcement shall be in accordance with Clause 8.14.4.2. Lap splices in longitudinal reinforcement shall be used only as specified in Clause 4.7.4.2.7.

For rectangular columns, the total cross-sectional area, A_{sh}, of transverse reinforcement shall not be less than the greater of

$$A_{sh} = 0.30 s h_c \frac{f'_c}{f_y}\left[\frac{A_g}{A_c} - 1\right]$$

$$A_{sh} = 0.12 s h_c \frac{f'_c}{f_y}\left[0.5 + \frac{1.25P_f}{\phi_c f'_c A_g}\right]$$

where

$$\left[0.5 + \frac{1.25P_f}{\phi_c f'_c A_g}\right] \geq 1.0$$

and s is the vertical spacing of transverse reinforcement.

A_{sh} shall be calculated for both principal axes of a rectangular column and the larger value shall be used.

Transverse reinforcement in plastic hinge regions shall be provided by single or overlapping hoops or spirals. Seismic cross-ties having the same bar size as the tie may be used. Each end of the seismic cross-tie shall engage a peripheral longitudinal reinforcing bar. Seismic cross-ties shall be alternated so that hooks that do not qualify as seismic hooks are not adjacent to each other in the horizontal and vertical directions.

4.7.4.2.6 Spacing of transverse reinforcement for confinement

Transverse reinforcement for confinement shall be provided in the plastic hinge regions specified in Clause 4.7.4.2.4 and shall extend into the top and bottom connections in accordance with Clause 4.7.4.4.

The centre-to-centre spacing shall not exceed the smallest of 0.25 times the minimum component dimension, six times the diameter of the longitudinal reinforcement, or 150 mm.

The centre-to-centre spacing of interlocking spirals or hoop cages in oblong columns shall not be greater than 0.75 times the diameter of the spiral or hoop cage. A minimum of four vertical bars shall be located within each overlapping region of the spirals or hoops.

4.7.4.2.7 Splices

Splices shall satisfy the requirements of this Clause and Clause 8.15.9.

Lap splices in longitudinal reinforcement shall not be located in plastic hinge regions and shall be permitted only within the centre half of column height unless the splices are located in a region where it is demonstrated that plastic hinging will not occur. The splice length shall not be less than the greater of 60 bar diameters or 400 mm. The centre-to-centre spacing of the transverse reinforcement over the length of the splice shall not exceed the smaller of 0.25 times the minimum cross-section dimensions of the component or 100 mm.

Welded splices in accordance with Clause 8.15.9.2 or mechanical connection splices in accordance with Clause 8.4.4.4 may be used if not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section and the distance between splices of adjacent bars is greater than the larger of 600 mm or $40d_b$ measured along the longitudinal axis of the column.

Δ 4.7.4.3 Wall-type piers

The requirements of this Clause shall apply to the design for the strong direction of a wall-type pier. The weak direction may be designed as a column in accordance with Clause 4.7.4.2. If the wall-type pier is not designed as a column in its weak direction, the limitations for shear resistance specified in this Clause shall apply.

The reinforcement ratio, both horizontally, ρ_h, and vertically, ρ_v, in any wall-type pier shall not be less than 0.0025, and ρ_v shall not be less than ρ_h.

Reinforcement spacing, both horizontally and vertically, shall not exceed 450 mm. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The shear resistance, V_r, of the pier shall be taken as the lesser of $2.25\phi_c f_{cr} bd$ and $(0.41\phi_c f_{cr} + \rho_h \phi_s f_y)bd$.

Horizontal and vertical reinforcement shall be provided at each face of a pier. Splices in horizontal reinforcement shall be staggered and splices in horizontal and vertical layers shall not occur at the same location.

Ties for end wall reinforcement need not extend across the strong direction.

4.7.4.4 Column connections

The design forces for column connections shall be those for capacity-protected elements in accordance with Clause 4.4.10.4.2. The development length for all longitudinal steel shall be 1.25 times that specified in Clause 8.15.2.

Column transverse reinforcement, as specified in Clause 4.7.4.2.5, shall be continued for a distance not less than the greater of 0.5 times the maximum column dimension or 400 mm from the face of the column connection into the adjoining component.

The shear resistance provided by the concrete in the joint of a frame or bent, in the direction under consideration, shall not exceed $2.5\phi_c f_{cr} bd$.

4.7.5 Piles

4.7.5.1 General

Pile reinforcing details shall meet the requirements of Clauses 4.7.5.2 to 4.7.5.4 and 8.23.

4.7.5.2 Seismic Performance Zone 1

No additional design provisions need to be considered for Seismic Performance Zone 1.

4.7.5.3 Seismic Performance Zone 2

4.7.5.3.1 General

Piles for structures in Seismic Performance Zone 2 shall meet the requirements of Clause 4.4.10.3.

Concrete piles shall be anchored to the pile footing or cap by embedment of pile reinforcement or by anchorages to develop uplift forces. The embedment length shall not be less than 1.25 times the development length required for the reinforcement specified in Clause 8.15.

Concrete-filled pipe piles shall be anchored by at least four dowels with a minimum steel ratio of 0.01. Dowels shall be embedded in the manner normally used for concrete piles.

Δ Section 5 — Methods of analysis

Annexes

E_s = modulus of elasticity of steel

e = for orthotropic steel decks, the distance centre-to-centre between adjacent closed ribs

F = when a bridge superstructure is analyzed in accordance with simplified methods, a width dimension that characterizes load distribution for a bridge, as specified in Clauses 5.7.1.2 to 5.7.1.5 and 5.7.1.6.2, m

F_m = when a bridge superstructure is analyzed in accordance with simplified methods, an amplification factor to account for the transverse variation in maximum longitudinal moment intensity, as compared to the average longitudinal moment intensity

F_v = when a bridge superstructure is analyzed in accordance with simplified methods, an amplification factor to account for the transverse variation in maximum longitudinal vertical shear intensity, as compared to the average longitudinal vertical shear intensity

G = shear modulus

G_{LT} = shear modulus of wood with respect to axes L and T shown in Figure A5.2.2

G_c = shear modulus of concrete

G_s = shear modulus of steel

I = moment of inertia of the cross-section of a beam

i_L = longitudinal moment of inertia per unit width

i_T = transverse moment of inertia per unit length

J = torsional inertia of a beam

j_L = longitudinal torsional inertia per unit width

j_T = transverse torsional inertia per unit length

K = torsional constant for a rectangular section

k = constant used in calculating maximum transverse vertical shear intensity due to live load in shear-connected beam bridges (see Clause 5.7.1.8.1)

L = for simply supported spans, the span; for continuous spans, the span specified in Clause A5.1.2

L_s = stringer span

ℓ_1, ℓ_2 = distances between points of inflection, as shown in Figure 5.6 for orthotropic steel decks

M_T = maximum longitudinal moment for one lane width of truck or lane loading, as applicable, including dynamic load allowance

Δ M_e = moment at the end of an individual compression member

M_g = for girder-type bridges, the maximum longitudinal moment per girder due to live load, including the effects of amplification for transverse variation in maximum longitudinal moment intensity and dynamic loading

$M_{g,avg}$ = average moment per girder due to live load, determined by sharing equally the total live load moment on the bridge cross-section among all girders in the cross-section

Δ M_{ns} = moment at the end of a compression member due to loads that cause no appreciable sway, calculated using first-order elastic analysis

Δ M_s = moment at the end of a compression member due to loads that cause appreciable sway, calculated using first-order elastic analysis

M_x = longitudinal bending moment per unit width

M_{xy} = longitudinal torsional moment per unit width

M_y = transverse bending moment per unit length

m = for bridges of the solid cross-section type, e.g., slabs, voided slabs, and wood deck bridges that span longitudinally, the maximum longitudinal moment per metre of width due to live load

m_{avg} = for bridges of the solid cross-section type, e.g., slabs, voided slabs, and wood deck bridges that span longitudinally, the average longitudinal moment per metre of width due to live load

Δ N = number of girders or longitudinal wood beams in the bridge deck width

n = modular ratio, E_s/E_c, for steel and concrete; modular ratio of girder or beam material to slab material; number of design lanes on a bridge

Δ P = wheel load of the CL-625 Truck

R = mean radius of curvature of the curved portion of a bridge that is curved in plan

R_L = modification factor for multi-lane loading in accordance with Clause 3.8.4.2

r_t = ratio of the thickness, t_1, of the deck slab at the exterior edge of a bridge deck to the thickness, t_2, at the edge of the flange of the external girder

S = centre-to-centre spacing of longitudinal girders of a deck-on-girder bridge; centre-to-centre spacing of circular voids of a voided slab bridge; centre-to-centre spacing of spines of a multi-spine bridge

Δ S_c = transverse distance from the free edge of the cantilever overhang to the centreline of the web of the external girder

Δ S_e = equivalent span of concrete deck in metres (see Clause 5.7.1.7.1)

Δ S_p = transverse distance of the free edge of the cantilever overhang to the supported edge (see Clause 5.7.1.6.1.1)

S_y = transverse shear rigidity per unit length

s_v = shear area per unit length, as specified in Clause A5.2.1

t = overall thickness of a slab

t_v = depth of circular or rectangular void in voided slabs

t_1 = slab thickness at the external edge of the deck slab

t_2 = slab thickness at the edge of the flange of the external girder

V_T = maximum longitudinal vertical shear for one lane width of truck or lane loading, as applicable, including dynamic load allowance

V_g = for girder-type bridges, the maximum longitudinal vertical shear per girder due to live load, including the effects of amplification for transverse variation in maximum longitudinal vertical shear intensity and dynamic loading

$V_{g,avg}$ = average shear per girder due to live load, determined by sharing equally the total live load shear on the bridge cross-section among all girders in the cross-section

V_y = maximum intensity of transverse vertical shear in shear-connected beam bridges

v = for slab, voided slab, and wood deck bridges that span longitudinally, the maximum longitudinal vertical shear per metre of width due to live load, including the effects of amplification for transverse variation in maximum longitudinal vertical shear intensity and dynamic loading

v_{avg} = average shear per metre of width due to live load for slab, voided slab, and wood deck bridges that span longitudinally, determined by sharing uniformly the total live load shear on the bridge cross-section over the width of the bridge cross-section

W_e = width of a design lane, m

w = transverse deflection of plates as used in the plate theory

x = transverse distance between the face of the railing and the longitudinal line where moment intensity is investigated in a cantilever slab, m

x, y = coordinates of a reference point on a cantilever slab, as shown in Figure 5.2

β = parameter specified in Clause 5.7.1.3

Δ δ_s = moment magnification factor accounting for second-order effects of vertical load acting on a structure in a laterally displaced configuration

ε = skew parameter specified in Clause 5.6.1.1

μ = lane width modification factor

ν = Poisson's ratio

ψ = skew angle; for orthotropic steel decks, the effective plate width factor for interior portions of the deck, as shown in Figure 5.6

ψ_p = for orthotropic steel decks, the effective plate width factor for exterior portions of the deck, as shown in Figure 5.6

Table 5.4
F and C_f for longitudinal bending moments in shallow superstructures corresponding to the fatigue limit state
(See Clause 5.7.1.2.2.2.)

Bridge type (see Clause 5.1)	No. of design lanes	Portion	*F*, m		C_f, %
			$3\text{ m} < L \le 10\text{ m}$	$L > 10\text{ m}$	
Type A or B	1	External	$3.80 + 0.04L$	4.20	$16 - (36/L)$
		Internal	$4.00 + 0.04L$	4.40	$16 - (36/L)$
	2	External	$3.60 + 0.26L$	$7.00 - (8/L)$	$16 - (36/L)$
		Internal	$3.20 + 0.30L$	$6.40 - (2/L)$	$16 - (36/L)$
	3	External	$3.30 + 0.30L$	$9.60 - (33/L)$	$16 - (36/L)$
		Internal	$3.00 + 0.40L$	$9.80 - (29/L)$	$12 - (36/L)$
	4 or more	External	$3.40 + 0.30L$	$12.00 - (56/L)$	$10 - (30/L)$
		Internal	$3.00 + 0.44L$	$12.00 - (46/L)$	$10 - (30/L)$
Δ Type C slab-on-girder	1	External	3.30	$3.50 - (2/L)$	$5 - (12/L)$
		Internal	$3.30 + 0.05L$	$4.40 - (6/L)$	$5 - (12/L)$
	2	External	3.60	$3.80 - (2/L)$	$5 - (15/L)$
		Internal	$2.80 + 0.12L$	$4.60 - (6/L)$	$5 - (15/L)$
	3	External	$3.60 + 0.01L$	$3.70 + \dfrac{(L-10)}{140}$	0
		Internal	$2.80 + 0.12L$	$4.80 - (8/L)$	0
	4 or more	External	3.80	$3.80 + \dfrac{(L-10)}{140}$	0
		Internal	$2.80 + 0.12L$	$5.00 - (10/L)$	0
Type C with 140 mm laminated wood deck*	1	External	3.40	3.40	0
		Internal	$2.80 + 0.06L$	$4.20 - (8/L)$	0
	2	External	3.60	$3.80 - (2/L)$	0
		Internal	$3.00 + 0.06L$	$4.40 - (8/L)$	0
	3	External	3.60	$3.80 - (2/L)$	0
		Internal	$3.00 + 0.06L$	$4.40 - (8/L)$	0
	4 or more	External	3.60	$3.80 - (2/L)$	0
		Internal	$3.00 + 0.06L$	$4.40 - (8/L)$	0
Type C with 290 mm laminated wood deck*	1	External	3.50	3.50	0
		Internal	$3.30 + 0.05L$	$4.80 - 20/(L + 10)$	$5 - (12/L)$
	2	External	$3.50 + 0.02L$	$3.90 - (2/L)$	$6 - (15/L)$
		Internal	$3.30 + 0.05L$	$5.40 - 3.20\sqrt{L-6}$	$6 - (15/L)$
	3	External	$3.50 + 0.02L$	$3.90 - (2/L)$	$10 - (21/L)$
		Internal	$3.40 + 0.06L$	$5.50 - 1350/(L + 20)^2$	$10 - (21/L)$
	4 or more	External	$3.50 + 0.02L$	$3.90 - (2/L)$	$10 - (21/L)$
		Internal	$3.60 + (L^2/200)$	$6.00 - (19/L)$	$10 - (21/L)$

(Continued)

Table 5.4 (Continued)

Bridge type (see Clause 5.1)	No. of design lanes	Portion	F, m		C_f, %
			3 m < L ≤ 10 m	L > 10 m	
Type C steel-grid-deck-on-girder with deck less than 100 mm thick	1	External or Internal	2.70	2.70	0
	2	External or Internal	2.80	2.80	0
	3	External or Internal	2.80	2.80	0
	4 or more	External or Internal	2.80	2.80	0
Type C steel-grid-deck-on-girder with deck 100 mm thick or thicker	1	External or Internal	3.60	3.60	0
	2	External or Internal	3.70	3.70	0
	3	External or Internal	3.70	3.70	0
	4 or more	External or Internal	3.70	3.70	0
Type C with wood plank deck	1	External or Internal	2.40	2.40	0
	2	External or Internal	2.40	2.40	0
	3	External or Internal	2.40	2.40	0
	4 or more	External or Internal	2.40	2.40	0
Type H sawn wood stringer bridge	1	External or Internal	3.60	—	0
	2	External or Internal	3.70	—	0
	3 or more	External	3.70	—	0
		Internal	3.80	—	0
Type H stress-laminated wood deck bridge spanning longitudinally	1	External or Internal	$3.10 + 0.08L$	$3.10 + 0.08L$	0
	2 or more	External or Internal	$3.40 + 0.07L$	$3.40 + 0.07L$	0
Type H longitudinal nail-laminated wood deck bridge	1 or more	External or Internal	1.70	—	0

(Continued)

Table 5.4 (Concluded)

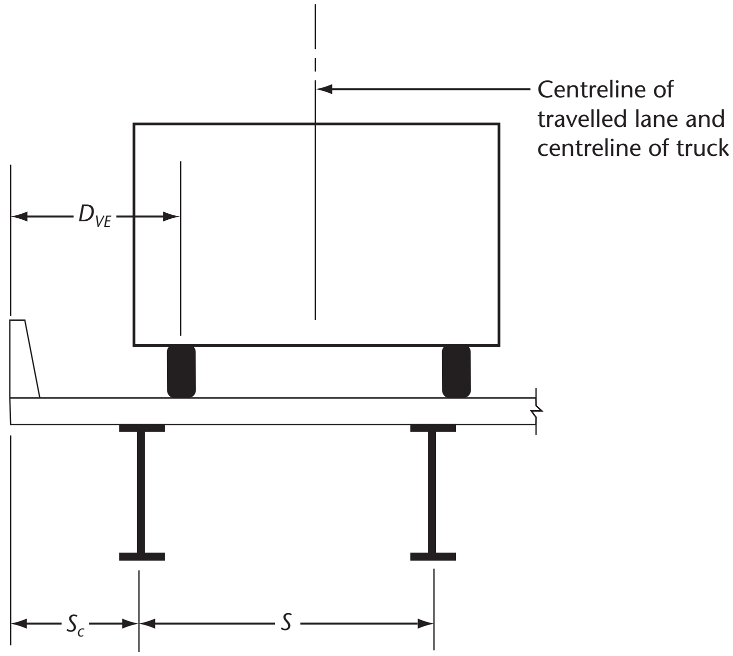
Bridge type (see Clause 5.1)	No. of design lanes	Portion	*F*, m		C_f, %
			3 m < *L* ≤ 10 m	*L* > 10 m	
Type H longitudinal laminates of wood-concrete composite deck bridge	1	External or Internal	3.20	3.20	0
	2	External or Internal	3.30	3.30	0
	3	External or Internal	3.30	3.30	0
	4 or more	External or Internal	3.30	3.30	0

**For girder bridges with laminated wood decks not* 140 *or* 290 mm *thick, linear interpolation based on the values for* 140 *and* 290 mm *decks shall be used.*

Table 5.5
C_e for longitudinal bending moments in shallow superstructures corresponding to the fatigue and vibration limit state

(See Clause 5.7.1.2.2.2.)

Bridge type (see Clause 5.1)	No. of design lanes	Portion	C_e, %	
			3 m < *L* ≤ 20 m	*L* > 20 m
Type C slab-on-girder	1	External	0	0
		Internal	0	0
	2	External	$30(D_{VE}-1)[1+0.4(D_{VE}-1)^2]$	$30(D_{VE}-1)\left[1+\frac{160(D_{VE}-1)^2}{L^2}\right]$
		Internal	0	0
	3	External	$26(D_{VE}-1)[1+0.4(D_{VE}-1)^2]$	$26(D_{VE}-1)\left[1+\frac{160(D_{VE}-1)^2}{L^2}\right]$
		Internal	0	0
	4 or more	External	$26(D_{VE}-1)[1+0.4(D_{VE}-1)^2]$	$26(D_{VE}-1)\left[1+\frac{160(D_{VE}-1)^2}{L^2}\right]$
		Internal	0	0
Other types	1 or more	External or Internal	0	0



Δ **Figure 5.1**
D_{VE} for slab-on-girder bridges
(See Clause 5.7.1.2.2.2.)

5.7.1.3 Longitudinal bending moments in multi-spine bridges

5.7.1.3.1

If all of the applicable conditions listed in Clause 5.7.1.1 are satisfied, the simplified method specified in Clause 5.7.1.3.2 may be used for multi-spine bridges.

5.7.1.3.2

The value of β shall be calculated as follows:

$$\beta = \pi\left[\frac{B}{L}\right]\left[\frac{D_x}{D_{xy}}\right]^{0.5}$$

where

B = for ultimate and serviceability limits states, the width of the bridge
= for the fatigue limit state, the width of the bridge, but not greater than three times the spine spacing, S

D_x = total bending stiffness, EI, of the bridge cross-section divided by the width of the bridge

D_{xy} = total torsional stiffness, GJ, of the bridge cross-section divided by the width of the bridge

The longitudinal bending moment per spine shall be calculated using the methods specified in Clause 5.7.1.2.1 for the ultimate and serviceability limit states and in Clause 5.7.1.2.2 for the fatigue limit state, except that S shall be taken as the centreline-to-centreline spacing of the spines and the applicable values of F and C_f shall be obtained from Table 5.6. No distinction shall be made between internal and external portions of the cross-section, and the value of C_e shall be taken as zero. At ultimate and serviceability limit states for bridges with more than four design lanes, the value of F shall be calculated as follows:

5.7.1.5.2

For bridges with up to four design lanes, the value of F for longitudinal vertical shear shall be obtained from Table 5.9. The method specified in Clause 5.7.1.4 for the applicable limit state shall be used, with S being the centreline-to-centreline spacing of spines and N the number of spines; the shear thus obtained shall be for one spine. The shear force determined for one spine shall be equally distributed to the two webs of the spine. The value of F_v for the purposes of this Clause shall be such that F_v is greater than or equal to F_m.

At ultimate and serviceability limit states for bridges with more than four design lanes, the value of F shall be calculated as follows:

$$F = F_4 \frac{nR_L}{2.80}$$

where

F_4 = value of F for four design lanes obtained from Table 5.9

Table 5.9
***F* for longitudinal vertical shear in multi-spine bridges**
(See Clause 5.7.1.5.2.)

Limit state	Number of design lanes	F, m
ULS or SLS	2	7.2
	3	9.6
	4	11.2
FLS	2 or more	4.25

5.7.1.6 Deck slab moments due to loads on the cantilever overhang

5.7.1.6.1 Transverse moments due to wheel loads on the cantilever overhang

∆ 5.7.1.6.1.1 Transverse moments in the cantilever overhang

For a cantilever slab of constant or linearly varying thickness, the intensity of transverse moment M_y due to a concentrated load P shall be calculated as follows:

$$M_y = \frac{2PA}{\pi\left[1+\left(\dfrac{Ax}{C-y}\right)^2\right]^2}$$

where

A = coefficient obtained from Figure 5.2

C = transverse distance of the load from the supported edge of the cantilever slab

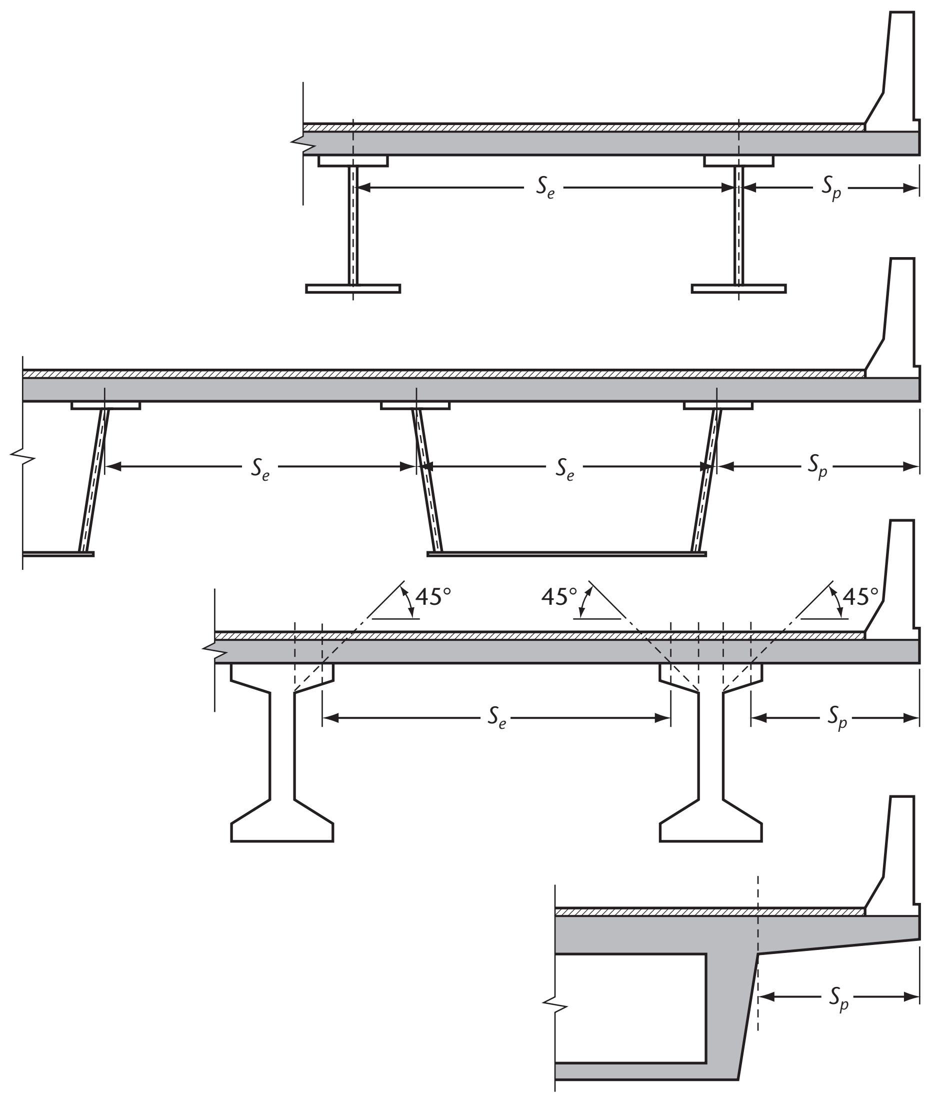
and x and y are the coordinates shown in Figure 5.2, with y less than C.

The relevant design moment intensity shall be obtained by multiplying M_y by (1 + DLA).

For the design moment intensity due to the vertical axle loads of the CL-625 Truck, the effects of individual loads shall be obtained and superimposed or, alternatively, the design moment intensity due to the CL-625 Truck may be obtained directly, without calculation, from Table 5.10 for stiffened and unstiffened overhangs, as applicable (Table 5.10 includes the factor [1 + DLA]). Edge stiffening is provided by a continuous concrete barrier. Barriers shall be continuously longitudinally reinforced and have a stiffness equal to or greater than provided by New Jersey barriers.

For those portions of the cantilever slab that are within a distance S_p of a transverse free edge of the slab, the transverse moment intensity shall be assumed to be $2M_y$ unless a more rigorous analysis is used.

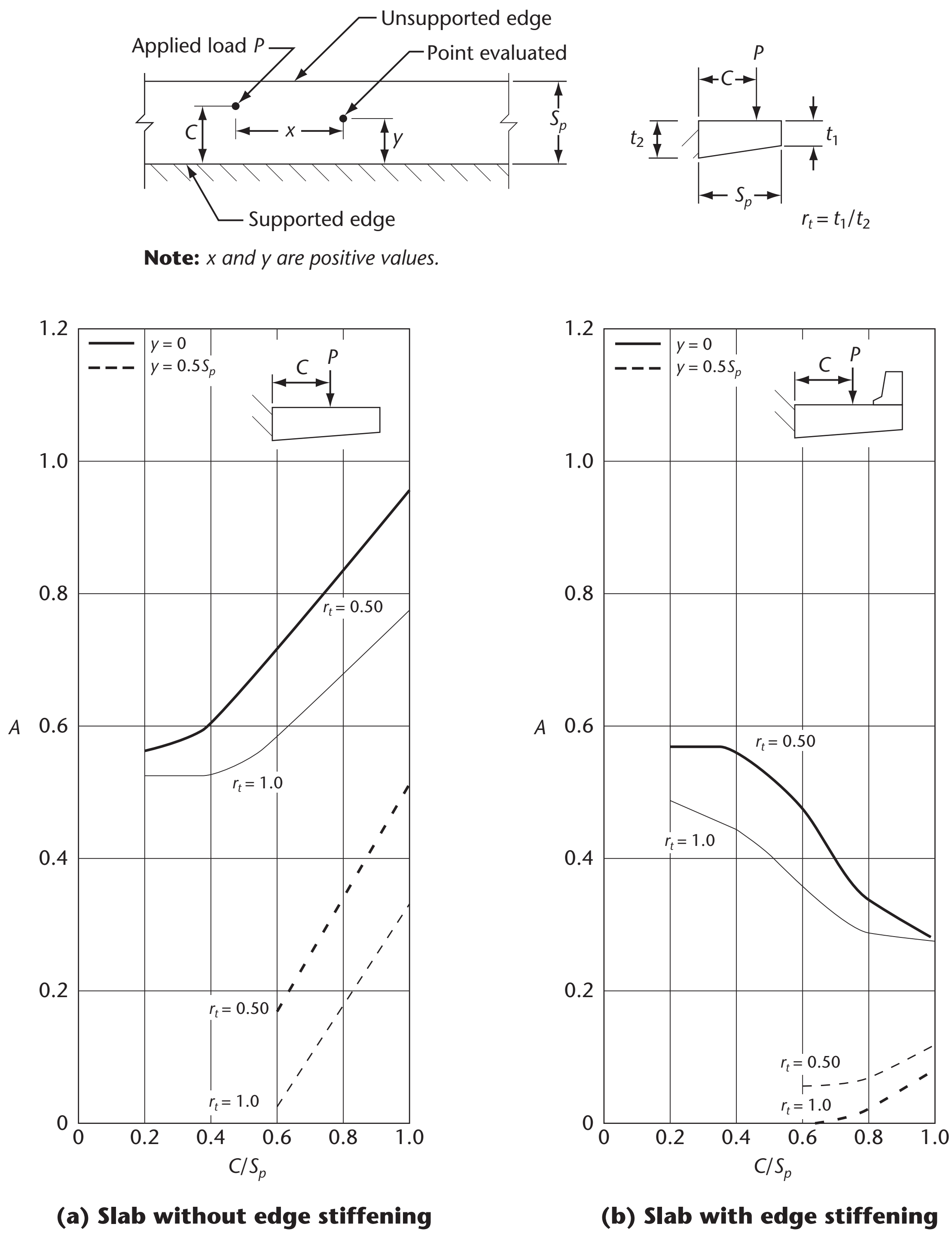
S_p is the transverse distance measured from the free edge of the cantilever overhang to the supported edge. The supported edge may be determined from Figure 5.1A for different types of superstructures.



Δ **Figure 5.1A**
Definition of S_p and S_e
(See Clauses 5.7.1.6.1.1 and 5.7.1.7.1.)

5.7.1.6.1.2 Transverse moments in the interior panel next to the cantilever overhang

In the absence of a more refined method of analysis, the transverse moments in the interior panel next to the cantilever overhang may be assumed to vary linearly from the values calculated in accordance with Clause 5.7.1.6.1.1 at the root of the cantilever overhang to zero at the girder next to the exterior girder.



Note: *For $r_t < 0.5$, use refined analysis.*

Δ

Figure 5.2
Calculation of *A*
(See Clause 5.7.1.6.1.1.)

Δ

Table 5.10
Maximum cantilever moments, M_y, due to unfactored CL-625 Truck wheel loads (*DLA* included), kN•m/m
(See Clause 5.7.1.6.1.1.)

	Unstiffened edge			Stiffened edge		
	Max. M_y in kN•m/m			Max. M_y in kN•m/m		
S_p, m	$r_t = 1.00$	$r_t = 0.75$	$r_t = 0.5$	$r_t = 1.00$	$r_t = 0.75$	$r_t = 0.5$
1.00	41	43	44	37	41	45
1.50	43	47	51	34	37	41
2.00	53	57	60	35	39	43
2.50	60	65	70	37	40	43
3.00	92	99	107	70	74	77

Notes:
(1) *Values obtained for $y = 0$, $C = S_p - 0.75$.*
(2) $r_t = t_1/t_2$.

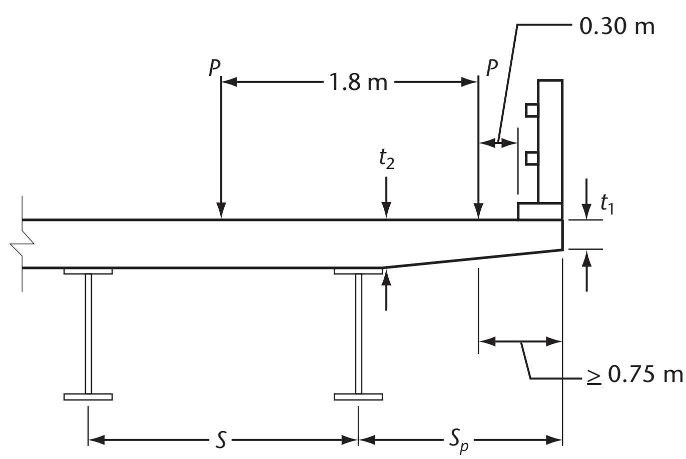


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Figure 5.3
Notation for cantilever moments
(See Table 5.10.)

5.7.1.6.2 Local longitudinal moment in cantilever slabs (main reinforcement parallel to traffic)
For longitudinal cantilever spans not longer than 3 m, the maximum intensity of local longitudinal moment, M_x, in kN•m/m, shall be calculated as follows:

$$M_x = PC/F$$

where

P = 87.5 kN wheel load of the CL-W Truck

C = longitudinal distance of P from the line of transverse support, m

F = $0.35C + 1.00$ (but shall not exceed 2.10 m)

The relevant design moment shall be obtained by multiplying M_x by (1 + *DLA*).

For longitudinal cantilever spans longer than 3 m, the methods specified in Clause 5.7.1.2.1 shall be used, with the span length, L, being taken as twice the cantilever span. The live loading to be used shall be in accordance with Clause 3.8.4.3(c).

5.7.1.6.3 Transverse moments in cantilever slabs due to railing loads

In determining transverse moments in cantilever slabs resulting from the barrier or railing loads specified in Clause 3.8.8.1 and applied in accordance with Clause 12.4.3.5, the method of analysis shall be
(a) a refined method in accordance with Clause 5.9; or
(b) yield line theory.

5.7.1.7 Transverse bending moments in decks

Δ 5.7.1.7.1 Concrete decks slabs supported on longitudinal girders

Concrete deck slabs shall be analyzed for positive and negative bending moments resulting from loads applied on the slabs. The analysis shall consider the bending moments induced in the longitudinal direction that agree with the assumptions used in the analysis of the transverse bending moments. The cantilever portions of concrete deck slabs shall be analyzed for transverse negative bending moments resulting from loads on the cantilever portions of the slabs or horizontal loads on barriers and railings. The cantilever portions of concrete deck slabs may be analyzed using Clause 5.7.1.6.

Concrete deck slabs (cantilever portions that are proportioned in accordance with the empirical design method of Clause 8.18.4 for the CL-625 Truck) need not be analyzed for transverse bending moments due to live load.

Concrete deck slabs that are supported on longitudinal girders may be analyzed for transverse bending using the simplified elastic method in which the maximum unfactored transverse moment intensity in the portion of the deck slab between the outer girders due to the CL-625 Truck shall be determined as follows:

(a) Except for portions of the deck slab within 1 m of a free edge, the deck slab shall be designed for an unfactored transverse live load moment intensity as follows:
   (i) for simple span deck slabs: $(S_e + 0.6)P/10$ kN•m/m, where S_e is the equivalent transverse span in metres, which can be determined from Figure 5.1A for different types of superstructures. P is 87.5 kN, the maximum wheel load of the CL-625 Truck; and
   (ii) for deck slabs continuous over three or more supports, the maximum bending moment, either positive or negative, shall be assumed to be 80% of that determined for a simple span.

   These moments shall be increased by the dynamic load allowance for a single axle, as specified in Clause 3.8.4.5.3.
(b) The portion of a deck slab within 1 m of a transverse free edge shall be reinforced to twice the level of the transverse reinforcement in the other portions of the deck slab unless equivalent local stiffening by diaphragms is provided in accordance with a requirement in another Section of this Code.
(c) The longitudinal moment intensity for distribution of wheel loads to be used with the transverse moment intensity specified in Item (a) shall be taken as $120/(S_e^{0.5})$% (but not to exceed 67% of the maximum transverse moment intensity) and shall be applied as a positive moment that produces tension in the bottom portion of the deck slab. The longitudinal reinforcement necessary to resist the longitudinal moment shall be used in the centre half of the span. The percentage may be reduced by 50% in the end quarters of the span.

5.7.1.7.2 Steel grid decks

5.7.1.7.2.1 General

Transverse bending moments due to live load in steel grid decks shall be determined as specified in this Clause and Clauses 5.7.1.7.2.2 and 5.7.1.7.2.3.

The grid floor shall be designed as continuous. For concrete-filled floors, moments may be determined in accordance with Clause 5.7.1.7.1 using the simplified elastic method for concrete decks.

The requirements for load distribution specified in Clauses 5.7.1.7.2.2 and 5.7.1.7.2.3 assume that the floor is composed of main elements that span between girders, stringers, or cross-beams, and of secondary elements that are capable of transferring load between the main elements. Reinforcement for secondary elements shall consist of bars or shapes welded to the main steel.

5.7.1.7.2.2 Decks filled with concrete

Floors filled with concrete that span perpendicular to the direction of traffic may be analyzed using the elastic method for concrete deck design for load distribution and moment calculation.

Floors that span longitudinally shall be designed for longitudinal moments determined by distributing one line of truck wheel loads over a width $E = 1.22 + 0.06S \leq 2.1$ m, where S is the span in metres.

Longitudinal edge beams shall be provided for all cantilevered slabs having main reinforcement parallel to traffic. The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb or slab and parapet.

The unfactored live load moment for the longitudinal edge beam shall equal $0.1PS$ kN•m for simple spans and $0.08PS$ kN•m for continuous spans, where S is the span, in metres, between points of support. Transverse cantilevered beams, diaphragms, or substructure locations are considered points of support. P is the maximum wheel load of the CL-625 Truck (87.5 kN), which shall be increased by $(1 + DLA)$.

The strength of the composite steel and concrete slab shall be determined using the "transformed area" method.

5.7.1.7.2.3 Open decks

A wheel load of one-tenth of the total weight of the CL-625 Truck (62.5 kN), further increased by $(1 + DLA)$, shall be distributed over a length and width equal to the wheel dimensions specified in Clause 3.8.3.2.

The strength of the section shall be determined using the moment of inertia method.

Edges of open grid steel decks shall be supported by suitable means as required. These supports may be longitudinal, transverse, or both, as required to support all edges properly.

5.7.1.7.3 Transverse laminated wood decking on sawn timber stringers

For bridges with sawn timber stringers, the maximum transverse moment intensity, M_y, due to the CL-625 Truck or to Level 2 or 3 evaluation trucks shall be calculated as follows:

$M_y = 2.40 + 0.47L_s$ for bridges with one design lane

$= 2.19 + 0.56L_s$ for bridges with more than one design lane

In these equations, L_s, the stringer span, is in metres and M_y is in kN•m/m.

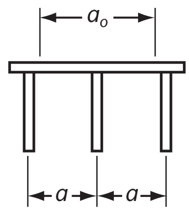
5.7.1.7.4 Transverse stress-laminated wood deck-on-girders

For bridges with stress-laminated wood decks, the transverse moment in the decking shall be calculated by assuming that a transverse line of wheels is sustained uniformly by a transverse strip of the decking of a width, D_t, measured in the longitudinal direction of the bridge span, with $D_t = 0.30 + 0.4S$ for decks with edge stiffening at the transverse free edges, and in which the flexural rigidity of the stiffening beam is greater than or equal to that of a transverse strip of the decking with a width, measured in the longitudinal direction of the bridge span, of 0.25 m. In this equation D_t and S are in metres and S is the girder spacing. If the stiffening beam is absent or has a flexural rigidity less than specified in this Clause, $D_t = 0.30 + 0.14S$ shall be used.

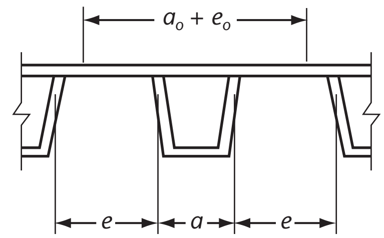
5.7.1.7.5 Transverse nail-laminated wood deck-on-girders

Transverse bending moments due to live load on transverse nail-laminated wood deck-on-girders shall be determined by distributing a wheel load over a width of 0.4 m plus the thickness of the wearing surface.

Table 5.11
Effective deck plate width for a longitudinal rib
(See Clause 5.8.2.2.1.)



Effective width = a_o



Effective width = $a_o + e_o$

Rib section properties for calculation of deck rigidity and flexural effects due to dead load	$a_o = a$	$a_o + e_o = a + e$
Rib section properties for calculation of flexural effects due to wheel loads	$a_o = 1.1a$	$a_o + e_o = 1.3(a + e)$

5.8.2.2.2 Longitudinal girders and transverse beams
The effective width of the deck acting as the top flange of a longitudinal superstructure component or transverse beam may be determined using an accepted method of analysis or may be taken as shown in Figure 5.6.

The effective span, L, shown as ℓ_1 and ℓ_2 in Figure 5.6 shall be taken as the actual span for simple spans and as the distance between points of dead load contraflexure for continuous spans.

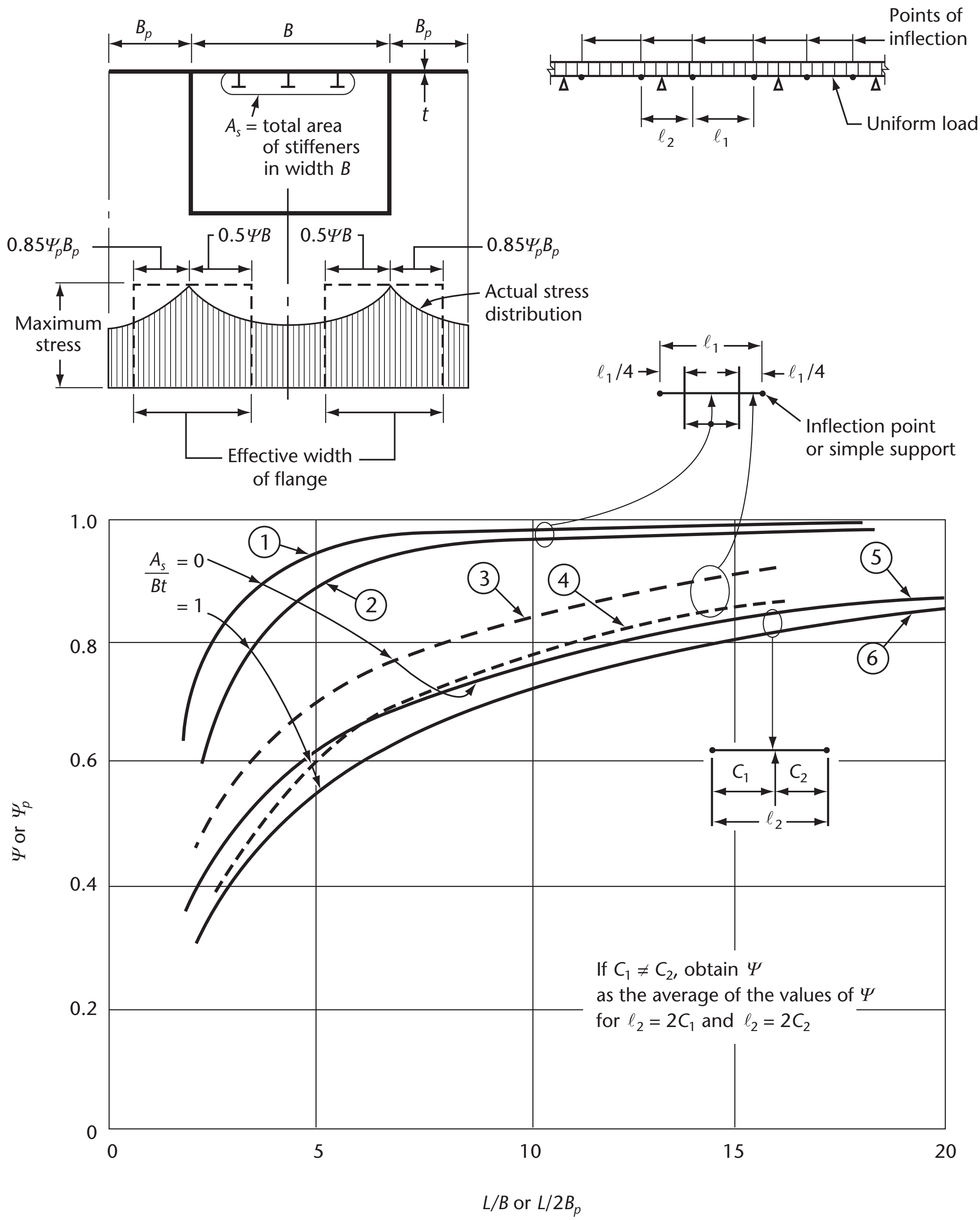


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Figure 5.6
Effective width of orthotropic deck
(See Clause 5.8.2.2.2.)

5.11.1.4 Damping
Equivalent viscous damping may be used to represent energy dissipation.

5.11.1.5 Natural frequencies
For the purpose of Clause 5.11.2, and unless otherwise specified by the Regulatory Authority, elastic undamped natural modes and frequencies of vibration shall be used. For the purpose of Clause 5.11.4 and Section 4, all relevant damped modes and frequencies shall be considered.

5.11.2 Elastic dynamic responses

5.11.2.1 Vehicle-induced vibrations
Vehicle-induced vibrations shall be accounted for by applying a dynamic load allowance, i.e., an equivalent static load equal to a fraction of the applied live load. The dynamic load allowance shall be as specified in Clause 3.8.4.5.

5.11.2.2 Wind-induced vibrations
In accordance with Clauses 3.10.4.1 and 3.10.4.2, wind-sensitive structures shall be analyzed for dynamic effects such as buffeting by turbulent or gusting winds and unstable wind–structure interaction such as vortex shedding, galloping, and flutter.

Slender or torsionally flexible structures shall be analyzed for lateral buckling, excessive thrust, and divergence.

Oscillatory deformations under wind that could lead to excessive stress levels, structural fatigue, and user inconvenience shall be avoided. Bridge decks, cable stays, and hanger cables shall be protected against excessive vortex and against oscillations induced by wind and rain. Where practical, the employment of dampers shall be considered to control excessive dynamic responses. Where dampers or shape modification are not practical, the structural system shall be changed to achieve such control.

5.11.3 Inelastic-dynamic responses

5.11.3.1 General
Energy dissipation by one or more of the following mechanisms during a major earthquake or ship collision may be taken into account:
(a) elastic or inelastic deformation of the object that could collide with the structure;
(b) inelastic deformation of the structure and its attachments;
(c) permanent displacements of the masses of the structure and its attachments; and
(d) inelastic deformation of special-purpose mechanical energy dissipaters.

5.11.3.2 Plastic hinges and yield lines
For the purpose of analysis, energy absorbed by inelastic deformation in a structural component may be assumed to be concentrated in plastic hinges and yield lines. The location of these sections may be established by successive approximation to obtain a lower bound solution for the energy absorbed. For these sections, moment-rotation hysteresis curves may be determined using verified analytic material models.

5.11.4 Analysis for collision loads
Where permitted by Section 3, dynamic analysis of ship collision may be replaced by an equivalent static elastic analysis. Where an inelastic analysis is specified, the effect of other loads that could be present shall be considered.

5.11.5 Seismic analysis
The minimum analysis requirements for seismic effects shall be as specified in Clauses 4.4.5, 4.5, and 4.11.5.

5.12 Stability and magnification of force effects

5.12.1 General

Stability effects are divided into two categories: member stability and structural stability. The stability of individual members, of the components of structural assemblies, and of structural systems shall be considered in the analysis. Stability analyses of structural assemblies and individual members shall be performed as specified in the clauses of Sections 8 to 10 and 16 that apply to the material(s) used for the members.

5.12.2 Member stability analysis for magnification of member bending moments

Member stability analysis shall be performed in order to account for
(a) the interaction between axial compression forces and bending moments or out-of-straightness of a member; and
(b) the possible increase of the bending moment magnitude between the two ends of a member.
Each member shall be considered individually.

Δ 5.12.3 Structural stability analysis for lateral sway

Structural stability analysis shall be performed to account for gravity loads undergoing lateral sway arising from horizontal loads or out-of-plumbness of the structure. This structural analysis shall encompass all members or structural components resisting the sway.

In lieu of a more refined second-order analysis, the following equations may be used:

$$M_e = M_{ns} + \delta_s M_s$$

where

M_e = moment at the end of an individual compression member

M_{ns} = moment at the end of a compression member due to loads that cause no appreciable sway, calculated using first-order elastic analysis

M_s = moment at the end of a compression member due to loads that cause appreciable sway, calculated using first-order elastic analysis

δ_s = moment magnification factor accounting for second-order effects of vertical load acting on a structure in a laterally displaced configuration

Second-order effects may be neglected if $\delta_s \le 1.05$.

The moment magnification factor, δ_s, may be calculated by one of the two following methods:

Method I

$$\delta_s = \frac{1}{1 - \dfrac{\sum P_f}{\phi_m \sum P_c}}$$

where

ΣP_f = summation of all vertical loads on the sway-resisting columns

ΣP_c = summation of all column-buckling loads in the sway-resisting system

ϕ_m = 0.75 for concrete elements

= 1.0 for structural steel elements

P_c shall be calculated similarly to P_c in Clause 8.8.5.3 for concrete elements and to C_e in Clause 10.9.4.2 for steel elements.

Method II

$$\delta_s = \frac{1}{1 - \dfrac{\sum P_f}{\phi_m \sum (S_s h)}}$$

where

ΣP_f = summation of all vertical loads on the sway-resisting columns

S_s = lateral stiffness of sway-resisting element

h = height of associated sway-resisting element

ϕ_m = 0.75 for concrete elements

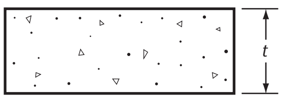
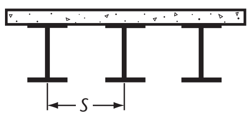
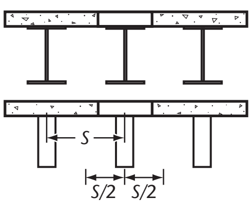
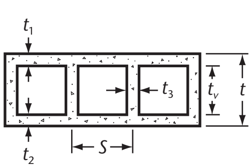
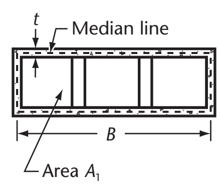
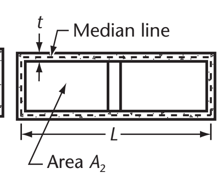
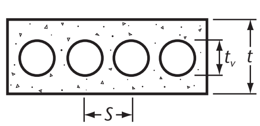
= 1.0 for structural steel elements

5.12.4 Structural stability analysis for assemblies of individual members

The structural stability of an assembly of individual members shall be considered for the condition of the buckling of such an assembly acting as a whole.

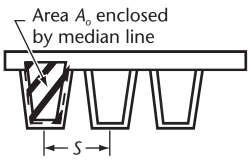
Table A5.2.2
Expressions for structural parameters

(See Clause A5.2.1.3.)

Bridge type and transverse section	Structural parameters				
	i_L	i_T	j_L	j_T	s_v
Slab 	$\dfrac{t^3}{12\left(1-v^2\right)}$	$\dfrac{t^3}{12\left(1-v^2\right)}$	$\dfrac{t^3}{6}$	$\dfrac{t^3}{6}$	May be ignored
Non-composite slab-on-girder 	$\dfrac{t^3}{12}+\dfrac{I_{xt}}{S}$ where I_{xt} = transformed moment of inertia of the girder about its own *x*-axis	$\dfrac{t^3}{12\left(1-v^2\right)}$	$\dfrac{t^3}{6}$	$\dfrac{t^3}{6}$	May be ignored
Composite slab-on-girder 	$\dfrac{I_x}{S}$ where I_x = the combined transformed moment of inertia of slab portion located in width S	$\dfrac{t^3}{12\left(1-v^2\right)}$	$\dfrac{t^3}{6}+\dfrac{J}{S}$ where J = transformed torsional inertia of the girder multiplied by $n_g = 0.88n$ for steel portions	$\dfrac{t^3}{6}$	May be ignored
Δ Voided slab with rectangular voids 	$\dfrac{t^3-t_v^3}{12}$	$\dfrac{t^3-t_v^3}{12}$	$\dfrac{2A_1^2}{B\sum\dfrac{ds}{t}}$  Intermediate webs ignored	$\dfrac{2A_2^2}{L\sum\dfrac{ds}{t}}$  Intermediate diaphragm, if any, ignored	*
Voided slab with circular voids 	$\dfrac{t^3}{12}-\dfrac{\pi t_v^4}{64S}$	$\dfrac{F_1 t^3}{12}$ (F_1 from Figure A5.2.1)	$\dfrac{F_2 t^3}{6}$ (F_2 from Figure A5.2.1)	$\dfrac{F_2 t^3}{6}$ (F_2 from Figure A5.2.1)	May be ignored

(Continued)

Table A5.2.2 (Concluded)

Bridge type and transverse section	Structural parameters				
	i_L	i_T	j_L	j_T	s_v
Δ Multi-spine girder 	$\frac{I_x}{S}$ where I_x = the combined transformed moment of inertia of slab portion located in width S	$\frac{t^3}{12(1-\nu^2)}$ for the portion of the deck between the spines. The value of i_T for the portion of the deck included in the spine is calculated by considering the total transverse stiffness of the spine, including that of the bracing and diaphragms within the box.	$\frac{4A_0^2}{S\sum\frac{ds}{n_g t}}$ where n_g = 1.0 for concrete portions = 0.88n for steel portions	$\frac{t^3}{6}$	May be ignored for slab between spines. See note (*) for portion within spine. The stiffness of internal braces in spines may be included.

* $s_v = \left[\frac{t_1^3 + t_2^3}{S^2}\right]\frac{E}{G}\left[\frac{t_3^3 S}{S t_3^3 + \left(t_1^3 + t_2^3\right)\left[\frac{t + t_v}{2}\right]}\right]$ *for voided slabs with rectangular voids.*

Note: *All parameters are in terms of deck slab concrete units.*

Table A5.2.3
Torsional constant, *K*, for rectangular sections where *a* ≥ *b*

(See Clause A5.2.1.3 and Table A5.2.4.)

a/b	1	1.2	1.5	2.0	2.5	3.0	4.0	5.0	10.0	≥ 420
K	0.141	0.166	0.196	0.229	0.249	0.263	0.281	0.291	0.312	0.333

8.4.4 Anchorages, mechanical connections, and ducts

8.4.4.1 Anchorages for post-tensioning tendons

When tested in an unbonded condition, anchorages for post-tensioning tendons shall develop at least 95% of the specified tensile strength of the tendons without exceeding the anticipated set. After tensioning and seating, anchorages shall sustain applied loads without slippage, distortion, or other changes that result in loss of prestress. The dimensions and details of the anchorages, including any reinforcement immediately behind the anchorages, shall be based on the specified strength of the tendon and the specified strength of the concrete at transfer.

Anchorages for external unbonded post-tensioning tendons shall also meet Approved dynamic tests.

8.4.4.2 Anchorages for reinforcing bars

Mechanical anchorage devices shall be capable of developing the yield strength of the reinforcing bars without damage to the concrete.

8.4.4.3 Mechanical connections for post-tensioning tendons

When tested in an unbonded condition, couplers for post-tensioning tendons shall develop 95% of the specified tensile strength of the tendons without exceeding the anticipated set. Couplers for external unbonded post-tensioning tendons shall also meet Approved dynamic tests.

Couplers and their components shall be enclosed in housings. The housings shall be long enough to permit the necessary movements and shall be provided with fittings to allow complete grouting. Couplers shall not reduce the elongation at rupture below the requirements of the tendon itself. Couplers shall not be used at points of sharp tendon curvature or in the vicinity of points of maximum moments.

8.4.4.4 Mechanical connections for reinforcing bars

Mechanical connections for reinforcing bars shall develop, in tension or compression (as required), the greater of 120% of the specified yield strength of bars or 110% of the mean yield strength of the actual bars used to test the mechanical connection.

The total slip of the reinforcing bars within the splice sleeve of the connector after loading in tension to $0.5f_y$ and relaxing to $0.05f_y$ shall not exceed the following measured displacements between gauge points straddling the splice sleeve:

(a) for bars sizes up to and including 45M: 0.25 mm; and
(b) for 55M bars: 0.75 mm.

8.4.4.5 Ducts

8.4.4.5.1 General

Sheaths for internal post-tensioning ducts shall be made of bright steel, galvanized steel, or plastic. The sheaths shall be corrugated and shall be non-reactive with concrete, tendons, and grout. The shape of corrugations shall be such that the sheaths can be completely filled with grout. Sheaths for external post-tensioning shall be made of plastic.

8.4.4.5.2 Size

For single-strand or bar tendons, the inside diameter of the sheaths for post-tensioning ducts shall be at least 6 mm larger than the nominal diameter of the strand or bar. For multiple-strand tendons, the inside cross-sectional area of the sheath shall be at least twice the cross-sectional area of the prestressing tendon.

The inside diameter of a circular sheath or an equivalent diameter of a non-circular sheath shall not exceed 40% of the least gross concrete thickness at the duct.

8.4.4.5.3 Steel sheaths

Sheaths shall be watertight under an internal pressure of 350 kPa. Rigid steel sheaths shall have a wall thickness of at least 0.6 mm and shall permit bending of the sheath to a minimum inside radius of curvature of 9 m without distress. Semi-rigid steel sheaths shall have a wall thickness of at least 0.25 mm and shall permit the bending of the sheath to a minimum inside radius of curvature of 3.5 m without distress.

8.4.4.5.4 Plastic sheaths

Unless otherwise Approved, plastic sheaths, including their splices, shall be made of high-density polyethylene conforming to ASTM D 3350 Cell Classification 324420C, shall be vapour tight, and shall remain vapour tight after tendon installation and stressing. The polyethylene sheath shall be manufactured in accordance with ASTM D 2239.

Plastic sheaths shall not be used when the radius of curvature of the tendon is less than 10 m. The sheaths shall be capable of bending to the specified minimum radius of curvature without local buckling or damage. The sheath wall thickness shall be such that for the specified minimum radius of curvature the remaining wall thickness, after a tendon movement of 750 mm under a tendon stress of 80% of the specified strength, will not be less than 1 mm. For curved sheaths, the radial force exerted by a single strand on the sheath wall shall not exceed 40 kN/m.

The stiffness of plastic sheaths shall be such that

(a) for sheaths with an inside diameter of 50 mm or less, a 3 m length supported at the ends will not deflect, under its own weight, more than 75 mm at room temperature (i.e., not less than 20 °C);

(b) for sheaths with an inside diameter of more than 50 mm, a 6 m length supported at the ends will not deflect, under its own weight, more than 75 mm at room temperature; and

(c) the sheath shall not dent more than 3 mm under a point load of 445 N applied through a 10M reinforcing bar between the corrugation ribs at room temperature.

Sheaths and their splices for external post-tensioning shall be smooth, seamless, and capable of withstanding a grouting pressure of at least 1000 kPa.

8.4.4.5.5 Vents and drains

Ducts shall be provided with vents and drains at appropriate locations.

8.4.4.5.6 Ducts at deviators

Within deviators, the sheaths for post-tensioning tendons shall consist of

(a) galvanized steel pipe in accordance with ASTM A53/A53M, Type E, Grade B, with a wall thickness not less than 3 mm, and bent to conform to the tendon alignment; or

(b) an Approved sheath detail.

Δ 8.4.4.6 Anchor rods and studs

Anchor rods and studs shall comply with Section 10.

8.4.5 Grout

8.4.5.1 Post-tensioning

Unless otherwise Approved, grout for post-tensioning ducts shall comply with CAN/CSA-A23.1 and have a compressive strength of at least 35 MPa at 28 days.

8.4.5.2 Other applications

Grout for other applications shall be Approved.

8.4.6 Material resistance factors

The material resistance factors specified in Table 8.1 shall be used to calculate the factored resistance

(e) The transformed area of bonded reinforcement may be included in the calculation of section properties. Before grouting, the loss of concrete area due to post-tensioning ducts, coupler sheaths, or transition trumpets shall be considered, except where such loss of area is insignificant. The modular ratio, n, shall not be taken as less than 6. An effective modular ratio of $2n$ may be used to transform the compression reinforcement for stress computations corresponding to permanent loads.

8.8.3 Assumptions for the ultimate limit states

In addition to the conditions of equilibrium and compatibility of strains, the calculations for the ultimate limit states shall be based on the material resistance factors specified in Clause 8.4.6 and the following shall apply to such calculations:

(a) Strain in the concrete shall be assumed to vary linearly over the depth of the section, except for deep beams, which shall satisfy the requirements of Clause 8.10.

(b) Strain changes in bonded reinforcement shall be assumed to be equal to strain changes in the surrounding concrete.

(c) The maximum usable strain at the extreme concrete compression fibre shall be assumed to be 0.0035 unless the concrete is confined and a higher value of strain can be justified. In the latter case, a strain compatibility analysis shall be used.

(d) Except for the strut-and-tie model of Clause 8.10, the stress in the reinforcement shall be taken as the value of the stress determined using strain compatibility based on a stress-strain curve representative of the steel reinforcement to be used, multiplied by ϕ_s or ϕ_p.

(e) The tensile strength of the concrete shall be neglected in the calculation of the factored flexural resistance.

(f) The relationship between concrete strain and the concrete compressive stress may be assumed to be rectangular, parabolic, or any other shape that results in a prediction of strength in substantial agreement with the results of comprehensive tests. In this regard, an equivalent rectangular concrete stress distribution may be used, i.e., a concrete stress of $\alpha_1 \phi_c f_c'$ is uniformly distributed over an equivalent compression zone, bounded by the edges of the cross-section and a straight line parallel to the neutral axis at a distance $a = \beta_1 c$ from the fibre of maximum compressive strain, where c is the shortest length between the fibre of maximum compressive strain and the neutral axis, $\alpha_1 = 0.85 - 0.0015 f_c' \geq 0.67$ and $\beta_1 = 0.97 - 0.0025 f_c' \geq 0.67$.

8.8.4 Flexural components

8.8.4.1 Factored flexural resistance

The factored flexural resistance shall be calculated in accordance with Clause 8.8.3.

8.8.4.2 Tendon stress at the ultimate limit states

The value of f_{ps} for components with bonded tendons shall be computed using a method based on strain compatibility and using stress-strain curves representative of the steel, except that if c/d_p is less than or equal to 0.5, the following expression may be used:

$$f_{ps} = f_{pu} (1 - k_p c/d_p)$$

where k_p is 0.3 for low-relaxation strands, 0.4 for smooth high-strength bars, and 0.5 for deformed high-strength bars, and the value of c shall be determined assuming a stress of f_{ps} in the tendons.

For components with unbonded tendons, f_{ps} shall be taken as f_{se} unless a detailed analysis accounting for deformations demonstrates that a higher value can be used.

External tendons shall be treated as unbonded tendons.

8.8.4.3 Minimum reinforcement

The total amount of reinforcement shall be such that the factored flexural resistance, M_r, of the component is at least 1.20 times the cracking moment. This requirement may be waived if the factored flexural resistance provided is at least one-third greater than the minimum resistance required for factored loads.

8.8.4.4 Cracking moment

A component shall be assumed to crack when the moment at a section is such that a tensile stress of f_{cr}, as specified in Clause 8.4.1.8, is induced in the concrete.

8.8.4.5 Maximum reinforcement

The amount of reinforcement provided shall be such that the factored flexural resistance, M_r, is developed with c/d not exceeding 0.5. This requirement may be waived if it is demonstrated to the satisfaction of the Regulatory Authority that the consequences of reinforcement not yielding are acceptable.

Δ 8.8.4.6 Prestressed concrete stress limitations

The stresses in the concrete shall not exceed the following:

(a) At transfer and during construction:
   (i) compression: $0.60f'_{ci}$;
   (ii) tension in components without reinforcing bars in the tension zone: $0.50f_{cri}$. Where the calculated tensile stress exceeds $0.50f_{cri}$, reinforcing bars in which the tensile stress is assumed to be 240 MPa shall be provided to resist the total tensile force in the concrete, calculated on the basis of an uncracked section; and
   (iii) tension at joints in segmental components:
      (1) without reinforcing bars passing through the joint in the tension zone: zero; and
      (2) with reinforcing bars passing through the joint in the tension zone: $0.50f_{cri}$.
   Where the calculated tensile stress is between zero and $0.50f_{cri}$, reinforcing bars in which the tensile stress is assumed to be 240 MPa shall be provided to resist the total tensile force in the concrete calculated on the basis of an uncracked section.

(b) At the serviceability limit states, if the tension in the concrete exceeds f_{cr}, Clause 8.12 shall apply. Tension shall not be permitted across the joints of segmental components unless bonded reinforcing bars pass through the joints in the tensile zone.

(c) In prestressed slabs with circular voids, the average compressive stress due to effective longitudinal prestress alone shall not exceed 6.5 MPa. In post-tensioned slabs with circular voids, the following shall apply:
   (i) an effective transverse prestress shall be provided to give a compressive stress of 4.5 MPa in the concrete above the longitudinal voids; and
   (ii) the thicknesses of the concrete above and below the voids shall not be less than 175 mm and 125 mm, respectively.

8.8.5 Compression components

8.8.5.1 General

The proportioning of cross-sections subject to combined flexure and axial compression shall be in accordance with Clause 8.8.3.

8.8.5.2 Slenderness effects

The proportioning of compression components shall be based on forces and moments determined from an analysis of the structure. Except as permitted by Clause 8.8.5.3, such an analysis shall include the influence of axial loads and variable moment of inertia on component stiffness and moments, the effect of deflections on the moments and forces, and the effects of the duration of the loads and prestressing forces.

8.9.3.8 Determination of ε_x

In lieu of more accurate calculations, ε_x shall be calculated as follows:

$$\varepsilon_x = \frac{M_f/d_v + V_f - V_p + 0.5N_f - A_{ps}f_{po}}{2\left(E_sA_s + E_pA_{ps}\right)}$$

Evaluation of this equation shall be based on the following:

(a) V_f and M_f are positive quantities and M_f shall not be less than $(V_f - V_p)d_v$.
(b) N_f shall be taken as positive for tension and negative for compression. For rigid frames and rectangular culverts, the value of N_f used to determine ε_x may be taken as twice the compressive axial thrust calculated by elastic analysis.
(c) A_s and A_{ps} are the areas of reinforcing bars and prestressing tendons in the half-depth of the section containing the flexural tension zone.
(d) f_{po} may be taken as $0.7f_{pu}$ for bonded tendons outside the transfer length and f_{pe} for unbonded tendons.
(e) In calculating A_s, the area of bars that terminate less than their development length from the section under consideration shall be reduced in proportion to their lack of full development.
(f) If the value of ε_x is negative, it shall be taken as zero or recalculated with the denominator replaced by $2(E_sA_s + E_pA_{ps} + E_cA_{ct})$. However, ε_x shall not be less than -0.20×10^{-3}.
(g) For sections closer than d_v to the face of the support, the value of ε_x calculated at d_v from the face of the support may be used in evaluating θ and β.
(h) If the axial tension is large enough to crack the flexural compression face of the section, the resulting increase in ε_x shall be taken into account. In lieu of more accurate calculations, the value calculated from the equation shall be doubled.
(i) θ and β may be determined from Clause 8.9.3.7 using a value of ε_x that is greater than that calculated from the equation in this Clause. However, ε_x shall not be greater than 3.0×10^{-3}.

8.9.3.9 Proportioning of transverse reinforcement

Near locations where the spacing, s, of the transverse reinforcement changes, the quantity A_v/s may be assumed to vary linearly over a length, h, centred on the location where the spacing changes.

8.9.3.10 Extension of longitudinal reinforcement

At every section, the longitudinal reinforcement shall be designed to resist the additional tensile forces caused by shear as specified in Clauses 8.9.3.11 and 8.9.3.12. Alternatively, for members not subjected to significant tension or torsion, these requirements may be satisfied by extending the flexural tension reinforcement a distance of $d_v\cot\theta$ beyond the location required by flexure alone.

8.9.3.11 Longitudinal reinforcement on the flexural tension side

Longitudinal reinforcement on the flexural tension side shall be proportioned so that at all sections the factored resistance of the reinforcement, taking account of the stress that can be developed in this reinforcement, is greater than or equal to $F_{\ell t}$, calculated as follows:

$$F_{\ell t} = \frac{M_f}{d_v} + 0.5N_f + \left(V_f - 0.5V_s - V_p\right)\cot\theta$$

where M_f and V_f are taken as positive quantities and N_f is positive for axial tension and negative for axial compression. In this equation, d_v may be taken as the flexural lever arm at the factored resistance.

8.9.3.12 Longitudinal reinforcement on the flexural compression side

Longitudinal reinforcement on the flexural compression side of the section shall be proportioned so that the factored tensile resistance of this reinforcement, taking account of the stress that can be developed in this reinforcement, shall be greater than or equal to the force $F_{\ell c}$, calculated as follows:

$$F_{\ell c} = 0.5N_f + \left(V_f - 0.5V_s - V_p\right)\cot\theta - \frac{M_f}{d_v}$$

where M_f and V_f are taken as positive quantities and N_f is positive for axial tension and negative for axial compression.

8.9.3.13 Compression fan regions

In regions adjacent to maximum moment locations, the cross-sectional area of longitudinal reinforcement on the flexural tension side of the member need not exceed the cross-sectional area required to resist the maximum moment acting alone. This exception shall apply only when the support or the load at the maximum moment location introduces direct compression into the flexural compression face of the member and the member is not subject to significant torsion.

8.9.3.14 Anchorage of longitudinal reinforcement at exterior supports

At exterior direct-bearing supports, the longitudinal reinforcement on the flexural tension side for the member shall be capable of resisting a tensile force of $(V_f - 0.5V_s - V_p)\cot\theta + 0.5N_f$, where V_s is based on the transverse reinforcement provided within a length of $d_v\cot\theta$ from the face of the support. However, V_s shall not be taken as greater than V_f. The tension force in the reinforcement shall be developed at the point where a line inclined at angle θ to the longitudinal axis and extending from the inside edge of the bearing area intersects the centroid of the reinforcement.

8.9.3.15 Transverse reinforcement for combined shear and torsion

For sections subjected to combined shear and torsion, the transverse reinforcement provided shall be at least equal to the sum of that required for shear and that required for the coexisting torsion.

8.9.3.16 Transverse reinforcement for torsion

The amount of transverse reinforcement required for torsion shall be such that T_r is greater than or equal to T_f.

8.9.3.17 Factored torsional resistance

The value of T_r shall be calculated as follows:

$$T_r = 2A_o \frac{\phi_s A_t f_y}{s}\cot\theta$$

where A_o is taken as $0.85A_{oh}$ and θ is as specified in Clause 8.9.3.6 or 8.9.3.7.

Δ 8.9.3.18 Cross-sectional dimensions to avoid crushing for combined shear and torsion

The cross-sectional dimensions to avoid crushing for combined shear and torsion shall be as follows:

(a) For box sections:

$$\frac{V_f - V_p}{b_v d_v} + \frac{T_f p_h}{1.7A_{oh}^2} \le 0.25\phi_c f_c'$$

If the wall thickness of the box section is less than A_{oh}/p_h, the second term in this expression shall be replaced by $T_f/(1.7A_{oh}t)$, where t is the wall thickness at the location where the stresses are being checked.

(b) For other sections:

$$\sqrt{\left[\frac{V_f - V_p}{b_v d_v}\right]^2 + \left[\frac{T_f p_h}{1.7A_{oh}^2}\right]^2} \le 0.25\phi_c f_c'$$

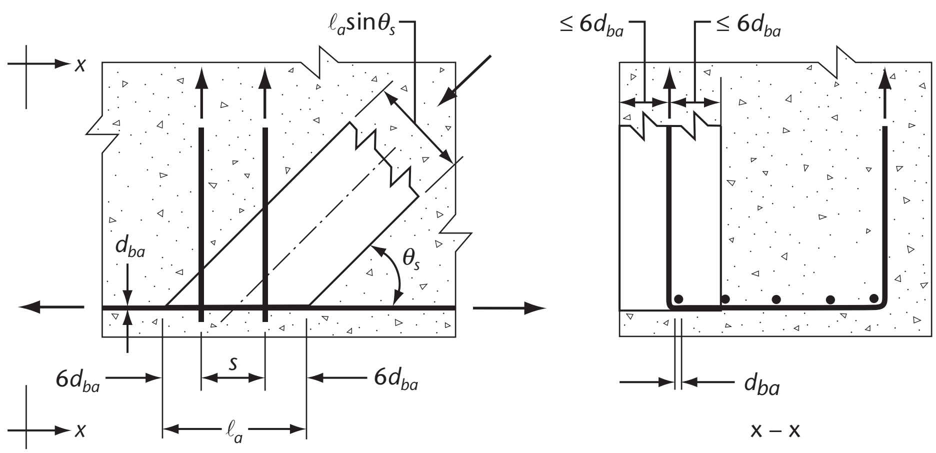
8.10.3 Proportioning of a compressive strut

8.10.3.1 Strength of strut

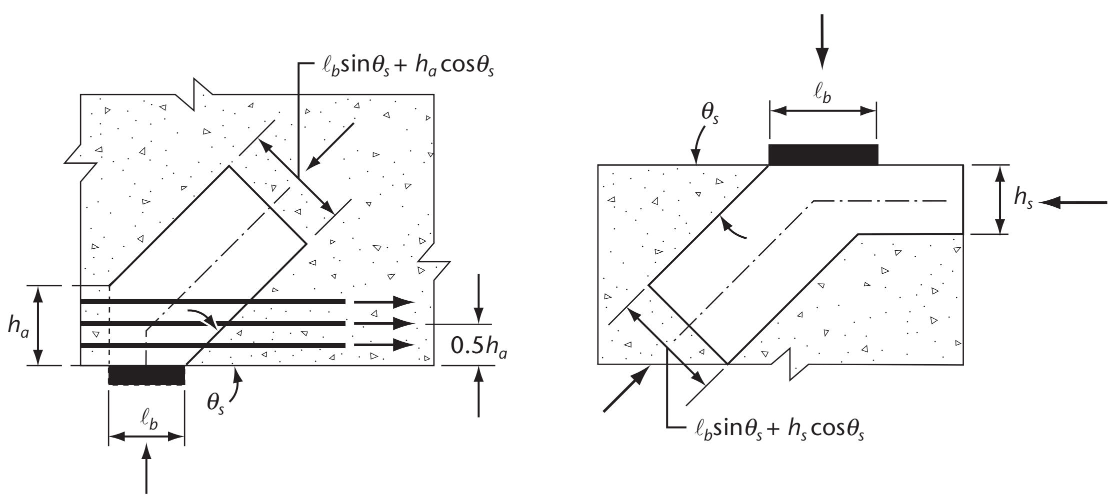
The dimensions of the strut shall be large enough to ensure that the calculated compressive force in the strut does not exceed $\phi_c A_{cs} f_{cu}$, where A_{cs} and f_{cu} are determined in accordance with Clauses 8.10.3.2 and 8.10.3.3, respectively.

8.10.3.2 Effective cross-sectional area of strut

The value of A_{cs} shall be calculated by considering both the influence of the anchorage conditions at the ends of the strut, as shown in Figure 8.4, and the available concrete area.



(a) Strut anchored by reinforcement



(b) Strut anchored by bearing and reinforcement

(c) Strut anchored by bearing and strut

Figure 8.4
Influence of anchorage conditions on effective cross-sectional area of strut
(See Clause 8.10.3.2.)

8.10.3.3 Limiting compressive stress in strut

The value of f_{cu} shall be calculated as follows:

$$f_{cu} = \frac{f'_c}{0.8 + 170\varepsilon_1} \leq \alpha_1 f'_c$$

where ε_1 is calculated as $\varepsilon_s + (\varepsilon_s + 0.002)\cot^2\theta_s$, in which θ_s is the smallest angle between the compressive strut and the adjoining tensile ties and ε_s is the tensile strain in the tensile tie inclined at θ_s to the compressive strut.

8.10.3.4 Reinforced strut

If the compressive strut contains reinforcement that is parallel to the strut and has been detailed to develop its yield stress in compression, the calculated force in the strut shall not exceed $\phi_c f_{cu} A_{cs} + \phi_s f_y A_{ss}$. The strut shall be reinforced with lateral ties in accordance with Clause 8.14.4.3.

8.10.4 Proportioning of a tension tie

8.10.4.1 Strength of tie

The cross-sectional area of the reinforcement in a tension tie shall be large enough to ensure that the calculated tensile force in the tie does not exceed $\phi_s f_y A_{st} + \phi_p f_{py} A_{ps}$, where A_{st} is the cross-sectional area of the reinforcing bars in the tie and A_{ps} is the cross-sectional area of the tendons in the tie.

8.10.4.2 Anchorage of tie

The tension tie reinforcement shall be anchored so that it is capable of resisting the calculated tension in the reinforcement at the inner edge of the node region. For straight bars extending a distance x beyond the inner edge of the node region, where x is less than ℓ_d, the calculated stress shall not exceed $f_y(x/\ell_d)$, where ℓ_d is calculated in accordance with Clause 8.15.2.

8.10.5 Proportioning of node regions

Δ 8.10.5.1 Stress limits in node regions

Unless special confining reinforcement is provided, the calculated concrete compressive stress in the node regions shall not exceed the following (with α_1, as specified in Clause 8.8.3):

(a) $\alpha_1 \phi_c f'_c$ in node regions bounded by compressive struts and bearing areas;
(b) $0.88\alpha_1 \phi_c f'_c$ in node regions anchoring a tension tie in only one direction; and
(c) $0.76\alpha_1 \phi_c f'_c$ in node regions anchoring tension ties in more than one direction.

8.10.5.2 Satisfying stress limits in node regions

The stress limits in node regions may be considered satisfied if the following two conditions are met:

(a) the bearing stress in the node regions produced by concentrated loads or reactions does not exceed the stress limits specified in Clause 8.10.5.1; and
(b) the tensile tie reinforcement is uniformly distributed over an effective area of concrete at least equal to the tensile tie force divided by the stress limits specified in Clause 8.10.5.1.

8.10.6 Crack control reinforcement

Except for slabs and footings, components or regions that have been designed in accordance with Clauses 8.10.1 to 8.10.5 shall contain an orthogonal grid of reinforcing bars near each face. The spacing of this reinforcement shall not exceed 300 mm. The ratio of reinforcement area to gross concrete area shall not be less than 0.003, but the reinforcement need not be more than 1500 mm^2/m in each face and in each direction. If located within the tension tie, the crack control reinforcement may also be considered tension tie reinforcement.

8.11.2.3 Corrosion protection for reinforcement, ducts, and metallic components

Unless otherwise Approved, steel reinforcement, anchorages, and mechanical connections specified for use within 75 mm of a surface exposed to moisture containing de-icing chemicals shall have an Approved protective coating, be protected by other Approved methods of corrosion protection or prevention, or be of non-corrosive materials. Exposed inserts, fasteners, and plates shall be protected from corrosion by Approved methods. Sheaths for internal post-tensioning ducts specified for use within 100 mm of a surface subject to moisture containing de-icing chemicals shall be made of non-corroding material or with an Approved coating. The ends of pretensioning strands shall be protected by Approved methods when they are not encased in concrete.

8.11.2.4 Sulphate-resistant cements

Sulphate-resistant cement shall be specified for concrete in deep foundation units, footings, buried structures made of reinforced concrete, or other substructure components exposed to soils or water to an extent sufficient to cause a strong sulphate attack on concrete. Protection against sulphate attack shall be in accordance with CAN/CSA-A23.1.

8.11.2.5 Alkali-reactive aggregates

Aggregates for concrete shall be tested for susceptibility to alkali aggregate reaction. The evaluation and use of aggregates susceptible to alkali aggregate reaction shall be in accordance with CAN/CSA-A23.1 and CAN/CSA-A23.2-27A.

8.11.2.6 Drip grooves

Continuous drip grooves shall be formed on the underside of the bridge deck. The grooves shall be located close to the fascia and shall have minimum dimensions for depth and width of 20 mm and 50 mm, respectively. At expansion joints without joint armouring, the end of the concrete deck slab shall be provided with a drip groove. If joint armouring is provided, it shall cover the end of the deck slab and extend at least 50 mm below the concrete in order to form a drip projection.

8.11.2.7 Waterproofing

Unless otherwise Approved, concrete decks that are expected to be salted for winter maintenance or are exposed to a marine environment shall be waterproofed with an Approved waterproofing system.

Δ

Table 8.5
Minimum concrete covers and tolerances
(See Clause 8.11.2.2.)

Environmental exposure	Component	Reinforcement/ steel ducts	Concrete covers and tolerances: Cast-in-place concrete, mm	Concrete covers and tolerances: Precast concrete, mm
De-icing chemicals; spray or surface runoff containing de-icing chemicals; marine spray	(1) Top of bottom slab for rectangular voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	40 ± 10 — 60* ± 10	40 ± 10 55 ± 5 60* ± 10
	(2) Top surface of buried structure with less than 600 mm fill† Top surface of bottom slab of buried structure	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	50 ± 10 65 ± 5 70* ± 10
	(3) Top surface of structural component, except (1) and (2) above‡	Reinforcing steel Pretensioning strands	70 ± 20 —	55 ± 10 70 ± 5
		Post-tensioning ducts Longitudinal Transverse ($d_d \le 60$ mm Transverse ($d_d > 60$ mm)	 130* ± 15 90* ± 15 130* ± 15	 120* ± 10 80* ± 10 120* ± 10
	(4) Soffit of precast deck form	Reinforcing steel Pretensioning strands	— —	40 ± 10 38 ± 3
	(5) Soffit of slab less than 300 mm thick or soffit of top slab of voided deck	Reinforcing steel Pretensioning strands Post-tensioning ducts	50 ± 10 — 70* ± 10	45 ± 10 60 ± 5 65* ± 10
	(6) Soffit of slab 300 mm thick or thicker or soffit of structural component, except (4) and (5) above	Reinforcing steel Pretensioning strands Post-tensioning ducts	60 ± 10 — 80* ± 10	50 ± 10 65 ± 5 70* ± 10
	(7) Vertical surface of arch, solid or voided deck, pier cap, T-beam, or interior diaphragm	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 10 — 90* ± 10	60 ± 10 75 ± 5 80* ± 10
	(8) Inside vertical surface of buried structure or inside surface of circular buried structure	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	50 ± 10 65 ± 5 70* ± 10
	(9) Vertical surface of structural component, except (7) and (8) above	Reinforcing steel Pretensioning strands Post-tensioning ducts	70 ± 20 — 90* ± 15	55 ± 10 70 ± 5 75* ± 10
	(10) Precast T-, I-, or box girder	Reinforcing steel Pretensioning strands Post-tensioning ducts	— — —	35 +10 or –5 50 ± 5 55* ± 10

(Continued)

which case the maximum spacing shall not exceed 450 mm. Welded wire fabric of equivalent area may be used for ties.

Ties shall be arranged so that every corner bar and alternate longitudinal bar has lateral support provided by the corner of a tie having an included angle of not more than 135°, and no bar shall be farther than 150 mm clear on either side from such a laterally supported bar. Ties shall be located vertically not more than half a tie spacing above the footing or from other support, and not more than half a tie spacing below the lowest horizontal reinforcement in the components supported above.

8.14.5 Reinforcement for shear and torsion

8.14.5.1 Transverse reinforcement

Transverse reinforcement shall consist of one of the following forms:

(a) stirrups perpendicular to the axis of the component or at an angle of 45° or more to the longitudinal tension reinforcement, with the inclined stirrups oriented to intercept potential cracks;

(b) well-anchored tendons that are detailed and constructed to minimize seating and time-dependent losses and are perpendicular to the axis of the component or at an angle of 45° or more to the longitudinal tension reinforcement, with the inclined tendons oriented to intercept potential diagonal cracks;

(c) spirals; or

(d) welded wire fabric, with the wires perpendicular to the axis of the component.

Transverse reinforcement for shear shall be anchored in accordance with Clause 8.15.1.5.

8.14.5.2 Torsional reinforcement

Torsional reinforcement shall consist of longitudinal reinforcement and one of the following forms of transverse reinforcement:

(a) closed stirrups perpendicular to the axis of the component and anchored with 135° hooks;

(b) a closed cage of welded wire fabric perpendicular to the axis of the component; or

(c) spirals.

Δ 8.14.6 Maximum spacing of reinforcement for shear and torsion

If V_f is less than or equal to $(0.10\phi_c f_c' b_v d_v + V_p)$ and T_f is less than or equal to $0.25T_{cr}$, the spacing of the transverse reinforcement, s, measured in the longitudinal direction, shall not exceed the lesser of 600 mm or $0.75d_v$.

If V_f exceeds $(0.10\phi_c f_c' b_v d_v + V_p)$, or if T_f exceeds $0.25T_{cr}$, s shall not exceed the lesser of 300 mm or $0.33d_v$.

The spacing of longitudinal bars for torsion distributed around the perimeter of the stirrups shall not exceed 300 mm. At least one longitudinal bar with a diameter not less than 0.06 times the spacing of the stirrups and not smaller than 15M shall be placed inside each corner of the closed stirrups. The corner bars shall be anchored in accordance with Clause 8.15.2 or 8.15.5.

8.15 Development and splices

8.15.1 Development

8.15.1.1 General

The calculated tension or compression in the reinforcement at each section shall be developed on each side of that section by one or more of embedment length, end anchorage, and a hook or mechanical device. Hooks or mechanical devices may be used in developing the strength of the bars in tension only.

Tension reinforcement may be anchored by extending it into the compression zone or bending it and making it continuous with the reinforcement on the opposite face of the member.

Reinforcement shall extend beyond the point at which it is theoretically no longer required to resist flexure in accordance with the requirements of Clause 8.9.3.10.

The value of $\sqrt{f_c'}$ in Clause 8.4.1.8 used to compute f_{cr} in Clauses 8.15.2.2, 8.15.2.3, 8.15.3.1, 8.15.5.2, and 8.15.7.2 shall not exceed 8.0.

8.15.1.2 Positive moment reinforcement

At least 33% of the positive moment reinforcement in simply supported members and 25% of the positive moment reinforcement in continuous members shall extend along the same face of the member into the support. Such reinforcement shall extend at least 150 mm beyond the centreline of the exterior support and shall satisfy the requirements of Clause 8.9.3.10.

When a flexural member is part of the lateral-load-resisting system, the positive moment reinforcement required to be extended into the support shall be anchored so as to develop the yield strength in tension at the face of the support.

8.15.1.3 Negative moment reinforcement

Negative moment reinforcement in a continuous, restrained, or cantilever member, or any member of a rigid frame, shall be anchored in or through the supporting member by embedment length, hooks, or mechanical anchorage.

At least 33% of the total reinforcement provided for negative moment at the support shall have an embedment length beyond the point of inflection not less than the effective depth of the member, $12d_b$, or 0.06 of the clear span, whichever is greatest.

8.15.1.4 Special members

Adequate end anchorage shall be provided for tension reinforcement in flexural members where stress in the reinforcement is not directly proportional to moment. Such members include, but are not limited to, sloped, stepped, or tapered footings, brackets, deep beams, and members in which the tension reinforcement is not parallel to the compression face.

8.15.1.5 Anchorage of transverse reinforcement

Transverse reinforcement provided for shear shall extend as close to the compression and tension surfaces of the member as cover requirements and the proximity of other reinforcement permit.

Transverse reinforcement provided for shear shall be anchored at both ends by one of the following:

(a) For 15M and smaller bars and MD200 and smaller wire, a standard hook, as specified in Clause 8.14.1.1, around longitudinal reinforcement.

(b) For 20M and 25M stirrups, a standard hook, as specified in Clause 8.14.1.1, around longitudinal reinforcement, plus an embedment between mid-depth of the member and the outside end of the hook equal to or greater than $0.33\ell_d$.

(c) For each leg of welded smooth wire fabric forming single U-stirrups,
   - (i) two longitudinal wires running at a 50 mm spacing along the member at the top of the U; or
   - (ii) one longitudinal wire located not more than $0.25d$ from the compression face and a second wire closer to the compression face and spaced not less than 50 mm from the first. The second wire may be located on the stirrup leg beyond a bend or on a bend with an inside diameter of not less than $8d_b$.

(d) For each end of a single leg stirrup of welded smooth or deformed wire fabric, two longitudinal wires at a minimum spacing of 50 mm, with the inner wire at least $0.25d$ from the mid-depth of the member. The outer longitudinal wire at the tension face shall not be farther from that face than the portion of primary flexural reinforcement closest to the face.

(e) A mechanical anchor capable of developing the yield strength of the bar.

Pairs of U-stirrups or ties placed so as to form a closed unit shall be considered properly spliced when lapped for a length of $1.3\ell_d$. In components with a depth of at least 450 mm, such splices having A_bf_y not

The embedment depth of the anchor device shall be adequate to develop a factored tensile resistance at least equal to the factored shear force being transferred, except that for anchors transferring load by shear friction, the factored tensile resistance shall not be less than V_f/μ.

When the shear is transmitted by shear friction, shear lugs shall be considered effective only if they are perpendicular to the shear force and are located in a zone where compression is developed between the attachment and the concrete.

When the shear acts toward a free edge, the factored tensile resistance of concrete shall be based on a uniform tensile stress of $0.75\phi_c f_{cr}$ acting on the effective area as follows:

(a) for anchor bolts, studs, and bars, the effective stress area shall be that defined by the intersection of half of the 90° pyramid and the free edge;
(b) for shear lugs, the effective stress area shall be defined by the intersection of projected 45° planes from the edges of the shear lug and the free edge. The bearing area of the shear lug shall be excluded from the effective stress area; and
(c) where shear friction is employed, the effective stress area shall be defined by the intersection of the projected 45° planes and the free edge.

If the factored shear resistance of concrete is less than the factored shear force, reinforcement capable of resisting the factored shear force shall be provided across the potential failure surface.

8.16.7.4 Reinforcement

The reinforcement shall be proportioned for the factored force to be transferred by the anchor and shall be detailed to develop the required force on both sides of the potential failure surface.

8.16.7.5 Compressive resistance of concrete

The compressive resistance of concrete shall be determined in accordance with Clause 8.8.7.1. When compression exists over the entire base plate area, the bearing pressure on the concrete may be assumed to be uniform over an area equal to the width of the base plate multiplied by a length equal to the length of the base plate minus twice the eccentricity of the factored load normal to the base plate.

The moment resisted by the anchors shall be taken as the couple formed by the tensile resistance of the anchor determined in accordance with Clause 8.16.7.6.3 or 8.16.7.6.4, as applicable, and by the compressive resistance of the concrete determined in accordance with Clause 8.8.7.1.

8.16.7.6 Design requirements for anchors

8.16.7.6.1 General

Anchors shall have a minimum diameter of 15 mm.

8.16.7.6.2 Tensile resistance of bolts and studs

The factored tensile resistance of an anchor bolt or stud shall be as specified in Clause 10.19.2.1.

8.16.7.6.3 Tensile resistance of reinforcing bars

The factored tensile resistance, f_r, of an anchor made of a reinforcing bar shall be taken as $\phi_s f_y A_b$.

8.16.7.6.4 Shear resistance

8.16.7.6.4.1

When shear force is transferred through shear friction, the factored shear resistance of the anchorage system, V_r , shall be calculated as μF_r. The values of μ may be taken as follows:

(a) as-rolled steel plate against concrete or grout where the plate is embedded a full plate thickness below the concrete surface: 0.8;
(b) as-rolled steel plate against concrete or grout, with the contact plane coinciding with the concrete surface: 0.6; and
(c) as-rolled steel plate against grout, with the contact plane exterior to the concrete surface: 0.5.

Δ 8.16.7.6.4.2

When shear force is transferred through bearing, the shear resistance of the anchor shall be taken as the smallest of
(a) the factored shear resistance of an anchor rod or stud as specified in Clause 10.19.2.2;
(b) $V_r = 0.8\phi_s f_y A_b$ (for an anchor made of a reinforcing bar); and
(c) B_r as specified in Clause 8.16.7.3.

Δ 8.16.7.6.5 Combined tension and shear

An anchor rod or a stud required to develop resistance to combined tension and shear through bearing shall be proportioned in accordance with Clause 10.19.2.3.

An anchor made of a reinforcing bar required to develop resistance to combined tension and shear through bearing shall be proportioned in such a manner that $(V_f/V_r)^2 + (F_f/F_r)^2 \leq 1.0$.

For an anchor required to develop resistance to combined axial tension and shear through shear friction, the cross-sectional area shall be at least the sum of
(a) the area required by Clause 8.16.7.6.2 or 8.16.7.6.3; and
(b) the area required by Clause 8.16.7.6.4.

8.16.7.6.6 Combined tension and bending

An anchor required to develop resistance to combined tension and bending shall be proportioned to meet the requirements of Clause 10.19.2.4.

8.16.7.7 Additional requirements for grouted and adhesive anchors

8.16.7.7.1 General

Grouts and adhesives shall be formulated, mixed, and placed in accordance with Approved procedures established by tests.

Randomly selected grouted and adhesive anchors shall be tested in accordance with Clause 8.16.7.7.2 to a minimum of 110% of the factored load effect to verify load transfer capabilities. The tests may be waived if acceptable test and installation data are available.

Grouted and adhesive anchors installed in tensile zones of a concrete member shall be capable of sustaining the factored resistance in cracked concrete.

8.16.7.7.2 Testing of anchors

Tests shall be carried out by an Approved testing agency or by the anchor manufacturer using an Approved procedure and shall be certified by a suitably qualified person. Test reports shall present the testing program, procedures, results, and conclusions.

Tests shall be representative of the anchorage system with regard to the embedment depth, spacing, edge distance, load application, concrete type and strength, grouts or adhesives used, and expected environmental conditions. A minimum of five tests shall be carried out for each applicable combination of variables.

Tests conducted in tension shall use the embedment depth needed to attain the full capacity of the anchor. During testing of the tensile strength of an anchor, the testing device shall not apply compression to the concrete surface within a circle that is concentric with the anchor and has a diameter of four times the anchor embedment depth.

8.17 Seismic design and detailing

Seismic design and detailing shall meet the requirements of Section 4.

∆ Section 9 — Wood structures

where the frequency of butt joints is 1 in N_b, as specified in Clauses 9.21.2.2.5 and 9.22.2.2.2. For longitudinal nail-laminated wood decks other than wood-concrete composite decks, the value of k_b shall be calculated as follows:

$$k_b = 0.8(N_b - 1)/N_b$$

9.5.9 Treatment factor

The properties specified given in this Section incorporate adjustments for preservative treatment and incising. For wood treated with a fire retardant or other strength-reducing chemicals, the assumed properties shall be based on the documented results of tests that take into account the effects of time, temperature, and moisture content.

9.6 Flexure

Δ 9.6.1 Flexural resistance

The factored resistance, M_r, of glued-laminated members shall be the lesser of

$$M_r = \phi\, k_d\, k_{\ell s}\, k_m\, f_{bu}\, S$$

and

$$M_r = \phi\, k_d\, k_m\, k_{sb}\, f_{bu}\, S$$

The factored resistance, M_r, shall be calculated as follows for all other wood members:

$$M_r = \phi\, k_d\, k_{\ell s}\, k_m\, k_{sb}\, f_{bu}\, S$$

where f_{bu} is obtained from Tables 9.12 to 9.17, as applicable, and the values of k_d, $k_{\ell s}$, k_m, and k_{sb} are specified in Clauses 9.5.3, 9.6.3, 9.5.6, and 9.6.2, respectively.

Δ 9.6.2 Size effect

The value of k_{sb} for glued-laminated members shall be calculated as follows:

$$k_{sb} = 1.03\,(b \times L \times 10^{-6})^{-0.18}$$

where b is the beam width (for single-piece laminations) or the width of the widest piece (for multiple-piece laminations) and L is the length of beam segment from point of zero moment to point of zero moment.

The value of k_{sb} for sawn wood members shall be obtained from Table 9.4. The value of k_{sb} for members other than glued-laminated or sawn wood members shall be 1.0.

Table 9.4
Size-effect factors k_{sb} for flexure and k_{sv} for shear for all species and grades
(See Clauses 9.6.2, 9.7.2, and 9.22.5.2.)

	Larger dimension, mm						
Smaller dimension, mm	89	140	184	235	286	337	≥ 387
≤ 64	1.7	1.4	1.2	1.1	1.0	0.9	0.8
> 64 but < 114	1.7	1.5	1.3	1.2	1.1	1.0	0.9
≥ 114	—	1.3	1.3	1.2	1.1	1.0	0.9

9.6.3 Lateral stability

The value of $k_{\ell s}$ shall be obtained from Table 9.5, where b and d are, respectively, the width and depth of the beam or laminate, and C_s and C_k are calculated as follows:

$$C_s = \sqrt{\frac{L_u d}{b^2}}$$

$$C_k = \sqrt{\frac{E_{05}}{f_{bu}}}$$

where L_u is the laterally unsupported length of the component and f_{bu} and E_{05} are obtained from Tables 9.12 to 9.17, as applicable.

For laminated wood decks, or when the compression edge of a beam is effectively supported along its length, $k_{\ell s}$ shall be taken as 1.0.

When d/b is greater than 1.0, lateral support shall be provided at points of bearing to restrain torsional rotation.

A beam shall not have C_s greater than 30.0.

Table 9.5
Modification factor for lateral stability, $k_{\ell s}$
(See Clause 9.6.3.)

d/b	C_s	$k_{\ell s}$
≤ 1.0	—	1.0
> 1.0	≤ 10.0	1.0
> 1.0	> 10.0 but < C_k	$1 - 0.3(C_s/C_k)^4$
> 1.0	≥ C_k	$(0.70E_{05}) / (C_s^2 f_{bu})$

9.7 Shear

9.7.1 Shear resistance

The factored shear resistance, V_r, of a member of rectangular section shall be calculated as follows:

$$V_r = \phi\, k_d\, k_m\, k_{sv}\, f_{vu}\, A/1.5$$

where f_{vu} is obtained from Tables 9.12 to 9.17 and the values of k_d, k_m, and k_{sv} are as specified in Clauses 9.5.3, 9.5.6, and 9.7.2, respectively.

9.7.2 Size effect
The value of factor k_{sv} for sawn wood members shall be obtained from Table 9.4. The value of k_{sv} for glued-laminated timbers shall be $V^{-0.18}$.

9.7.3 Shear force and shear load
The factored shear resistance of a sawn member shall equal or exceed the factored shear force acting on the member (the shear effects of all loads acting within a distance from a support equal to the depth of the member need not be considered). The factored shear resistance of a glued-laminated member shall equal or exceed the factored shear load on the member, V_f, calculated as follows:

$$V_f = 0.82\left[\frac{1}{L}\int_0^L |V(x)|^5\, dx\right]^{0.2}$$

where

$|V(x)|$ = absolute value of the total factored shear force at a section at distance x along the length of the member

9.7.4 Shear modulus
The value of the shear modulus shall be 0.065 times the modulus of elasticity, E_{50}, obtained from Tables 9.14 to 9.17.

9.7.5 Vertically laminated decks
Shear shall be neglected in vertically laminated decks.

9.8 Compression members

∆ 9.8.1 General
The proportioning of compression members shall satisfy the following:

$$\left(\frac{P}{P_r}\right)^2 + \frac{M_c}{M_r} \le 1.0 \text{ (for uniaxial bending)}$$

$$\frac{P}{P_r} + \frac{M_x}{M_{xr}} + \frac{M_y}{M_{yr}} \le 1.0 \text{ (for biaxial bending)}$$

where

(a) the factored resistance in compression, P_r, is as specified in Clause 9.8.2.1;
(b) the factored resistance in flexure, M_r, is as specified in Clause 9.6.1;
(c) the factored resistances in flexure, M_{xr} and M_{yr}, for bending about the x- and y-axes, respectively, are calculated in the same manner as M_r;
(d) the amplified moment, M_c, is calculated in accordance with Clause 9.8.5.1 or 9.8.6 by taking into account the slenderness effects specified in Clause 9.8.3; and
(e) the amplified moments, M_x and M_y, acting about the x- and y- axes, respectively, are calculated in the same manner as M_c.

9.8.2 Compressive resistance parallel to grain

9.8.2.1 General
The factored compressive resistance parallel to the grain, P_r, shall be calculated as follows:

$$P_r = \phi\, k_m\, k_d\, k_{sp}\, k_c\, f_{pu}\, A$$

where k_{sp} is obtained from Clause 9.8.2.2 or 9.8.2.3, k_c is obtained from Clause 9.8.2.4, and f_{pu} is obtained from Tables 9.9 to 9.13.

9.8.2.2 Size factor for sawn wood in compression
The size factor, k_{sp}, for sawn wood in compression parallel to the grain shall be calculated as follows:

$$k_{sp} = 6.3(dL)^{-0.13} \le 1.3$$

where

d = dimension in the direction of buckling, mm

L = unsupported length associated with the member dimension, mm

9.8.2.3 Size factor for glued-laminated timber in compression
The size factor, k_{sp}, for glued-laminated timber in compression parallel to the grain shall be calculated as follows:

$$k_{sp} = 0.68V^{-0.13} \le 1.3$$

9.8.2.4 Slenderness factor
The slenderness factor, k_c, for members in compression parallel to the grain shall be calculated as follows:

$$k_c = \left[1 + \frac{k_m k_d k_{sp} f_{pu} C_c^3}{35E_{05}}\right]^{-1.0}$$

where C_c is determined in accordance with Clause 9.8.3.3 and E_{05} is obtained from Tables 9.12 to 9.17.

9.8.3 Slenderness effect

9.8.3.1 Effective length
The effective length of a compression member shall be taken as kL_u and, for members other than piles, the following requirements shall apply:

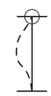
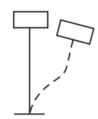
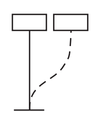
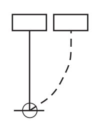
(a) the unsupported length, L_u, shall be taken as the centre-to-centre distance of lateral supports capable of sustaining a lateral restraint force of at least $0.04P$, together with any other force that is generated by the effects of end moments and lateral loading;

(b) for compression members braced against side-sway, the effective length factor, k, shall be taken as 1.0 unless rigorous analysis confirms a lower value; and

(c) for compression members not braced against side-sway, the effective length factor, k, corresponding to the end-restraint condition of the member, shall be obtained from Table 9.6 or shall be determined by rigorous analysis. For the latter case, the value of k shall not be taken as less than 1.0.

Table 9.6
Minimum values of the effective length factor, *k*

(See Clauses 9.8.3.1 and 9.8.3.2.)

End restraint	Minimum value of effective length factor, *k*
Held in position and restrained against rotation at both ends 	0.65
Held in position at both ends and restrained against rotation at one end 	0.80
Held in position but free to rotate at both ends 	1.00
Held in position and restrained against rotation at one end, and restrained against rotation, but not held in position, at the other end 	1.20
Held in position and restrained against rotation at one end, and partially restrained against rotation, but not held in position, at the other end 	1.50
Held in position at one end, but not restrained against rotation, and restrained against rotation, but not held in position, at the other end 	2.00
Held in position and restrained against rotation at one end, but not held in position or restrained against rotation at the other end 	2.00

9.8.3.2 Effective length of piles

When the finished pile projects above the ground and is not braced against buckling, the effective length shall be determined in accordance with Table 9.6 (using the value associated with the end restraint provided by the structure the pile supports) and in accordance with the following requirements:

(a) in firm ground, the lower point of contraflexure of the pile shall be taken at a depth below the ground level that is not greater than one-tenth of the exposed length of the pile;

(b) where the top stratum of the ground is soft clay or silt, the lower point of contraflexure of the pile shall be taken at a depth below the ground level that is not greater than one-half of the depth of penetration into this stratum or less than one-tenth of the exposed length of the pile; and

(c) a stratum of extremely soft soil, peat, or mud shall be treated as if it were water.

Where a pile is wholly embedded in soil, the effect of slenderness may be ignored.

Δ **9.8.3.3 Slenderness ratio**

For simple compression members of constant rectangular section, the slenderness ratio, C_c, shall not exceed 50 and shall be taken as

$$C_c = \frac{\text{effective length, } kL_u\text{, associated with width}}{\text{member width}}$$

and

$$C_c = \frac{\text{effective length, } kL_u\text{, associated with depth}}{\text{member depth}}$$

For sawn wood members, compression capacity shall be calculated separately for member width and member depth using the corresponding slenderness ratio.

For glued-laminated members, the greater slenderness ratio may be used to calculating compression capacity.

For piles and other round compression members, the slenderness ratio, C_c, shall not exceed 50 and shall be calculated as follows:

$$C_c = kL_u/0.866D_{eff}$$

9.8.4 Amplified moments

At the ultimate limit state, the effect of lateral deflection in causing or amplifying bending due to axial loads shall be considered as follows:

(a) for members not braced against side-sway, when C_c is greater than 11.6; and

(b) for members braced against side-sway, when C_c is greater than $17.3 - 5.8M_1/M_2$.

9.8.5 Rigorous evaluation of amplified moments

9.8.5.1 General

When the approximate method of Clause 9.8.6 is not adopted, the amplified moment, M_c, shall be obtained by taking account of the effect of factored axial loads in amplifying the moments due to end eccentricities, bow, and lateral loads in the unsupported length, L_u. The unsupported length shall be determined in accordance with Clause 9.8.3.1 or 9.8.3.2, the end eccentricity in accordance with Clause 9.8.5.2 or 9.8.5.3, and the bow moments in accordance with Clause 9.8.5.4.

9.8.5.2 End eccentricity

All compression members, except piles, shall be analyzed for end eccentricity at each end. The eccentricity shall be taken as the greater of

(a) the eccentricity corresponding to the maximum end moment associated with the axial load; and

(b) 0.05 times the lateral dimension of the member in the plane of the flexure being considered.

The eccentricity corresponding to Item (b) shall be assumed to cause uniaxial bending with single curvature.

9.8.5.3 End eccentricity in piles

When lateral displacement of the pile butt is prevented, the moment, M_p, shall be determined at a section 0.55 times the effective length below the butt, and shall be calculated as the product of P and e_o, plus the effects of end moments and the moments due to lateral loads. The value of e_o shall be obtained from Table 9.7.

Δ

Table 9.15
Specified strengths and moduli of elasticity for glued-laminated Douglas fir timber, MPa
(See Clauses 9.5.5, 9.6.1, 9.6.3, 9.7.1, 9.7.4, 9.8.2.1, 9.8.2.4, 9.10, and 9.12.2.)

	CSA stress grade					
Type of stress	24f-E bending grade	24f-EX bending grade	20f-E bending grade	20f-EX bending grade	16c-E compression grade	18t-E tension grade
Bending moment positive, f_{bu}	27.5	27.5	23	23	12.6	21.9
Bending moment negative, f_{bu}	12.6	27.5	12.6	23	12.6	21.9
Longitudinal shear, f_{vu}	1.4	1.4	1.4	1.4	1.4	1.4
Compression parallel to grain, f_{pu}	26.4*	26.4*	26.4*	26.4*	26.4	26.4
Compression parallel to grain combined with bending, f_{pu}	26.4*	26.4	26.4*	26.4	26.4	26.4
Compression perpendicular to grain, f_{qu}	5.8	5.8	5.8	5.8	5.8	5.8
Axial tension at gross section, f_{tg}	13.4*	13.4	13.4*	13.4	13.4	15.7
Axial tension at net section, f_{tn}	17.9*	17.9	17.9*	17.9	17.9	20.1
Modulus of elasticity						
E_{50}	12 100	12 100	11 800	11 800	11 800	13 100
E_{05}	10 600	10 600	10 200	10 200	10 200	11 400

**The use of this stress grade for this primary application is not recommended.*

Notes:

(1) *Designers should check the availability of grades before specifying.*

(2) *The values in this Table are based on the following standard conditions:*
- *(a) semi-wet service conditions; and*
- *(b) standard term duration of load.*

9.12.3 Vertically laminated beams

The factored resistance in flexure for beams composed of vertical laminations shall be calculated as for load-sharing systems in sawn wood.

9.12.4 Camber

Glued-laminated beams shall be cambered by the sum of 1/600 of the span plus twice the calculated deflection due to the unfactored dead loads.

9.12.5 Varying depth

When there is a variation in the depth of a flexural member, the bevel of the laminates on the tension side shall not be steeper than 7% and the factored fibre stress shall not be less than 50% of the specified strength.

9.12.6 Curved members
The requirements of this Section shall apply only to glued-laminated members with a radius greater than 12 m. In such members the reduction in capacity due to curvature may be ignored.

9.13 Structural composite lumber

9.13.1 Materials
Structural composite lumber shall be laminated veneer lumber or parallel strand lumber manufactured from Douglas fir.

9.13.2 Specified strengths and moduli of elasticity
The specified strengths and moduli of elasticity shall be obtained from ASTM D 5456, as modified by the procedures specified in CAN/CSA-O86. Typical values for some representative products are specified in Table 9.16.

Table 9.16
Typical specified strengths and moduli of elasticity for structural composite lumber, MPa
(See Clauses 9.5.5, 9.6.1, 9.6.3, 9.7.1, 9.7.4, 9.8.2.1, 9.8.2.4, and 9.10.)

	Laminated veneer lumber	Parallel strand lumber
Type of stress		
Bending at extreme fibre, f_{bu}	32.1	33.2
Longitudinal shear — Parallel, f_{vu}*	3.3	3.3
Longitudinal shear — Perpendicular, f_{vu}*	2.0	2.4
Compression parallel to grain, f_{pu}	31.2	33.2
Compression perpendicular to grain — Parallel, f_{qu}*	8.6	8.6
Compression perpendicular to grain — Perpendicular, f_{qu}*	5.5	5.5
Axial tension parallel to grain, f_{tu}	20.0	27.5
Modulus of elasticity		
E_{50}	13 000	13 000
E_{05}	11 300	11 300

**To glueline for laminated veneer lumber and to wide face of strand for parallel strand lumber.*
Note: *These values are provided for illustrative purposes; the design values shall be obtained after verification of the structural properties and adjustment factors of the proprietary products.*

9.14 Wood piles

9.14.1 Materials
Wood pile materials shall comply with CSA CAN3-O56.

9.14.2 Splicing
Splicing of wood piles shall require Approval.

9.14.3 Specified strengths and moduli of elasticity
The specified strengths and moduli of elasticity for round wood piles shall be obtained from Table 9.17.

Δ Section 10 — Steel structures

Section 10
Steel structures

10.1 Scope

This Section specifies requirements for the design of structural steel bridges, including requirements for structural steel components, welds, bolts, and other fasteners required in fabrication and erection. Requirements related to the repeated application of loads and to fracture control and fracture toughness for primary tension and fracture-critical members are also specified.

10.2 Definitions

The following definitions apply in this Section:

Brittle fracture — a type of fracture in structural materials without prior plastic deformation that usually occurs suddenly.

Buckling load — the load at which a member or element reaches a condition of instability.

Camber — the built-in deviation of a bridge member from straight, when viewed in elevation.

Class — a designation of structural sections with regard to the width-to-thickness ratios of their constituent elements and their flexural-compressive behaviour.

Coating — an Approved protective system for steel, e.g., galvanizing, metallizing, a paint system, or coal tar epoxy.

Composite beam or girder — a steel beam or girder structurally connected to a concrete slab so that the beam and slab respond to loads as a unit.

Composite column — a column consisting of a steel tube filled with concrete, with or without internal reinforcement.

Critical net area — the area with the least tensile or tensile-shear resistance.

Element — a flat plate or plate-like component of a structural member.

Erection diagrams — drawings that show the layout and dimensions of a steel structure and from which shop details are made. They also correlate the fabricator's piece marks with locations on the structure.

Fatigue — initiation of microscopic cracks and propagation of such cracks into macroscopic cracks caused by the repeated application of load.

Fatigue limit — the level of stress range below which no fatigue crack growth is assumed to occur.

Fixed joint — a joint that allows rotation but not translation.

Flush — weld reinforcement not exceeding 1 mm in height that has a smooth, gradual transition with the surrounding plate (and involving grinding where necessary).

Fracture-critical membersmembers or portions of members, including attachments, in a single load path structure that are subject to tensile stress and the failure of which can lead to collapse of the structure.

Gauge — the distance between successive holes, measured at right angles to the direction of the force in the member.

Lateral torsional buckling — the buckling of a member involving lateral deflection and twisting.

Local buckling — the buckling of a plate element (as distinct from the buckling of the member as a whole).

Matching electrode — an electrode with an ultimate strength closest to and greater than the minimum specified ultimate strength of the base metal.

Notch toughness — the ability of steel to absorb tensile strain energy in the presence of a notch.

Post-buckling resistance — the ability of plate elements to resist additional load after initial elastic buckling.

Primary tension members — members or portions of members, including attachments (but not fracture-critical members or secondary components) that are subject to tensile stress.

Proposal — a constructor's submission of changes, when engineering design is required, that affects either the original design or the method of construction or shipping of a structure.

Prying action — an additional force introduced into fasteners as a result of deformation of the parts that they connect.

Single load path structure — a structure in which failure of a single structural component could lead to a total collapse.

Slenderness ratio — the effective length of a member divided by the radius of gyration, both with respect to the same axis.

Slip-critical connection — a connection where slippage cannot be tolerated, including connections subject to fatigue or to frequent load reversal or where the resulting deflections are unacceptable.

Smooth — a profile of weld reinforcement where any uneven surface has been ground away and the remaining metal profile merges gradually with the surrounding plate. In order to be regarded as smooth, weld reinforcements that remain after grinding are limited to 2 mm for plate thicknesses of 50 mm and less and 3 mm for plate thicknesses greater than 50 mm.

Snug-tight — the tightness of a bolt that is attained after a few impacts of an impact wrench or the full effort of a person using a spud wrench.

Stress range — the algebraic difference between the maximum and the minimum stresses caused by fatigue loading, where tensile stress has the opposite sign to compressive stress.

Stress range category — a category that establishes the level of stress range permitted in accordance with the classification of the detail and the number of the design stress cycles.

Tension-field action — the truss-like behaviour of a plate girder panel under shear force that develops after shear-buckling of the web and is characterized by diagonal tensile forces in the web and compressive forces in the transverse stiffeners.

Web crippling — the local failure of a web plate in the immediate vicinity of a concentrated load or reaction.

10.3 Abbreviations and symbols

10.3.1 Abbreviations

The following abbreviations apply in this Section:

CJP — complete joint penetration
FLS — fatigue limit state
PJP — partial joint penetration
SLS — serviceability limit state
ULS — ultimate limit state

10.3.2 Symbols

The following symbols apply in this Section:

A = area, mm^2

A' = area enclosed by the median line of the wall of a closed section, mm^2

A_b = cross-sectional area of a bolt, based on nominal diameter, mm^2

A_c = area of concrete in a tube pile, mm^2; transverse area of concrete between the longitudinal shear planes that define A_{cv}, mm^2

A_{ce} = area of concrete in compression in a composite column, mm^2

A_{cf} = area of compression flange of a steel section, mm^2

A_{cv} = critical area of longitudinal shear planes in the concrete slab, one on each side of the steel compression flange, extending from the point of zero moment to the point of maximum moment, mm^2

A_{de} = effective cross-sectional area of the deck, including longitudinal ribs, mm^2

A_f = area of bottom flange of box girders, including longitudinal stiffeners, mm^2; area of flanges of plate girder, mm^2

A_g = gross area, mm^2

Δ A_{gv} = gross shear area for block shear failure (see Clause 10.8.1.3.2.4), mm^2

A_m = area of fusion face, mm^2

A_n = critical net area, mm^2; total net area of a member tributary to the particular lap splice, including elements not directly connected, mm^2; tensile stress area, mm^2

Δ A_{ne} = effective net area (reduced for shear lag), mm^2

Δ A_{n1}, A_{n2}, A_{n3} = net areas of the connected plate elements subject to load transfer by a transverse weld, two longitudinal welds, or a single longitudinal weld, respectively, mm^2

A_r = area of reinforcing steel within the effective width of a concrete slab, mm^2

A_{rL} = area of longitudinal reinforcement within the concrete area A_c, mm^2

A_{rt} = area of transverse reinforcement crossing the longitudinal shear planes of A_{cv}, mm^2

A_s = area of steel section, mm^2; area of stiffener or pair of stiffeners, mm^2; tensile stress area of bolt, mm^2

A_{sc} = area of shear connector, mm^2

A'_{sc} = area of steel section in compression (see Clause 10.11.6.2.2), mm^2

A_{st} = tensile stress area, mm^2

A'_{st} = area of steel section in tension (see Clause 10.11.6.2.2), mm^2

A_w = web area or shear area, mm^2; size of effective throat area of weld, mm^2

Δ $ADTT$ = average daily truck traffic

$ADTT_f$ = single lane average daily truck traffic for fatigue

a = spacing of transverse stiffeners, mm; depth of compression block in a concrete slab, mm; transverse distance between centroids of groups of fasteners or welds connecting the batten to each main component, mm; clear distance between webs of a trough at deck level, mm

a' = the larger of e, the clear distance between stiffener troughs at deck level, and a, the clear distance between webs of a trough at deck level, mm

B = ratio in interaction equation for composite columns

B_r = factored bearing resistance of a member or component, N

B_1, B_2 = geometric coefficients for a laterally unsupported monosymmetric I-beam

b = half of width of flange of I-sections and T-sections, mm; full width of flange of channels, Z-sections, and stems of tees, mm; distance from free edge of plates to the first line of bolts or welds, mm; width of stiffener, mm; width of bottom flange plate between webs of box girder, mm

b_c = width of concrete at the neutral axis, mm (see Clause 10.9.5.5)

b_e = effective width of concrete slab, mm

b_f = width of widest flange of curved welded I-girders, mm

b_s = width of compression flange between longitudinal stiffeners, mm; distance from web to nearest longitudinal stiffener, mm

C = coefficient in formula for area of stiffener; coefficient in formula for moment resistance of unstiffened compression flanges of composite box girders

Δ C_L = correction factor for fatigue truck weight

C_c = factored compressive resistance of concrete, N

C_e = Euler buckling load, N

C_{ec} = Euler buckling load of a concrete-filled hollow structural section, N

C_f = factored compressive force in a member or component at ULS, N

C_r = factored compressive resistance of a member or component, N; factored compressive resistance of steel acting at the centroid of the steel area in compression, N; factored compressive resistance of reinforcing steel, N

C_r' = factored compressive resistance of concrete area, A_c, of a column, N

C_{rc} = factored compressive resistance of a composite column, N

C_{rcm} = factored compressive resistance of composite column that can coexist with M_{rc} when all of the section is in compression, N

C_{rco} = factored compressive resistance of composite column of zero slenderness ratio, N

C_{rx} = factored compressive resistance of a member or component about the major axis, N

C_s = factored compressive force in steel of composite beam when the plastic neutral axis is in the steel section, N; coefficient in equation for moment resistance of stiffened compression flanges of composite box girders

C_w = warping torsional constant, mm^6

C_y = axial compressive force at yield stress, N

C_1, C_2 = limiting values of compressive resistance of slab, N

c_1 = coefficient related to the slip resistance of a bolted joint

D = stiffener factor; outside diameter of circular section, mm; diameter of rocker or roller, mm; weld leg size, mm

d = depth, mm; depth of beam or girder, mm; diameter of bolt or stud shear connector, mm; longitudinal distance centre-to-centre of battens, mm

d_c = depth of compression portion of web in flexure, mm

d_s' = distance from extreme compression fibre to centroid of reinforcing steel, mm

E_c = modulus of elasticity of concrete, MPa

E_s = modulus of elasticity of steel, MPa

e = edge distance, mm; lever arm between the factored compressive resistance, C_r, and the factored tensile resistance, T_r, of the steel, mm; clear distance between stiffener troughs at deck level, mm

e' = lever arm between the factored compressive resistance, C_r', and the factored tensile resistance, T_r, of the steel, mm

e_c = lever arm between the factored tensile resistance and the factored compressive resistance of the concrete, mm

e_r = lever arm between the factored tensile resistance and the factored compressive resistance of the reinforcing steel, mm

e_s = lever arm between the tensile resistance and the compressive resistance of the steel, mm

F_{cr} = shear buckling stress, MPa; buckling stress of plate in compression, MPa; lateral torsional buckling stress, MPa

F_e = critical torsional or flexural torsional elastic buckling stress, MPa

F_{ex} = elastic flexural buckling stress about the major axis, MPa

F_{ey} = elastic flexural buckling stress about the minor axis, MPa

F_{ez} = elastic torsional buckling stress, MPa

Δ F_m = average of the tensile yield and ultimate strengths, MPa

F_s = ULS shear stress, MPa

F_{sr} = fatigue stress range resistance, MPa

F_{srt} = constant amplitude threshold stress range, MPa

F_{st} = factored force in stiffener at ULS, N

F_t = tension field component of post-buckling stress, MPa

F_u = specified minimum tensile strength, MPa

F_y = specified minimum yield stress, yield point, or yield strength, MPa

F_{yc} = yield strength of a column, MPa

f_b = calculated bending stress, MPa

f_c' = specified compressive strength of concrete, MPa

Δ f_{cr} = cracking strength of concrete, MPa

f_g = axial global tensile stress in a deck induced by flexure and axial tension in the main longitudinal girders, MPa

f_s = coexisting shear stress due to warping torsion, MPa

f_{sr} = calculated FLS stress range at the detail due to passage of the CL-W Truck or of a tandem set of axles, MPa

f_{vg} = the simultaneous global shear stress in the deck, MPa

f_w = warping normal stress, MPa

f_y = specified minimum yield strength of reinforcing steel, MPa

G_s = shear modulus of elasticity of structural steel, MPa

g = transverse spacing between fastener gauge lines, mm; distance from heel of connection angle to first gauge line of bolts in outstanding legs, mm

H = coefficient for flexural torsional buckling

h = clear depth of web between flanges, mm; width of rectangular hollow section, mm; height of shear connector, mm; height of stiffener, mm; height of trough, mm

h' = length of inclined portion of a rib web, mm

h_c = clear depth of column web, mm

h_n = variable used to calculate M_{rc} of a circular hollow structural section

h_p = depth of subpanel of a girder, mm

I = moment of inertia, mm^4

I_s = moment of inertia of longitudinal compression flange stiffener, mm^4

I_t = moment of inertia of transverse compression flange stiffener, mm^4; moment of inertia of transformed section, mm^4

I_x = major axis moment of inertia, mm^4

I_y = minor axis moment of inertia of the whole cross-section, mm^4

I_{y1}, I_{y2} = moment of inertia of upper and lower flanges, respectively, about the *y*-axis of symmetry, mm^4

J = St. Venant torsional constant, mm^4

j = coefficient used in determining moment of inertia of stiffeners

K = effective length factor

K_x, K_y, K_z = effective length factor with respect to *x*-, *y*-, or *z*-axis

k = distance from outer face of flange to toe of flange-to-web fillet, mm

k_s = coefficient related to the slip resistance of a bolted joint; plate buckling coefficient

k_v = shear buckling coefficient

k_1, k_2 = buckling coefficients

L = length, mm; span length between simple connections at girder ends, mm; connection length in direction of loading, equal to the distance between the first and last bolts in bolted connections and to the overall length of the weld pattern in welded connections, mm; laterally unsupported distance from one braced location to an adjacent braced location, mm; length of roller or rocker, mm; length of cut-out in a closed cross-section member measured parallel to the longitudinal axis of the member, mm; length of a compression flange between points of lateral restraint, mm

L_c = length of channel shear connector, mm

L_n = length of segment parallel to the force, mm

ℓ = length in which warping restraint is developed, mm

M_L = bending moment in beam or girder at SLS due to live load, N•mm

Δ M_a = factored bending moment at one-quarter point of unbraced segment, N•mm

Δ M_b = factored bending moment at midpoint of unbraced segment, N•mm

Δ M_c = factored bending moment at three-quarter point of unbraced segment, N•mm

M_d = bending moment in beam or girder at SLS due to dead load, N•mm

M_f = factored bending moment in member or component at ULS, N•mm

M_{fb} = factored bending moment in transverse beam at ULS, N•mm

M_{fd} = factored bending moment in beam or girder at ULS due to dead load, N•mm

M_{fl} = factored bending moment in beam or girder at ULS due to live load, N•mm

M_{fr} = factored bending moment in longitudinal rib at ULS, N•mm

M_{fsd} = factored bending moment in beam or girder at ULS due to superimposed dead load, N•mm

M_{fw} = factored bending moment in plane of a girder flange due to torsional warping, N•mm

M_{fx} = factored bending moment in member or component about the *x*-axis of the cross-section at ULS, N•mm

M_{fy} = factored bending moment in member or component about the *y*-axis of the cross-section at ULS, N•mm

M_{f1} = smaller factored end moment of beam-column at ULS, N•mm

M_{f2} = larger factored end moment of beam-column at ULS, N•mm

M_p = plastic moment resistance (= ZF_y), N•mm

M_r = factored moment resistance of member or component, N•mm

M_{rb} = factored moment resistance of transverse beam, N•mm

M_{rc} = factored moment resistance of composite column, N•mm

M_{rr} = factored moment resistance of longitudinal rib, N•mm

M_{rx} = factored moment resistance of member or component about the *x*-axis of the cross-section, N•mm

M'_{rx} = reduced factored moment resistance of curved non-composite I-girder, N•mm

M_{ry} = factored moment resistance of member or component about the *y*-axis of the cross-section, N•mm

M_{sd} = bending moment in beam or girder at SLS due to superimposed dead load, N•mm

M_u = critical elastic moment of a laterally unbraced beam, N•mm

M_y = yield moment, N•mm

m = number of faying surfaces or shear planes in a bolted joint (equal to one for bolts in single shear and two for bolts in double shear)

N = length of bearing of an applied load, mm; number of shear connectors

N_a = number of additional shear connectors per beam at point of contraflexure

N_c = specified number of design stress cycles

N_d = number of design stress cycles experienced for each passage of the design truck (see Table 10.5)

n = number of equally spaced longitudinal stiffeners in box girders; number of parallel planes of battens; number of bolts; modular ratio, E_s/E_c; number of studs arranged transversely across a flange at a given location; coefficient for axial buckling resistance

P = factored force to be transferred by shear connectors, N

p = pitch of threads, mm; pitch between bolts, mm; reduction factor for multi-lane fatigue loading

Q = moment of area, about the neutral axis of the composite section, of the transformed compressive concrete area in positive moment regions or in negative moment regions that are prestressed, mm^3; for non-prestressed sections in negative moment regions, moment of the transformed area of reinforcement embedded in the concrete, mm^3

Q_f = factored torsional moment in a member at ULS, N•mm

Q_r = factored torsional resistance, N•mm

q_r = factored shear resistance of shear connectors, N

q_{sr} = range of interface shear, N

Δ R = radius of curvature of girder web, mm (see Clause 10.13.6.1); horizontal radius of curvature, mm (see Clause 10.7.4.3); transition radius as shown in Example 12 of Figure 10.6

R_s = vertical force for proportioning connection of transverse stiffener to longitudinal stiffener in box girders, at ULS, N

R_v = reduced normal stress factor, taking coexisting warping shear stresses into account

Δ R_w = vertical force for proportioning connection of transverse stiffener to web in box girders, at ULS, N (see Clause 10.18.3.2.2); strength reduction factor for multiple orientation fillet welds

R_1, R_2 = non-dimensional width-to-thickness demarcation ratios between yielding, inelastic buckling, and elastic buckling of compression flange; radius of roller or rocker and of groove of supporting plate, respectively, mm

r = radius of gyration, mm

r_c = radius of gyration of the concrete area, mm

r_x = radius of gyration of a member about its strong axis, mm

r_y = radius of gyration of a member about its weak axis, mm

r_0 = centroidal radius of gyration (see Clause 10.9.3.2), mm

S = elastic section modulus of steel section, mm^3; short-term load, N

S' = elastic modulus of composite section comprising the steel section, reinforcement, and prestressing steel within the effective width of the slab with respect to the flange or reinforcing steel under consideration, mm^3

S_e = effective section modulus, mm^3

S_h = elastic modulus of longitudinal stiffener with respect to the base of the stiffener, mm^3

S_n, S_{3n} = elastic modulus of section comprising the steel beam or girder and the concrete slab, calculated using a modular ratio of n or $3n$, respectively, mm^3

S_t = section modulus of transverse stiffener, mm^3

s = centre-to-centre spacing between successive fastener holes in the line of load, mm; centre-to-centre spacing of each group of shear studs, mm

T = tension in bolt at SLS, N; total load on column, N

T_f = factored tensile force in member or component at ULS, N

T_r = factored tensile resistance of a steel section, member, or component, of reinforcing steel, or of the effective width of a deck, including the longitudinal ribs, N

T_s = factored tensile resistance of steel section or component, N; minimum service temperature, °C

T_t = Charpy V-notch test temperature, °C

T_u = specified minimum tensile resistance, taken as follows:

(a) for parallel wire strands, the product of the sum of the areas of the individual wires and the specified minimum tensile strength of the wires, N; and

(b) for helical strands and wire ropes, the specified minimum tensile resistance established by test, taking into account the actual configuration such as socketing and bending over cable bands, N

t = thickness, mm; average thickness of channel shear connector flange, mm; thickness of flange, mm; thickness of end connection angles, mm; thickness of stiffener, mm

t_b = thickness of beam flange, mm; thickness of bottom flange, mm

t_c = thickness of concrete slab, mm; thickness of column flange, mm

t_{de} = effective thickness of deck plate, taking into account the stiffening effect of the surfacing, mm

t_r = thickness of rib, mm

t_t = thickness of top flange, mm

U = factor to account for moment gradient and for second-order effects of axial force acting on the deformed member

Δ U_t = efficiency factor

V = shear in a bolt or bolts at SLS, N

V_H = horizontal shear between troughs in orthotropic deck bridge due to shear, V_{LL+I}, due to live load and impact, as specified in Table 10.8, N

V_{LL+I} = shear due to live load and impact, N

V_f = factored shear force at ULS, N

V_r = factored shear resistance of member or component, N; shear range, N

V_s = slip resistance at SLS, N
V_{sr} = range of shear force at FLS resulting from passage of CL-W Truck, N
V_u = longitudinal shear in concrete slab of a composite beam, N
W = load level in CL-W, kN
w = web thickness, mm; thickness of channel shear connector web, mm; width of plate, mm
w_c = thickness of column web, mm
w_n = length of a segment, normal to a force, mm
X = curvature correction factor for transverse stiffener requirements
X_u = ultimate strength of weld metal, as rated by electrode classification number, MPa
x = subscript relating to the strong axis of a member
$\bar{x}$ = distance perpendicular to axis of member from the fastener plane to the centroid of the portion of the area of the cross-section under consideration, mm
x_0 = x-coordinate of shear centre with respect to centroid, mm
Y = ratio of specified minimum yield point of web steel to specified minimum yield point of stiffener steel
y = subscript relating to the weak axis of a member; design life, years
y_b = distance from centroid of a steel section to bottom fibre of a steel beam or girder, mm
y_b' = distance from centroid of the lower portion of a steel section under tension or compression to bottom fibre of a beam or girder, mm
y_{bc} = distance from plastic neutral axis of a composite section to bottom fibre of a steel beam or girder, mm
Δ y_c = maximum distance from the neutral axis to the extreme outer fibre of the composite section (if applicable), mm
Δ y_0 = y-coordinate of shear centre with respect to centroid, mm
Δ y_s = distance from the neutral axis to the extreme outer fibre of the steel section (maximum distance for non-symmetrical sections), mm
y_t = distance from centroid of a steel section to top fibre of a steel beam or girder, mm
y_t' = distance from centroid of the upper portion of a steel section under tension or compression to top fibre of a steel beam or girder, mm
y_{tc} = distance from plastic neutral axis of a composite section to top fibre of a steel beam or girder, mm
Z = plastic section modulus of a steel section, mm^3; curvature parameter
Z_{sr} = allowable range of interface shear in an individual shear connector, N
β = value derived from a recursive equation, radians (see Clause 10.9.5.5)
β_x = coefficient of monosymmetry
γ = fatigue life constant
Δ Δ = specified camber at any section, mm
Δ_{DL} = camber at any point along the length of a span
Δ Δ_{SDL} = camber at any point along the length L, calculated to compensate for deflection due to dead loads on the composite section (if applicable), mm
Δ Δ_m = maximum value of $\Delta_{DL} + \Delta_{SDL}$ within length L, mm
Δ_r = additional camber for horizontally heat-curved beams, mm
θ = angle of inclination of web plate of box girders to the vertical, degrees; angle of weld axis to line of action of force, degrees
κ = ratio of the smaller factored moment to the larger factored moment at opposite ends of an unbraced length (positive for double curvature and negative for single curvature)

λ = non-dimensional slenderness parameter in column formula; slenderness parameter

λ_c = slenderness parameter for concrete portion of composite column

λ_e = equivalent slenderness parameter

ρ = factor modifying contribution of steel to compressive resistance of composite column

ρ_b, ρ_w = curvature correction factors

τ, τ' = factors modifying contributions of steel and concrete, respectively, to compressive resistance of composite column

ϕ_b = resistance factor for bolts

ϕ_{be} = resistance factor for beam web bearing, end

ϕ_{bi} = resistance factor for beam web bearing, interior

ϕ_{br} = resistance factor for load bearing in bolted connections

ϕ_c = resistance factor for concrete

ϕ_r = resistance factor for reinforcement

ϕ_s = resistance factor for steel

ϕ_{sc} = resistance factor for shear connectors

ϕ_{tc} = resistance factor for steel cables in tension

ϕ_w = resistance factor for welds

ψ = ratio of total cross-sectional area to that of both flanges

ω_1 = coefficient used to determine equivalent uniform bending effect in beam-columns

ω_2 = coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient

10.4 Materials

10.4.1 General

Clauses 10.4.2 to 10.4.7 shall apply unless deviations from their requirements are Approved.

The fracture toughness of steel shall meet the requirements of Clause 10.23.3.

Plates provided from coils shall be used only if it can be demonstrated that the levelling process used in manufacturing produces plate with longitudinal residual stresses that are balanced about mid-thickness. In addition, after levelling, plates shall conform to the flatness tolerances specified in CSA G40.20, and the elongation and impact properties, after testing in accordance with CSA G40.20, shall be to the satisfaction of the Engineer.

Δ 10.4.2 Structural steel

Structural steel shall conform to CSA G40.21, except as provided below. The modulus of elasticity of structural steel, E_s, shall be taken as 200 000 MPa and the shear modulus of elasticity of structural steel, G_s, shall be taken as 77 000 MPa.

Weathering steel members shall be of Type A atmospheric corrosion-resistant steel as specified in CSA G40.21.

Fracture-critical members and primary tension members shall be of Type AT, Type WT, or Type QT steel as specified in CSA G40.21.

ASTM A 588/A 588M steel may be substituted for CSA G40.21 Grade 350A steel. ASTM A 588/A 588M steel may be substituted for CSA G40.21 Grade 350AT steel when the Charpy impact energy requirements are verified by the submission of test documentation.

10.4.3 Cast steel

Cast steel shall comply with ASTM A 27/A 27M, ASTM A 148/A 148M, or ASTM A 486/A 486M.

10.4.4 Stainless steel
Stainless steel shall comply with ASTM A 167.

Δ 10.4.5 Bolts
Bolts shall comply with ASTM A 325, ASTM A 325M, ASTM A 490, ASTM A 490M, ASTM F 1852, or ASTM F 2280. Bolts less than M16 or 5/8 inch in diameter shall not be used in structural applications.

ASTM A 325/A 325M or ASTM A 490/A 490M high-strength bolts for use with uncoated corrosion-resistant steel shall be Type 3 unless corrosion protection is provided by an approved protection system. ASTM A 490 and A 490M bolts shall not be galvanized or plated.

Δ 10.4.6 Welding electrodes
Except as permitted by Clause 10.23.4.5, welding electrodes, electrode/gas, or electrode/flux combinations shall be low hydrogen (i.e., a level of H16 or less) and shall comply with CSA W47.1, CAN/CSA-W48, and CSA W59.

Δ 10.4.7 Stud shear connectors
Material requirements for stud shear connectors and the qualification of the shear connector base shall comply with CSA W59, Appendix H. Only studs of Type B shall be used.

10.4.8 Cables

10.4.8.1 Bright wire
Bright wire shall comply with ASTM A 510.

10.4.8.2 Galvanized wire
Galvanized wire shall comply with ASTM A 641/A 641M.

10.4.8.3 Bridge strand and wire rope
Bridge strand shall comply with ASTM A 586. Wire rope shall comply with ASTM A 603.

Δ 10.4.9 High-strength bars
High-strength bars shall be used only with the approval of the Regulatory Authority and shall comply with ASTM A 722/A 722M. The Engineer shall specify specific notch toughness requirements based on the intended use.

10.4.10 Galvanizing and metallizing
Galvanizing shall comply with CAN/CSA-G164 and CSA G189. Zinc metallizing shall comply with CSA G189.

10.4.11 Identification

10.4.11.1 Identified steels
The specifications of the materials and products used, including type or grade if applicable, shall be identified by
(a) mill test certificates or manufacturer's certificates satisfactorily correlated to the materials or products to which they pertain; or
(b) legible markings on the material or product made by the manufacturer in accordance with the applicable material or product standard.

Otherwise, Clause 10.4.11.2 shall apply.

∆ 10.4.11.2 Unidentified steels
Structural steels not identified as specified in Clause 10.4.11.1 shall not be used unless tested by an Approved testing laboratory in accordance with CSA G40.20/G40.21. The results of such testing, taking into account both mechanical and chemical properties, shall form the basis for classifying the steels as to specification. Once classified, the specified minimum values for steel at the applicable specification grade shall be used for design.

10.4.12 Coefficient of thermal expansion
The coefficient of linear thermal expansion for steel shall be taken as 12×10^{-6}/°C.

∆ 10.4.13 Pins and rollers
Pins and rollers greater than 175 mm in diameter shall be forged and annealed or forged and normalized. Pins and rollers 175 mm or less in diameter shall be forged and annealed, forged and normalized, or of cold-finished carbon-steel shafting.

10.5 Design theory and assumptions

10.5.1 General
Structural members and components shall be proportioned to satisfy the requirements for the ultimate, serviceability, and fatigue limit states.

10.5.2 Ultimate limit states
The factored resistances specified in this Section shall be equal to or greater than the effect of factored loads specified in Section 3 for all relevant ULS considerations, including strength, rupture, bending, buckling, lateral-torsional bucking, sliding, overturning, and uplift.

10.5.3 Serviceability limit states

10.5.3.1 General
The SLS considerations shall be those of deflection, yielding, slipping of bolted joints, and vibration.

∆ 10.5.3.2 Deflection
The requirements of Clause 10.16.4 and Section 3 shall apply.

10.5.3.3 Yielding
Members of all classes of sections shall be proportioned so that general yielding does not occur. Localized limited yielding shall be permitted.

10.5.3.4 Slipping of bolted joints
The requirements of Clause 10.18 shall apply.

10.5.3.5 Vibration
The requirements of Section 3 shall apply.

∆ 10.5.3.6 Transportability
The Engineer shall consider transportability for components of unusual geometry, weight, or dimensions.

10.5.4 Fatigue limit state
The requirements of Clause 10.17 shall apply.

10.5.5 Fracture control
The requirements of Clause 10.23 shall apply.

10.5.6 Seismic requirements
The requirements of Clause 4.8 shall apply.

Δ 10.5.7 Resistance factors
Resistance factors shall be taken as follows:
(a) flexure: $\phi_s = 0.95$;
(b) shear: $\phi_s = 0.95$;
(c) compression: $\phi_s = 0.90$;
(d) tension: $\phi_s = 0.95$;
(e) torsion: $\phi_s = 0.90$;
(f) tension in cables: $\phi_{tc} = 0.55$;
(g) reinforcing steel in composite construction: $\phi_r = 0.90$;
(h) concrete in composite construction: ϕ_c as specified in Section 8;
(i) bolts: $\phi_b = 0.80$;
(j) load bearing in bolted connections: $\phi_{br} = 0.80$;
(k) welds: $\phi_w = 0.67$;
(l) shear connectors: $\phi_{sc} = 0.85$;
(m) beam web bearing, interior: $\phi_{bi} = 0.80$; and
(n) beam web bearing, end: $\phi_{be} = 0.75$.

10.5.8 Analysis
Unless other methods are Approved, the methods of analysis used shall be as specified in this Section and Section 5. The design of supporting members shall provide for the effect of any significant moment or eccentricity arising from the manner in which a beam, girder, or truss is connected or supported.

10.5.9 Design lengths of members

10.5.9.1 Span lengths
Span lengths shall be taken as the distance between centres of bearings or other points of support.

10.5.9.2 Compression members

10.5.9.2.1 General
The design of a compression member shall be based on its effective length, KL.

The unbraced length, L, shall be taken as the length of the compression member measured centre-to-centre of restraints.

The unbraced length may differ for different cross-sectional axes of a member. For the bottom level of a multi-level bent or for a single-level bent, L shall be measured from the top of the base plate.

The effective length factor, K, shall be as specified in Clauses 10.5.9.2.2, 10.5.9.2.3, or 10.5.9.2.4, depending on the potential failure modes and whether failure is by buckling or in-plane bending.

10.5.9.2.2 Failure modes involving in-plane bending
The effective length shall be taken as the actual unbraced length, i.e., $K = 1.0$, for beam-columns that would fail by in-plane bending, but only if, when applicable, the sway effects have been included in the analysis of the structure to determine the end moments and forces acting on the beam-columns.

10.5.9.2.3 Failure modes involving buckling
The effective length for axially loaded columns that would fail by buckling and for beam-columns that would fail by out-of-plane lateral torsional buckling shall be based on the rotational and translational restraint afforded at the ends of the unbraced length.

∆ 10.5.9.2.4 Compression members in trusses
The effective length for members that would fail by in-plane bending shall be taken as the actual unbraced length, i.e., $K = 1.0$. The effective length for members that would fail by buckling shall be based on the rotational and translational restraint afforded at the ends of the unbraced length. For half-through or pony-truss spans, the buckling load of the compression chord shall be determined in accordance with Clause 10.14.3.6.

10.6 Durability

10.6.1 General
The requirements of Clauses 10.6.2 to 10.6.7 shall apply unless superseded by the requirements of the Regulatory Authority.

10.6.2 Corrosion as a deterioration mechanism
The deterioration mechanisms considered for steel components shall include corrosion.

10.6.3 Corrosion protection
Corrosion protection shall be provided by alloying elements in the steel, protective coatings, or other Approved means. The type and degree of corrosion protection to be provided shall be shown on the Plans.

10.6.4 Superstructure components

10.6.4.1 General
The minimum corrosion protection shall be as specified in Table 10.1 for the applicable superstructure component and environmental exposure condition.

∆ 10.6.4.2 Structural steel
Structural steel, including diaphragms and bracing but excluding surfaces in contact with concrete and the contact surfaces of bolted joints, shall be coated with an Approved coating system for a minimum distance of 3000 mm from the ends of girders at expansion joints.

Surfaces of girders that are subject to water runoff from the deck shall either be coated with an Approved coating system or the cross-sections shall be increased to account for the estimated loss of section over the design life of the structure.

10.6.4.3 Cables, ropes, and strands
All wires in the cables of suspension bridges and the stay cables of cable-stayed bridges shall be hot-dip galvanized. Suspension bridge and arch bridge hangers and other ropes or strands shall be hot-dip galvanized.

The completed main cables of suspension bridges shall also be treated with zinc dust paste and wrapped with soft-annealed galvanized wire.

The stay cables of cable-stayed bridges shall also be encased in a tube or sheath filled with an Approved grease or wax.

10.6.4.4 High-strength bars
When not sheathed and grouted, high-strength bars shall be hot-dip galvanized.

10.6.4.5 Steel decks
In marine environments and in areas where roadways are likely to be salted for winter maintenance, steel decks, except for open grid decks, shall be waterproofed and provided with a skid-resistant wearing surface.

10.6.5 Other components
The minimum protective measures for steel components not covered by Clauses 10.6.4.2 to 10.6.4.5 other than superstructure components shall be as specified in Table 10.2 for the applicable environmental exposure condition.

Stainless steel inserts in submerged members shall be electrically connected to the reinforcement.

Table 10.1
Corrosion protection for superstructure components
(See Clause 10.6.4.1.)

	Environmental exposure condition									
	No direct chlorides			**Air-borne chlorides or light industrial atmosphere**			**Heavy industrial atmosphere**			
Component	**Wet, rarely dry**	**Dry, rarely wet**	**Cyclical wet/dry**	**Wet, rarely dry**	**Dry, rarely wet**	**Cyclical wet/dry**	**Wet, rarely dry**	**Dry, rarely wet**	**Cyclical wet/dry**	**Marine**
All superstructures (minimum)	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat	Investigate	Investigate	Coat
Structure with clearance of less than 3 m over stagnant water or less than 1.5 m over fresh water	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat
Structure over depressed roadways with tunnel effect	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat
Open grid decks	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize
Structure supporting open grid decks	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat
Faying surfaces of joints	—	—	—	—	—	—	—	—	—	—
Cables, ropes, and strands (see also Clause 10.6.4.3)	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize

Table 10.2
Corrosion protection for other components
(See Clause 10.6.5.)

Component	Environmental exposure condition											
	No direct chlorides			Air-borne chlorides or light industrial atmosphere			Heavy industrial atmosphere			Marine or de-icing runoff	In fresh water	In ground-water
	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry	Wet, rarely dry	Dry, rarely wet	Cyclical wet/dry			
Substructures	Coat	Uncoated weathering steel	Uncoated weathering steel	Coat	Uncoated weathering steel	Uncoated weathering steel	Coated	Investigate site conditions	Investigate site conditions	Coated	Uncoated	Uncoated
Sheet piling	Coat or increase section thickness	Uncoated	Coat or increase section thickness	Coat or increase section thickness	Uncoated	Coat or increase section thickness	Coat or increase section thickness	Coat or increase section thickness	Coat or increase section thickness	Coat or increase section thickness	Uncoated	Uncoated
Light poles, luminaires, and sign support structures	Galvanize	Uncoated weathering steel	Galvanize	Galvanize	Galvanize	Galvanize	Investigate site conditions	Investigate site conditions	Investigate site conditions	Investigate site conditions	—	—
Deck drains	Galvanize	Uncoated weathering steel	Uncoated weathering steel	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	—	—
Expansion joints	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	Galvanize or metallize	—	—
Bearings (excluding stainless steel and faying surfaces)	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	Galvanize, metallize, or coat	—	—
Faying surfaces of bearing assemblies (excluding stainless steel and Teflon®)	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	Coat	—	—
Moving components or rockers, roller bearings, and pins	Grease	Grease	Grease	Grease	Grease	Grease	Grease	Grease	Grease	Grease	—	—
Railings	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	—	—
Utility supports and hardware	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	Galvanize or epoxy coat	—	—
Components of mechanically stabilized earth structures, bin walls, and gabions	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize	Galvanize

Δ 10.6.6 Areas inaccessible after erection
Areas inaccessible after erection shall be marked in the Plans and shall be given an Approved protective coating before erection.

The inside surfaces of sealed hollow structural sections and sealed orthotropic deck ribs need not be protected.

10.6.7 Detailing for durability

10.6.7.1 Drip bars
Drip bars shall be secured to the bottom flanges of plate girders near expansion joints.

10.6.7.2 Interior bracing
Interior bracing shall be detailed to allow access for inspection and maintenance over the full length of the bridge.

10.6.7.3 Angles and tees
Angles and tees exposed to the environment shall be placed with their vertical legs or webs extending downward wherever practical.

10.6.7.4 End floor beams and end diaphragms
End floor beams and end diaphragms under expansion joints shall be arranged to permit coating and future maintenance of surfaces that are exposed to surface runoff. The end diaphragms of box girders shall be detailed to prevent ingress of water into the boxes.

10.6.7.5 Overpasses
Girder sections of overpasses over expressways and over urban streets with traffic speed limits greater than 70 km/h shall be detailed to minimize the detrimental effects of salt spray.

10.6.7.6 Pockets and depressions
Pockets and depressions that could retain water shall be avoided, provided with effective drainage, or filled with water-repellent material.

Δ 10.7 Design details

Δ 10.7.1 General
Members and connections shall be detailed to minimize their susceptibility to corrosion, fatigue, brittle fracture, and lamellar tearing.

10.7.2 Minimum thickness of steel
The minimum thickness of steel shall be as follows:
(a) gusset plates for main members and all material in end floor beams and end diaphragms and their connections: 10 mm;
(b) closed sections, e.g., tubular members or closed ribs in orthotropic decks that are sealed against entry of moisture: 6 mm;
(c) webs of rolled shapes: 6 mm;
(d) webs of plate girders and box girders: 10 mm; and
(e) other structural steel except for fillers, railings, and components not intended to resist loads: 8 mm.

10.7.3 Floor beams and diaphragms at piers and abutments

Floor beams and diaphragms at piers and abutments shall be designed to facilitate jacking of the superstructure unless the main longitudinal members are designed to be jacked directly.

10.7.4 Camber

10.7.4.1 Design

Girders with spans 25 m long or longer shall be cambered to compensate at least for dead load deflections and to suit the highway profile grade. For composite beams, an allowance shall also be made for the effects of creep and shrinkage of the concrete.

The Plans shall show

(a) the deflection of the girders due to the dead load of the steel alone; and

(b) the deflection due to the full dead load, including that of the steel, slab, barriers, sidewalks, and wearing surface.

For spans shorter than 25 m, the deflections and the profile of the concrete deck slab over the beams may be accommodated by increasing the slab thickness over the beams in lieu of providing a camber, if specified on the Plans.

10.7.4.2 Fabrication

Shop drawings shall show the total camber diagram to be used as a web cutting profile. The camber diagram shall include compensation for the deflection due to full dead load, an allowance for fabrication and welding distortion, and an allowance (if applicable) for the vertical alignment of the highway.

Δ 10.7.4.3 Horizontally heat-curved rolled or welded beams

For rolled beams and welded I-section plate girders that are heat curved to obtain a horizontal curvature, additional camber shall be added to compensate for the non-recoverable vertical deflection that occurs during construction and in service. The total camber shall be calculated as

$$\Delta = \left(\frac{\Delta_{DL} + \Delta_{SDL}}{\Delta_m}\right)\left[\Delta_m + \Delta_r\right]$$

where

$$\Delta_r = \frac{0.02L^2F_y}{E_s}\left[\frac{305\ 000 - R}{260\ 000}\right]\left(\frac{y_s + y_c}{y_s y_c}\right) \geq 0$$

where

Δ = specified camber at any section, mm

Δ_{DL} = camber at any point along length L, to compensate for deflection due to dead loads on the steel section only, mm

Δ_{SDL} = camber at any point along length L, to compensate for deflection due to dead loads on the composite section (if applicable), mm

Δ_m = maximum value of $\Delta_{DL} + \Delta_{SDL}$ within length L, mm

Δ_r = additional camber, mm

L = span length for simple spans, mm

= distance between the points of dead load contraflexure for continuous spans (see Figure A5.1.1), mm

F_y = specified minimum yield stress of flanges, MPa

E_s = modulus of elasticity of steel, MPa

R = horizontal radius of curvature, mm

y_s = distance from the neutral axis to the extreme outer fibre of the steel section (maximum distance for non-symmetrical sections), mm

y_c = maximum distance from the neutral axis to the extreme outer fibre of the composite section (if applicable), mm

10.7.5 Welded attachments

All attachments to primary tension and fracture-critical members, including transverse and longitudinal stiffeners, shall be connected by continuous welds. Longitudinal stiffeners shall be spliced by complete joint penetration groove welds.

10.8 Tension members

10.8.1 General

10.8.1.1 Proportioning

Tension members shall be proportioned on the basis of their gross and effective net cross-sectional areas and an examination of block tearout of the material. In cases where not all portions of a cross-section are directly connected to the adjoining elements, an effective net area shall be calculated as an allowance for shear lag.

10.8.1.2 Slenderness

The slenderness ratio of a tension member shall not exceed 200 unless otherwise Approved.

10.8.1.3 Cross-sectional areas

Δ 10.8.1.3.1 General

The gross and net cross-sectional areas to be used in calculating the resistance of a tension member shall be as follows:

(a) The gross cross-sectional area, A_{gv}, shall be the sum of the products of the thickness times the gross width of each element in the cross-section, measured perpendicular to the longitudinal axis of the member.

(b) The net cross-sectional area, A_n, shall be determined by summing the net areas of each segment along a potential path of minimum resistance, calculated as follows:
  (i) $A_n = w_n t$ for any segment normal to the force (i.e., in direct tension);
  (ii) $A_n = w_n t + s^2 t/4g$ for any segment inclined to the force

  where

  w_n = net width
  = gross width – sum of hole diameters in the gross width

Deductions for fastener holes shall be made using a hole diameter 2 mm greater than the specified hole diameter for punched holes. This allowance shall be waived for drilled holes or holes that are subpunched and reamed to the specified hole diameter.

Δ 10.8.1.3.2 Effective net area accounting for shear lag effects

In general, each portion of the cross-section of a tension member shall be connected at its ends with sufficient fasteners (bolts or welds) to transmit the load attributable to the portion being connected. Where this is not practicable, an effective net area shall be calculated as

$$A_{ne} = A_n\left(1 - \frac{\bar{x}}{L}\right)$$

where

$\bar{x}$ = distance perpendicular to axis of member from the fastener plane to the centroid of the portion of the area of the cross-section under consideration.

In the absence of a more precise method, the effective net area for shear lag shall be established as described in Clauses 10.8.1.3.2.1 to 10.8.1.3.2.5.

Δ 10.8.1.3.2.1 Bolted tension elements

When fasteners transmit load to each of the cross-sectional elements of a member in tension in proportion to their respective areas, the effective net area is equal to the net area, i.e., $A_{ne} = A_n$.

When bolts transmit load to some, but not all, of the cross-sectional elements and when the critical net area includes the net area of unconnected elements, the effective net area shall be taken as follows:

(a) for WWF, W, M, or S shapes with flange widths at least two-thirds the depth and for structural tees cut from those shapes, when only the flanges are connected with three or more transverse lines of fasteners: $A_{ne} = 0.90A_n$;

(b) for angles connected by only one leg with
  (i) four or more transverse lines of fasteners: $A_{ne} = 0.80A_n$; and
  (ii) fewer than four transverse lines of fasteners: $A_{ne} = 0.60A_n$; and

(c) for all other structural shapes connected with
  (i) three or more transverse lines of fasteners: $A_{ne} = 0.85A_n$; and
  (ii) with two transverse lines of fasteners: $A_{ne} = 0.75A_n$.

Δ 10.8.1.3.2.2 Welded tension elements

When a tension load is transmitted by welds, the effective net area shall be computed as

$$A_{ne} = A_{n1} + A_{n2} + A_{n3}$$

where

A_{n1}, A_{n2}, A_{n3} = net areas of the connected plate elements subject to one of the following methods of load transfer:

(a) for elements connected by transverse welds, A_{n1}:

$$A_{n1} = wt$$

(b) for elements connected by longitudinal welds along two parallel edges, A_{n2}:
  (i) when $L \geq 2w$:

$$A_{n2} = 1.00\ wt$$

  (ii) when $2w > L \geq w$:

$$A_{n2} = 0.50\ wt + 0.25\ Lt$$

  (iii) when $w > L$:

$$A_{n2} = 0.75\ Lt$$

  where

  L = average length of welds on the two edges, mm

  w = plate width (distance between welds), mm

(c) for elements connected by a single longitudinal weld, A_{n3}:
  (i) when $L \geq w$:

$$A_{n3} = \left(1 - \frac{\bar{x}}{L}\right) wt$$

  (ii) when $w > L$:

$$A_{n3} = 0.50\ Lt$$

where

$\bar{x}$ = eccentricity of the weld with respect to the centroid of the connected element, mm

L = length of weld in the direction of the loading, mm

The outstanding leg of an angle is considered connected by the (single) line of weld along the heel.

Δ 10.8.1.3.2.3 Rational analysis

Larger values of the effective net area may be used if justified by tests or rational analysis.

Δ 10.8.1.3.2.4 Block shear — Tension member, beam, and plate connections

The factored resistance for a potential failure involving the simultaneous development of tensile and shear component areas shall be taken as follows:

$$T_r = \phi_u \left[U_t A_n F_u + 0.6 A_{gv} F_m \right]$$

where

U_t = efficiency factor

= 1.0 when the failure pattern is symmetrical and the load concentric with the block. Otherwise, U_t is obtained from the following Table for specific applications.

Connection type	U_t
Flange connected tees	1.0
Angles connected by one leg and stem connected tees	0.6
Coped beams — one bolt line	0.9
Coped beams — two bolt lines	0.3

A_n is the net area in tension as defined in Clause 10.8.1.3.1 and A_{gv} is the gross area in shear, taken as the sum of the products of the thickness times the gross length of each segment of the block shear failure surface parallel to the applied force.

$$F_m = \frac{(F_y + F_u)}{2} \text{ for } F_y < 485 \text{ MPa, otherwise, } F_m = F_y$$

Note: *The term $0.6A_{gv}F_m$ in the above equation for T_r may be used to predict bolt tear out capacity along two parallel planes adjacent to the bolt hole.*

Δ 10.8.1.3.2.5 Angles

For angles, the gross width shall be the sum of the widths of the legs minus the thickness. The gauge for holes in opposite legs shall be the sum of the gauges from the heel of the angle minus the thickness.

10.8.1.4 Pin-connected members in tension

In pin-connected members in tension, the net area, A_n, across the pin hole and normal to the axis of the member shall be at least 1.33 times the cross-sectional area of the body of the member. The net area beyond the pin hole of any section on either side of the axis of the member, measured at an angle of 45° or less to the axis of the member, shall be not less than 0.9 times the cross-sectional area of the member.

The distance from the edge of the pin hole to the edge of the member, measured transverse to the axis of the member, shall not exceed four times the thickness of the material at the pin hole.

The diameter of a pin hole shall be not more than 1 mm larger than the diameter of the pin.

Δ 10.8.2 Axial tensile resistance

The factored tensile resistance, T_r, shall be taken as the least of

(a) $\phi_s A_g F_y$;
(b) $0.85\phi_s A_n F_u$; and
(c) $0.85\phi_s A_{ne} F_u$.

10.8.3 Axial tension and bending

Members subjected to bending moments and axial tensile forces shall satisfy the following relationship:

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1.0$$

where

M_r = $\phi_s M_p$ for Class 1 and 2 sections
= $\phi_s M_y$ for Class 3 sections

$$\frac{M_f}{M_r} - \frac{T_f Z}{M_r A} \leq 1.0 \text{ for Class 1 and 2 sections}$$

$$\frac{M_f}{M_r} - \frac{T_f S}{M_r A} \leq 1.0 \text{ for Class 3 sections}$$

Note: *Section classes are specified in Clause 10.9.2.1. M_r is specified in Clause 10.10.2 for Class 1 and 2 sections and in Clause 10.10.3 for Class 3 sections.*

10.8.4 Tensile resistance of cables

The factored axial tensile resistance, T_r, shall be taken as

$$T_r = \phi_{tc} T_u$$

10.9 Compression members

10.9.1 General

10.9.1.1 Cross-sectional area

Compression members shall be proportioned based on the gross area of the cross-section calculated by summing the products of the thickness and gross width of each element taken normal to the axis of the member.

10.9.1.2 Method of calculation

Provided that the requirements of Table 10.3 are met, the expressions for compressive resistance in Clause 10.9.3 shall apply. Flexural buckling with respect to the principal axes of the cross-section and torsional or flexural-torsional buckling shall be considered. Methods for calculating the compressive resistance of members, other than those specified in Clause 10.9.3, shall require Approval.

10.9.1.3 Slenderness

The slenderness ratio shall not exceed 120 for main compression members or 160 for secondary and bracing members.

10.9.2 Width-to-thickness ratio of elements in compression

10.9.2.1 General

Structural sections shall be designated as Class 1, 2, 3, or 4 depending on the width-to-thickness ratio of the elements that make up the cross-section and on the conditions of loading. A Class 1 section is one that will attain the plastic moment capacity, adjusted for the presence of axial force if necessary, and permit subsequent redistribution of bending moment. A Class 2 section is one that will attain the plastic moment capacity, adjusted for the presence of axial force if necessary, but not necessarily permit subsequent moment redistribution. A Class 3 section is one that will attain the yield moment capacity, adjusted for the presence of axial force if necessary. A Class 4 section is one in which the slenderness of the elements making up the cross-section exceeds the limits of Class 3. The capacity of a Class 4 section shall be treated on a case-by-case basis in accordance with this Code.

The width-to-thickness ratios of elements subject to compression shall not exceed the limits specified in Table 10.3.

Δ

Table 10.3
Width-to-thickness ratio of elements in compression

(See Clauses 10.9.1.2, 10.9.2.1, 10.10.2.1, and 10.10.3.1.)

Description of element	Class 1	Class 2	Class 3‡
Legs of angles and elements supported along one edge, except as covered elsewhere in this Table	—	—	$b/t \le 200/(\sqrt{F_y})$
Angles in continuous contact with other elements; plate girder stiffeners	—	—	$b/t \le 200/(\sqrt{F_y})$
Stems of T-sections	$b/t \le 145/(\sqrt{F_y})$*	$b/t \le 170/(\sqrt{F_y})$*	$b/t \le 340/(\sqrt{F_y})$
Flanges of I- or T-sections; plates projecting from compression elements; outstanding legs of pairs of angles in continuous contact†	$b/t \le 145/(\sqrt{F_y})$	$b/t \le 170/(\sqrt{F_y})$	$b/t \le 200/(\sqrt{F_y})$
Flanges of channels	—	—	$b/t \le 200/(\sqrt{F_y})$
Flanges of rectangular hollow structural shapes	$b/t \le 420/(\sqrt{F_y})$	$b/t \le 525/(\sqrt{F_y})$	$b/t \le 670/(\sqrt{F_y})$
Flanges of box girder sections; flange cover plates and diaphragm plates between lines of fasteners or welds	$b/t \le 525/(\sqrt{F_y})$	$b/t \le 525/(\sqrt{F_y})$	$b/t \le 670/(\sqrt{F_y})$
Perforated cover plates	—	—	$b/t \le 840/(\sqrt{F_y})$
Webs in axial compression	$h/w \le 670/(\sqrt{F_y})$	$h/w \le 670/(\sqrt{F_y})$	$h/w \le 670/(\sqrt{F_y})$
Webs in flexural compression	$h/w \le 1100/(\sqrt{F_y})$	$h/w \le 1700/(\sqrt{F_y})$	$h/w \le 1900/(\sqrt{F_y})$
Webs in combined flexural and axial compression	$\frac{h}{w} \le \frac{1100}{\sqrt{F_y}}\left[1-0.39\frac{C_f}{C_y}\right]$	$\frac{h}{w} \le \frac{1700}{\sqrt{F_y}}\left[1-0.61\frac{C_f}{C_y}\right]$	$\frac{h}{w} \le \frac{1900}{\sqrt{F_y}}\left[1-0.65\frac{C_f}{C_y}\right]$
Circular and multi-sided hollow sections in axial compression	—	—	$\frac{D}{t} \le \frac{23\,000}{F_y}$
Circular and multi-sided hollow sections in flexural compression	$\frac{D}{t} \le \frac{13\,000}{F_y}$	$\frac{D}{t} \le \frac{18\,000}{F_y}$	$\frac{D}{t} \le \frac{66\,000}{F_y}$

**Class 1 and 2 sections subjected to flexure having an axis of symmetry in the plane of loading unless the effects of asymmetry of the section have been included in the analysis.*

†Can be considered a Class 1 or 2 section, as applicable, only if angles are continuously connected by adequate mechanical fasteners or welds and there is an axis of symmetry in the plane of loading.

‡A Class 4 section is a section that exceeds the limits of a Class 3 section.

10.9.2.2 Elements supported along one edge

For elements supported along only one edge that is parallel to the direction of the compressive force, the width, b, shall be taken as follows:

(a) for plates: the distance from the free edge to the line of bolts or welds;
(b) for legs of angles, flanges of channels and zees, or stems of tees: the full nominal dimension; and
(c) for flanges of I-shapes and tees: one-half of the flange width.

10.9.2.3 Elements supported along two edges

For elements supported along two edges that are parallel to the direction of the compressive force, the width, b or h, as applicable, shall be taken as follows:

(a) for flange or diaphragm plates in built-up sections, b shall be taken as the distance between adjacent lines of bolts or lines of welds;
(b) for the sides of rectangular hollow structural sections, b or h shall be taken as the clear distance between edge-supporting elements less two wall thicknesses;
(c) for webs of built-up sections, h shall be taken as the distance between the nearest lines of bolts connecting either edge of the web or as the clear distance between flanges when welds are used; and
(d) for webs of rolled sections, h shall be taken as the clear distance between flanges.

10.9.2.4 Thickness

In all cases, the thickness of elements shall be taken as the nominal thickness. For tapered flanges, the thickness shall be taken as that at the midpoint of the element.

10.9.2.5 Multi-sided hollow sections

For multi-sided hollow sections that approximate a circle, D shall be taken as the diameter of the circle that inscribes the outside of the midpoint of the flats of the section.

10.9.3 Axial compressive resistance

Δ 10.9.3.1 Flexural buckling

The factored axial compressive resistance, C_r, of a member conforming to the limitations specified in Clauses 10.9.1 and 10.9.2 shall be taken as

$$C_r = \phi_s A F_y (1 + \lambda^{2n})^{-1/n}$$

where

$\lambda = \dfrac{KL}{r}\sqrt{\dfrac{F_y}{\pi^2 E_s}}$

n = 1.34, except for welded H-shapes with flame-cut flange edges and hollow structural sections manufactured in accordance with CSA G40.20, Class H (i.e., hot-formed or cold-formed stress-relieved sections), where $n = 2.24$

10.9.3.2 Torsional or flexural-torsional buckling

The torsional or flexural-torsional buckling resistance of asymmetric, singly symmetric, and cruciform sections shall be calculated by using $n = 1.34$ and replacing λ in Clause 10.9.3.1 by λ_e, as follows:

$$\lambda_e = \sqrt{F_y / F_e}$$

10.9.5 Composite columns

10.9.5.1 General

The requirements of Clause 10.9.5 shall apply to composite columns consisting of steel hollow structural sections completely filled with concrete. The type of concrete, its strength, and its other properties shall comply with Section 8.

Δ 10.9.5.2 Application

Hollow structural sections designated as Class 1, 2, or 3 sections shall be assumed to carry compressive load as composite columns. Class 4 hollow structural sections that are completely filled with concrete and are designed as composite columns shall have, for walls of rectangular sections, width-to-thickness ratios that do not exceed $1350/\sqrt{F_y}$, and for circular sections, outside diameter-to-thickness ratios of circular sections that do not exceed $28\,000/F_y$.

10.9.5.3 Axial load on concrete

The axial load assumed to be carried by the concrete at the top level of a column shall be only that portion applied by direct bearing on the concrete. A base plate or similar means shall be provided for load transfer at the bottom.

Δ 10.9.5.4 Compressive resistance

The factored compressive resistance of a composite column, C_{rc}, shall be taken as

$$C_{rc} = \tau C_r + \tau' C_r'$$

where

C_r = the value specified in Clause 10.9.3.1

C_r' = $0.85\phi_c f_c' A_c \lambda_c^{-2}\left[\sqrt{1+0.25\lambda_c^{-4}} - 0.5\lambda_c^{-2}\right]$

where

λ_c = $\dfrac{KL}{r_c}\sqrt{\dfrac{f_c'}{\pi^2 E_c}}$

E_c = initial elastic modulus for concrete, taking into consideration the effects of long-term loading for normal weight concrete, with f_c' expressed in megapascals

= $2500(1+S/T)\sqrt{f_c'}$

where

S = short-term load

T = total load on the column

$\tau = \tau'$ = 1.0; except for circular hollow sections with a height-to-diameter ratio (L/D) of less than 25, for which

$$\tau = \frac{1}{\sqrt{1+\rho+\rho^2}}$$

and

$$\tau' = 1+\left[\frac{25\rho^2\tau}{D/t}\right]\left[\frac{F_y}{0.85f_c'}\right]$$

where

ρ = $0.02\,(25 - L/D)$

10.9.5.5 Bending resistance

The factored bending resistance, M_{rc}, of a composite concrete-filled hollow structural section shall be taken as

$$M_{rc} = C_r e + C_r' e'$$

where

(a) for a rectangular hollow structural section:

$$C_r = \frac{\phi_s A_s F_y - C_r'}{2}$$

$$C_r' = \phi_c a(b - 2t) f_c'$$

$$C_r + C_r' = T_r$$

$$= \phi_s A_{st} F_y$$

Note: *The concrete in compression is taken to have a rectangular stress block of intensity f_c' over a depth of $a = 0.85c$, where c is the depth of concrete in compression.*

(b) for a circular hollow structural section:

$$C_r = \phi_s F_y \beta \frac{Dt}{2}$$

$$e = b_c\left[\frac{1}{(2\pi - \beta)} + \frac{1}{\beta}\right]$$

$$C_r' = \phi_c f_c'\left[\frac{\beta D^2}{8} - \frac{b_c}{2}\left[\frac{D}{2} - a\right]\right]$$

$$e' = b_c\left[\frac{1}{(2\pi - \beta)} + \frac{b_c^2}{1.5\beta D^2 - 6b_c(0.5D - a)}\right]$$

where

β = value in radians derived from the following recursive equation:

$$\beta = \frac{\phi_s A_s F_y + 0.25\phi_c D^2 f_c'\left[\sin(\beta/2) - \sin^2(\beta/2)\tan(\beta/4)\right]}{0.125\phi_c D^2 f_c' + \phi_s D t F_y}$$

b_c = $D \sin(\beta/2)$

a = $b_c/2 \tan(\beta/4)$

Conservatively, M_{rc} may be taken as

$$M_{rc} = \left(Z - 2th_n^2\right)\phi_s F_y + \left[2/3(0.5D - t)^3 - (0.5D - t)h_n^2\right]\phi_c f_c'$$

where

$$h_n = \frac{\phi_c A_c f_c'}{2D\phi_c f_c' + 4t\left(2\phi_c F_y - \phi_c f_c'\right)}$$

Z = plastic modulus of the steel section alone

10.9.5.6 Axial compression and bending resistance

Members required to resist both bending moments and axial compression shall be proportioned analogously with Clause 10.9.4 so that

$$\frac{C_f}{C_{rc}} + \frac{B\omega_1 M_f}{M_{rc}\left[1 - \frac{C_f}{C_{ec}}\right]} \le 1.0$$

$$\frac{M_f}{M_{rc}} \le 1.0$$

$$B = \frac{C_{rco} - C_{rcm}}{C_{rco}}$$

where

C_{rco} = factored compressive resistance with $\lambda = 0$

C_{rcm} = factored compressive resistance that can coexist with M_{rc}

M_{rc} = the value specified in Clause 10.9.5.5

Conservatively, B may be taken as equal to 1.0.

10.10 Beams and girders

10.10.1 General

10.10.1.1 Cross-sectional area

Beams and girders shall be proportioned on the basis of the geometric properties of the gross section, except that a deduction shall be made for the area of the bolt holes exceeding 15% of the gross flange area.

10.10.1.2 Flange cover plate restrictions

Flanges of welded beams or girders shall consist of single plates or a series of plates, joined end-to-end by complete penetration groove welds. The use of welded partial-length cover plates shall require Approval.

10.10.1.3 Lateral support

Lateral support of compression flanges shall be provided by adequate connection to the deck or by bracing capable of restraining lateral displacement and twisting of the beams and girders unless it can be demonstrated that such restraint is developed between the steel beam and the concrete slab. Wood decks shall not be considered to provide lateral support unless the deck and fastenings are designed for this purpose.

10.10.1.4 Flange-to-web connections

Welds connecting flanges to webs shall be proportioned to resist interface shear due to bending combined with any loads that could be transmitted from the flange to the web other than by direct bearing.

10.10.2 Class 1 and 2 sections

10.10.2.1 Width-to-thickness ratios

Class 1 and 2 sections subject to flexure and having an axis of symmetry in the plane of loading shall meet the requirements of Clause 10.9.2. For calculating the limiting width-to-thickness ratios of the web of monosymmetric sections, h in Table 10.3 shall be replaced by $2d_c$.