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PRECISIONSM RESOLUTION TARGETS

**Behaviour of Slender Webs of WWF Shapes and Stiffened Bridge
Girders Subjected to Concentrated In-Plane Loads : Experimental
Investigation**

by

Dayle Prowse

BSc. Civil Engineering, University of Alberta, Edmonton, Alberta, Canada

A thesis submitted to the
Faculty of Graduate Studies and Research
in partial fulfillment of the requirements
for the degree of

Master of Engineering*

Department of Civil and Environmental Engineering
Carleton University
Ottawa, Canada

November, 1996

*The Masters Degree in Civil Engineering
is a joint program with the University of Ottawa, administered by
the Ottawa-Carleton Institute for Civil Engineering

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Department	1990-91	1991-92	1992-93	1993-94	1994-95	1995-96	1996-97	1997-98	1998-99	1999-00	2000-01	2001-02	2002-03	2003-04	2004-05	2005-06	2006-07	2007-08	2008-09	2009-10	2010-11	2011-12	2012-13	2013-14	2014-15	2015-16	2016-17	2017-18	2018-19	2019-20	2020-21	2021-22	2022-23	2023-24	2024-25	2025-26	2026-27	2027-28	2028-29	2029-30	2030-31	2031-32	2032-33	2033-34	2034-35	2035-36	2036-37	2037-38	2038-39	2039-40	2040-41	2041-42	2042-43	2043-44	2044-45	2045-46	2046-47	2047-48	2048-49	2049-50	2050-51	2051-52	2052-53	2053-54	2054-55	2055-56	2056-57	2057-58	2058-59	2059-60	2060-61	2061-62	2062-63	2063-64	2064-65	2065-66	2066-67	2067-68	2068-69	2069-70	2070-71	2071-72	2072-73	2073-74	2074-75	2075-76	2076-77	2077-78	2078-79	2079-80	2080-81	2081-82	2082-83	2083-84	2084-85	2085-86	2086-87	2087-88	2088-89	2089-90	2090-91	2091-92	2092-93	2093-94	2094-95	2095-96	2096-97	2097-98	2098-99	2099-00	2100-01	2101-02	2102-03	2103-04	2104-05	2105-06	2106-07	2107-08	2108-09	2109-10	2110-11	2111-12	2112-13	2113-14	2114-15	2115-16	2116-17	2117-18	2118-19	2119-20	2120-21	2121-22	2122-23	2123-24	2124-25	2125-26	2126-27	2127-28	2128-29	2129-30	2130-31	2131-32	2132-33	2133-34	2134-35	2135-36	2136-37	2137-38	2138-39	2139-40	2140-41	2141-42	2142-43	2143-44	2144-45	2145-46	2146-47	2147-48	2148-49	2149-50	2150-51	2151-52	2152-53	2153-54	2154-55	2155-56	2156-57	2157-58	2158-59	2159-60	2160-61	2161-62	2162-63	2163-64	2164-65	2165-66	2166-67	2167-68	2168-69	2169-70	2170-71	2171-72	2172-73	2173-74	2174-75	2175-76	2176-77	2177-78	2178-79	2179-80	2180-81	2181-82	2182-83	2183-84	2184-85	2185-86	2186-87	2187-88	2188-89	2189-90	2190-91	2191-92	2192-93	2193-94	2194-95	2195-96	2196-97	2197-98	2198-99	2199-00	2200-01	2201-02	2202-03	2203-04	2204-05	2205-06	2206-07	2207-08	2208-09	2209-10	2210-11	2211-12	2212-13	2213-14	2214-15	2215-16	2216-17	2217-18	2218-19	2219-20	2220-21	2221-22	2222-23	2223-24	2224-25	2225-26	2226-27	2227-28	2228-29	2229-30	2230-31	2231-32	2232-33	2233-34	2234-35	2235-36	2236-37	2237-38	2238-39	2239-40	2240-41	2241-42	2242-43	2243-44	2244-45	2245-46	2246-47	2247-48	2248-49	2249-50	2250-51	2251-52	2252-53	2253-54	2254-55	2255-56	2256-57	2257-58	2258-59	2259-60	2260-61	2261-62	2262-63	2263-64	2264-65	2265-66	2266-67	2267-68	2268-69	2269-70	2270-71	2271-72	2272-73	2273-74	2274-75	2275-76	2276-77	2277-78	2278-79	2279-80	2280-81	2281-82	2282-83	2283-84	2284-85	2285-86	2286-87	2287-88	2288-89	2289-90	2290-91	2291-92	2292-93	2293-94	2294-95	2295-96	2296-97	2297-98	2298-99	2299-00	2300-01	2301-02	2302-03	2303-0
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Green Technology www.green-tech.com **0203**
Technology www.tech.com **0203**
Home Cinema www.hc.com **0204**

Pure Sciences	
Category	Percentage
General	0.285
Agricultural	0.749
Architectural	0.025
Chemistry	0.025
Engineering	0.025
Medical	0.025
Organic	0.025
Pharmaceutical	0.071
Physical	0.024
Public	0.025
Statistics	0.721
Mathematics	0.025
Physics	
General	0.025
Astronomy	0.024
Astronomy and	
Astrophysics	0.025
Atmospheric Sciences	0.025
Atomic	0.025
Electronics and Electricity	0.027
Elementary Physics and	
High Energy	0.025
Fluid and Plasma	0.025
Materials	0.025
Medical	0.025
Optics	0.025
Other	0.025
Relativity	0.025
Solid State	0.021
Statistics	0.025
Applied Sciences	
Applied Mathematics	0.025
Computer Science	0.025

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Civil	0.00
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Mechanical	0.00
Marine	0.00
Materials Science	0.00
Mathematical	0.00
Metallurgy	0.00
Mining	0.00
Nuclear	0.00
Textile	0.00
Transportation	0.00
Sanitary and Municipal	0.00
System Science	0.00
Geodesy	0.00
Geomatics Engineering	0.00
Fluid Technology	0.00
Fluid Technology	0.00

Department of Civil and Environmental Engineering

The undersigned recommend to the Faculty of Graduate
Studies and Research acceptance of the thesis

**Behaviour of Slender Webs of WWF Shapes and Stiffened Bridge Girders
Subjected to Concentrated In-plane Loads : Experimental Investigation**

submitted by

Dayle Prowse

in partial fulfilment of the requirements for
the degree of Master of Engineering

Dr. D. J. L. Kennedy, External Examiner

Dr. J. L. Humar, Chair, Department of Civil and
Environmental Engineering

Dr. Stephen J. Kennedy, Thesis Supervisor

Carleton University
Ottawa Ontario, Canada
November, 1996

Abstract

This experimental study is an investigation of the behaviour of webs of slender WWF sections and bridge girders under concentrated in-plane loading. Previous tests done by Benichou (1994) and Kirkhope (1995) tested the full range of W and WWF shapes manufactured in Canadian Mills. This research, combined with analyses recently completed by Prabha (1996), led to the development of the equivalent column concept and appropriate design equations applicable to W and WWF sections. This research considers the behaviour of built-up sections, the next topic of investigation.

A total of eight tests were conducted. Four tests consisting of two interior reaction tests and two end reaction tests were made on two WWF 985x71 sections. These two sections had different welds with the web to flange welds either on both sides of the web, as is the normal practice, or on one side only. Four tests with interior loads were made on bridge girders with longitudinal stiffeners and with h/w ratios varying from 175 to 285. The test data acquired from these tests, together with measurements of the initial imperfections, and geometric and material properties, provide the basis for further analytical and statistical investigations to be extended to modify the equivalent column models to include built-up sections, as may be required.

Acknowledgements

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List of Symbols

a	distance between vertical stiffeners
a₁	measured distance from flange tip to web face
b	width of flange tip to the face of the web
Br	bearing resistance of web
c	resistance factor
d,d₁,d₂	depths of beam
E	modulus of elasticity
E_s	strain hardening modulus
F_y	yield stress of plate material
h	height of web plate
I	moment of inertia
K	effective length factor of equivalent column
k	distance from outer face of flange to the base of the web fillet
L	length of beam
N	length of patch load
P_{cr}	elastic buckling load of equivalent column
P_{ult}	ultimate capacity of web

P_y	yield load of column
t	thickness of flange plate
w	thickness of web plate
v	coefficient of variation
α, β	dimensions defining Roberts (1981) collapse mechanism
ϵ_y	strain at yield
ϵ_{sh}	strain hardening strain
ϵ_u	strain at ultimate
ϵ_f	strain at fracture
λ	slenderness parameter
λ_e	equivalent column slenderness parameter
ϕ	material resistance factor
ν	Poisson's ratio
$\sigma(x)$	uniform stress on equivalent column at a certain depth
σ_y	stress at yield
σ_{sy}	static yield stress
σ_u	stress at ultimate
σ_f	stress at fracture
σ_{max}	maximum uniform stress on equivalent column

Chapter 1

Introduction

1.1 General Introduction

The bearing resistance of webs of rolled shapes, WWF sections and bridge girders must be sufficient to sustain all concentrated loads which may be applied by beams, purlins, or columns framing into the section or from wheel loads on gantry girders. In addition to these regular loading conditions, girders may be required to sustain various types of extreme loading conditions during construction, such as roller loads (i.e. Hillman rollers) which occur when a bridge is launched during construction. If the web capacity is not properly predicted either an excessive or inadequate number of bearing stiffeners may be provided. Obviously, a correct estimate of the resistance of the web is necessary for a reliable and economical design. Questions about the applicability of current design equations may lead to uncertainty, and in general, over-design and increased cost. The development of a consistent, unified, and easily understood approach for predicting the capacity of webs under concentrated loads therefore is of interest.

The design equations for calculating the bearing resistance should cover the full range of behaviour from web yielding to elastic and inelastic web buckling for all shapes currently being mass produced by North American Mills, and for customized shapes either, with or without longitudinal stiffeners like those being used in long, multi-span, bridge girders.

1.2 Web Behaviour and Bearing Resistance

A chronological list of researchers with key words, reproduced and amended from Benichou (1994), is given in Figure 1.1. A detailed description of the work prior to 1994 can be found in Benichou (1994). A brief summary of recent research including work done by Benichou (1994), Kirkhope (1995) and Prabha (1996) is presented in the following sections.

1.2.1 Elastic Web Buckling

Elastic web buckling solutions and buckling coefficients based on classical plate buckling (energy methods) and numerical methods (finite difference and finite element analysis) were developed by Sommerfield (1906), and Timoshenko (1910), and White and Cottingham (1962), to determine the effects of plate geometry, boundary conditions, load

effects, assumed deflected shapes, and internal force distributions. These, however, are of limited value because they have not been applied to realistic geometries and load conditions, the slenderness parameter has not been identified, the corresponding rational, mathematical models have not been developed, and there is no evidence of any test ever failing by elastic web buckling.

1.2.2 Inelastic Web Buckling and Web Yielding

While elastic buckling solutions were developed analytically, many tests were also done to establish empirical bearing capacities for inelastic web buckling and web yielding. Although many tests were done in the last 60 years, the empirical relationships are generally unsuitable for developing a rational model as the data are incomplete or the tests were conducted on shapes that either no longer exist, or were made of a low grade steel that is no longer used. It is however, of interest to discuss the historical background of these findings and their relevancy to this study.

Based on observations of Bergfelt et al. (1968, 1970, 1971), Bergfelt and Lindgren (1974), Bagchi and Rockey (1975), Skaloud and Novak (1972), Drdacky and Novotny (1977), Bergfelt (1979), and Roberts and Rockey (1979) developed a lower bound plastic mechanism model requiring the development of four plastic hinges in the flange and three yield lines in the web. Using the conservation of energy method and by minimizing the

critical load with respect to the locations of the plastic hinges in the flange, the authors developed two semi-empirical formulae to predict the inelastic web buckling capacity (equation 1.1) and web yielding (equation 1.2) with the lesser of the two governing the capacity of the web. These are, respectively:

$$P_u = \frac{4M_w}{\beta} + \frac{(4\beta + 2N - 2\eta)M_f}{\alpha \cos \theta} \quad (1.1)$$

$$P_u = 2(4M_w F_w)^{1/2} + F_w N \quad (1.2)$$

where, P_u = ultimate load, N

M_w = web plastic moment per unit width of web, N·mm

M_f = plastic moment capacity of flange, N·mm

β = location of plastic hinges, mm

θ = angle of yield lines, mm

α = location of yield lines, mm

η = length of web which yielded in direct compression, mm

Subsequently, Roberts, (1981), conducted more tests on slender short-spanned plate girders and modified the expression for inelastic web buckling to:

$$P_c = 0.5w' \sqrt{E F_y \frac{t}{w}} \left(10 + 3.0 \left(\frac{N}{d} \right) \left(\frac{w}{t} \right)^{1.5} \right) \quad (1.3)$$

where, P_c = web crippling load, N

t = flange thickness, mm

d = web panel depth, mm

This equation was adopted into the CSA Standard CAN/CSA-S16.1-M89, where the bearing resistance of webs failing by inelastic web buckling (crippling) for interior load cases is given in Clause 15.9 as follows:

$$B_r = 300\phi w^2 \left[1 + 3 \left(\frac{N}{d} \right) \left(\frac{w}{t} \right)^{1.5} \right] \sqrt{F_y \frac{t}{w}} \quad (1.4)$$

where, B_r = factored bearing resistance, N

ϕ = resistance factor, 0.9

When comparing these two equations there is a difference of 1.34 times the value of

$\frac{300}{0.5\sqrt{E}}$, based on a reassessment of Roberts' data. This equation was used for end

reactions with the value of 300 reduced to 150 based on the assumption that in the limit

for short reaction lengths, the load spreads out when transmitted through the flange to the web on one side rather than two as for interior loads.

The upper limit to inelastic behaviour is defined as web yielding which in turn is defined by S16.1 Clause 15.9 (a) (i) as:

$$B_r = 1.10\phi_w(N + 5k)F_y \quad (1.5)$$

This expression is based on experimental research provided by Graham *et al.* (1959), on welded interior beam to column moment connections. The critical area is based on yielding of the web of the column opposite the compression flange of the beam with a load dispersion of 2.5 to 1.0.

The applicability of these design equations is questioned because:

1. The web yielding equation (Eq. 1.5) is based on small load lengths equal to the beam flange thickness and stocky webs. A calibration coefficient of 1.1 was introduced when the standard was adapted to a limit states design format, rather than redefining the critical area.
2. The inelastic web buckling (crippling) equations (Eq. 1.4) are based on the empirical expressions given by Robert's (1981), which in turn is based on experimental observations from tests on shapes which had geometric properties and slenderness ratios that generally

lay outside the range of values for W and WWF shapes produced by Canadian Mills. The tests were also conducted on relatively slender plates, and generally on a small scale. This leads to problems with regard to providing realistic welding details, loaded lengths, boundary conditions, and imperfections. The limitations of flange thickness to web width ratio greater than or equal to 3.0 and the loaded length to web depth ratio less than or equal to 0.2 are not given as part of Clause 15.9, although they are specifically stated by Roberts.

Based on the historical development of these design expressions in CAN/CSA S16.1-M89 including the obvious questions related to the applicability, the lack of a unified approach to define all forms of probable failure (i.e. web yielding, elastic web buckling), and the semi-empirical nature of the design equations (which have no physical interpretation for design engineers), Benichou (1994), Kirkhope (1995), and Prabha (1996) conducted further research. The objectives of these programs were:

1. to provide quality test data for the full range of W and WWF shapes produced in Canadian Mills that exhibit all probable web failure modes;
2. to conduct finite element analyses to obtain a whole field description of the web behaviour. These are verified against test data;

3. to develop a rational mathematical model and the corresponding design equations to resolve some of the issues cited previously.

Benichou's work encompassed experimental testing and finite element analyses of the full range of W shapes produced by Canadian mills for both interior loading conditions and end reaction tests. He developed an equivalent column concept which took into account the internal stress distribution, the buckled shape of the web, the equivalent column dimensions for both W and WWF shapes. However, as a result of further experimental research conducted on WWF shapes, Kirkhope (1995) concluded these equations were in general not applicable to WWF shapes. His work served as a verification for finite element results recently completed by Prabha (1996).

From the finite element analysis, Prabha (1996), was able to revise Benichou's original equivalent column concept to make it applicable to WWF shapes. Prabha showed that the equivalent column width of $N+5k$ originally determined by Benichou is not applicable to WWF shapes, the normal stress distributions were different, the assumed buckled shapes utilized inconsistent boundary conditions, and the energy method solution that was used was based on columns not plates. Instead, a new equivalent column width of $N+10t$, using the energy method of Timoshenko and Gere (1961) for the buckling of

thin plates, was proposed as well as a new curve fit for the average normal stresses in the form below.

$$\sigma(z) = \sigma_{\max} \left(\frac{z}{h} \right)^p \quad (1.6)$$

where, $\sigma(z)$ = average normal stress distribution at any depth, z, MPa

$\sigma_{\max}$ = maximum average normal stress at root of fillet weld on the web, MPa

p = any positive integer

From this, the elastic critical buckling load for a fixed end plate with the load distribution described above can be determined from:

$$P_{cr} = f(N + 10t) \frac{\pi^2 D}{h^2} \quad (1.7)$$

where, f is a function of h and N+10t as described by Prabha (1996) and D is the plate flexural rigidity. Prabha (1996) also gives an equivalent slenderness parameter so that column buckling curves can be used. This is found from:

$$\frac{P_{cr}}{P_y} = \frac{1}{\lambda_e^2} \quad (1.8)$$

where, $\lambda_e = c \left(\frac{h}{r} \right) \sqrt{\frac{F_y}{\pi^2 E}}$

r = radius of gyration of the equivalent column model cross section

c = a plate buckling coefficient which is a function of the boundary conditions, the displaced shape, the average normal stress distribution, and aspect ratio of the plate.

Prabha (1996), working on webs subjected to concentrated loads, proposed a new Clause 15.9.1 for the factored bearing resistance of a web.

$$B_r = \phi A F_y (1 + \lambda_e^{2n})^{-\frac{1}{n}} \quad (1.9)$$

where, $n = 2.24$ and λ_e is the equivalent slenderness parameter as defined previously.

In this summary, only beams with h/w ratios of 87 or less were studied and therefore any plate girders or customized built-up WWF's are excluded. Currently, WWF sections with thin webs up to an h/w of 285 are being fabricated but very little experimental work has been done on beams with very thin webs and research is needed to establish the bearing resistance of such sections.

1.3 Objectives

The focus of this study is to enable engineers to design steel girders more accurately and economically. Therefore some recently constructed steel bridges were examined to determine the types of girders currently being used as shown in Table 1.1. The h/w ratios range from 129 (the Annacis River Bridge in British Columbia) to 309 (Rideau Bridge in Ontario), with about 85% of the aspect ratios lying between 160 and 285, categorizing the girders as thin-webbed beams. Based on this, the girders selected for this research had h/w ratios of 175 to 285. The length of the beams tested, to ensure that true flexural action was achieved, ranged from 5.5m to 7m, respectively.

Most of the bridge girders cited were designed and put into use without regard to experimental evidence to determine the actual capacity of their webs to concentrated loads. Engineers referred to alternate codes, specifications or material standards, such as the British Standards (BS 5400) that deal with web crippling in considerably more detail. Unfortunately, mixing of codes is not conducive to good design and should be done carefully.

The objectives of this experimental study are:

- 1. to design a number of stiffened plate girders that are representative of the full range of geometric parameters currently used in typical bridge construction that will fail by inelastic or elastic web buckling or web yielding;**
- 2. to design a series of Watt Mechanisms to provide lateral support to the plate girders with minimal restraint in the plane of the web;**
- 3. to design an LVDT support device to give a consistent reference point to improve the accuracy and reliability of transverse displacements of the web;**
- 4. to test each girder to determine the complete range of web behaviour, generating thereby all the data required for the verification of the results of finite element analyses and the rational mathematical design models that will be used to describe the behaviour of thin-webbed plate girders subjected to concentrated loads;**
- 5. to test two WWF 987x71 sections with an h/w ratio of 180, one with fillet welds on both sides of the web and one with welds on one side only, for interior loads and end reactions in order to provide the data to complete the series of tests on WWF sections.**

1.4 Outline

After a general introduction, Chapter 1 presents an analysis of Clause 15.9 in CAN/CSA S16.1-M94, a state of the art review including a general description of the behaviour of webs subjected to concentrated loads, and the justification for this research. Chapter 2 describes the experimental program and Chapter 3 presents the test results and a description of web behaviour under concentrated loads and the failure modes. The reduced data presented in Chapter 4 provide the information required for an accurate comparison and verification of the corresponding results of future finite element analysis and for the continued development of the equivalent column concept for bridge girders. A summary, conclusions and recommendations for future work are given in Chapter 5.

Figure 1.1 Research Development for Web Buckling

Figure 1.1 (cont.) Research Development for Web Buckling

Figure 1.1 (cont.) Research Development for Web Buckling

Bridge	Company	w x h	b/w ratio	Construction Method	Stiffener Spacing
Peace River Bridge Alberta	Buckland and Taylor	16x4545	284	launched	1300mm-1875mm
Deserter's Canyon Bridge	Buckland and Taylor	12x3000	250	launched	5110mm-5500mm
Castelgar-Robson Bridge British Columbia	Buckland and Taylor	16x2500	156	launched	1520mm-3042mm
French Creek Bridge British Columbia	Buckland and Taylor	14x2250 10x2250	160 225	crane	1900mm-2389mm
Rideau Bridge -Hunt Club Ontario	DELCAN	4944x16 16x4197	309 262	crane	1100mm-2500mm
Burlington Bay Skyway Ontario	Ontario Ministry of Transportation	16x2570 10x2570	184 257	crane	2125mm-3600mm
Annacis River Bridge British Columbia	DELCAN	14x1800	129	crane	273mm-1455mm
Perley Bridge Ontario	CANRON	14x2350	168	launched	1902mm-3806mm
Rideau River Bridge S.B.L. Ontario	DELCAN	12x2062	172	crane	1000mm-4000mm

Table 1.1 Typical Current Bridge Girder Dimensions

Chapter 2

Experimental Program

2.1 General

The experimental program was designed to obtain test data that describes the behaviour and strength of thin-webbed girders subjected to in-plane concentrated loads. The data is analyzed and presented in a format suitable for comparison with the results of finite element analyses.

The bridge girders and WWF 985 x 71 were selected to give a sample of slender plate girders with h/w values varying from 175 to 285. They were designed with shear span ratios of 1.67 to 1.96, with spans of 2750 mm to 3500 mm to avoid deep beam behaviour. The flanges were sized and enough lateral support was provided by the Watt mechanisms so that the moment capacity of the girders would not be exceeded in the tests. Vertical stiffeners had to be added in order to increase the shear resistance beyond the probable load that would cause local failure of the web. For the WWF 985x71 sections,

doubler plates were clamped to the web over the reaction points to prevent local instability and failure at that location. For the end reaction tests the doubler plates were removed from one end and added to the central portion of the beam under the load for the same reason. The vertical and longitudinal stiffeners of the bridge girders were designed according to the Ontario Highway Bridge Design Code (1992).

The nominal geometric properties for the test girders are given in Table 2.1 and are shown in Figure 2.1. Also listed is the elastic critical buckling load as determined by Prabha (1996), from a bifurcation analysis using the commercial finite element program LUSAS, the nominal geometric properties and a value for the modulus of elasticity of 200,000 MPa. In total, eight web instability tests were conducted: two (one interior load and one end reaction) on each WWF 985x71 (one with a one-sided flange to web fillet weld, designated WWF 985x71 (a) and one with a two-sided fillet weld, designated WWF 985x71 (b)) and one interior load test on each of the four bridge girders. Two WWF 1850x273 girders were made so that a comparison of the behaviour and load carrying capacity of the web could be made depending on the location of the longitudinal stiffener with respect to the loaded flange. The WWF 1850x273 (a) and the WWF 1850x273 (b) girders have the longitudinal stiffener located at $0.2h$ and $0.8h$ from the loaded flange, respectively. Based on experience from similar type tests on W and WWF shapes, improvements were made to the lateral support devices and to the LVDT support frame to eliminate the possibility of rigid body rotation displacements. Minor modifications were

made to the location of displacement and strain measurements program to include the testing of the bridge girders.

2.2 Test Set-up

The test set-up consisted of a frame supporting a 4450 kN (500 Ton) hydraulic actuator, the test specimens supported on knife-edge roller assemblies, Watt mechanisms and lateral support columns, a hydraulic control system, and a data acquisition system for reading the electronic instrumentation. Details, cross-sections and elevations are shown in Figure 2.2, 2.3 and 2.4.

2.2.1 Loading System and Support Details

The vertical load was applied at midspan for the interior tests, through load plates of 100 mm and 200 mm lengths for the WWF 985x71, and the bridge girders, respectively. Additional load plates of 50 mm were used for the end reaction tests of the WWF 985x71. Otherwise, all reactions were taken directly through the knife-edge roller assemblies.

2.2.2 Lateral Support (Watt Mechanisms)

Six Watt mechanisms were built to restrain the test specimens from displacing laterally without restricting movement of the restrained point in the vertical and horizontal directions for maximum distances of 75 mm and 280 mm, respectively, as illustrated in Figure 2.6. These were attached at four locations along the top flange and two on the bottom as shown in Figure 2.5. The Watt mechanisms were designed specifically to be used with the support columns available in the laboratory. Because of this, the lengths of the arms had to be 770 mm each and the centre HHS section a length of 600 mm. Each of the balls in the joints were attached to threaded rods which could be rotated for horizontal adjustments to ensure a tight fit and to eliminate any lateral movement as can be seen in Figure 2.7.

2.2.3 Instrumentation and Measurement

Measurements of the applied loads, displacements, rotations, and strains which describe the behaviour of the test specimen, were recorded intermittently during the test using LABCON and a Hewlett Packard data acquisition system. These data were evaluated and are presented graphically in Chapter 3 in a form that is readily comparable to the load versus out-of-plane web displacements and principal stresses determined from non-linear finite element analyses of the test specimens.

A total of 62 electrical resistant strain gauges were used to measure strains on the flanges and webs of the WWF 985x71 and 76 strain gauges on the bridge girders. Four gauges were placed at approximately $b/3$ from the edge of the top flange of each beam and girder on either side of the load plate, with four more gauges mirroring them on the underside, providing sufficient data to determine the in-plane force due to bending and any localized bending of the top flange during testing. Two gauges were placed on both sides of the bottom flange at midspan to determine the in-plane tensile force due to bending. These locations and numbering sequences are illustrated in Figure 2.8. The locations of the strain gauge rosettes on the web and the corresponding numbering sequence are illustrated in Figures 2.9, 2.10 (a) and (b), and 2.11(a) and (b), for the WWF 985x71; the bridge girders with the longitudinal stiffener located closest to the loaded flange; and the WWF 1850x273 (b), respectively. The locations of the rosettes were based on experience from previous tests by Benichou (1994) and Kirkhope (1995) but modified to account for the longitudinal stiffener. Ten rosettes were placed on either side of the web for the WWF 985x71; 13 for the bridge girders WWF 1450x247, WWF 1850x273 (a), and WWF 1850x257 with nine within the short panel closest to the load plate; and 13 for the WWF 1850x273 (b) all within the panel closest to the load plate.

Micro-measurement high elongation strain gauges with a resistance of $120.0 \pm 0.15\%$ ohms, a gauge factor of $2.055 \pm 0.5\%$ at 24°C and a gauge length of 8 mm. The

flanges were gauged with Showa strain gauges with a resistance of $119.9 \pm 0.5\%$ ohms were used on both sides of the specimens, a gauge factor of $2.12 \pm 1.0\%$ at 20°C and a gauge length of 5 mm.

Displacements were measured by Linear Variable Differential Transformers (LVDTs). Again, as essentially two different specimens were being tested, the WWF 985x71 shapes and the bridge girders, two different LVDT layouts had to be chosen. These particular layouts allow monitoring of the effect of the longitudinal stiffener on the buckled shape of the web, the deflected shapes of the webs, and any rotation and displacement of the flanges along the centre or at the ends.

Like the strain gauges, the positioning of the LVDTs was based on data collected in similar previous tests. For the testing of the WWF 985x71 shapes, 21 LVDTs were used to map displacements in all directions and any rotations. Because of the sizes of the bridge girders, 21 LVDTs were needed to map the web alone, and 30 LVDTs were used in total. The layouts for each of the test types are in Figures 2.12, 2.13, and 2.14. On all tests, four LVDTs were used to measure rotation at both ends of the beams, three LVDTs to measure the vertical displacements of the bottom flange, and two measured the rotation and vertical displacement of the top flange around the load plate (centre of the beam) as seen in Figure 2.15. For the two end reaction tests of the WWF 985x71 shapes, the two

LVDTs on the top flange were moved to the end of the beam to measure the rotation there.

The load was controlled and monitored using a calibrated pressure transducer on the pressure line attached to the hydraulic actuator and the LABCON program. The pressure transducer actuator combination was calibrated in the 1780 kN Tinius Olsen testing machine with an MTS controller. The pressure between the electric pump and the solenoid valve was adjusted to minimize the pressure difference across the valve to provide adequate control of the load steps. For the WWF 985x71 shapes an Enerpac handpump was used to complete the test as the electric pump, solenoid valve, hydraulic control system return loop was too slow and too coarse to give small load increments. At best, the load with the electric pump could be controlled to ± 10 kN.

At the end of each test, additional observations and measurements were taken to verify the data collected.

2.3 Alignment and Test Procedure

2.3.1 Alignment and Initial Loading

The flange-web junction at midspan of the test specimen was centered underneath the actuator, and the longitudinal alignment adjusted to centre the specimen between the lateral support columns. The Watt mechanisms and the corresponding load plate was centered under the actuator. All load and reaction plates were leveled with dental plaster to ensure uniform bearing on the flanges. The LVDT support frames and LVDTs were attached and positioned as shown previously. The electronic instrumentation was connected to the data acquisition system and checked to ensure that it was functional. Subsequently, a small preload was applied to seat the test specimen. The load was removed just enough to maintain contact between the actuator and the load plate, and the keeper plates for the knife-edge roller assemblies were removed.

2.3.2 Test Procedure

The test specimens were loaded monotonically until failure using the following loading sequence:

1. initial readings were taken at zero load with the actuator just in contact with the load plate;
2. the first load increment of approximately 15 kN was applied;
3. all electronic measurements were taken and the readings scanned to ensure the complete operation of the system;
4. additional load was applied in appropriate increments as determined by the load deflection curve and the behaviour of the test specimen; observations were made and recorded with respect to deflection, yielding and yield patterns;

After failure, photographs were taken to document the failure and any additional observations or measurements were recorded in the experimental logbook. The specimen was unloaded and the residual out-of-plane deformations were recorded.

2.4 Ancillary Tests

Ten tension coupons were cut with a bandsaw from the flange and web plates of each of the specimens as shown in Figure 2.16, with the exception of the WWF 985x71 shapes which were taken from a 500 mm long section from the locations shown in Figure 2.17. These were milled and tested according to ASTM Standard E8M-91 (1991) using a gauge length of 200 mm and a width of 40 mm. A typical tension coupon is also shown in

Figure 2.16. The cross-sectional dimensions of the reduced section were determined by taking ten measurements of the thicknesses and widths along this length with a digital micrometer. Each coupon had a pair of strain gauges, one mounted on either side, in the longitudinal direction. Two additional transverse strain gauges were attached on coupons 'L2' and 'L4' (or 'T2' and 'T4'), for the bridge girders (see Figure 2.16 for numbering) and coupons '5' and '7' for the WWF 985x71 shapes (see Figure 2.17 for numbering). Larger strains were measured mechanically with dividers and a scale and manually entered into a computer file later. All tension coupons were tested in a 1780 kN Tinius Olsen testing machine. The stress/strain data derived from these measurements was then used subsequently to determine the material properties for each steel plate.

	b/w ratio	L	b	h	t	w	a	longitudinal stiffener	transverse stiffener	Elastic Buckling Load (kN) Prabha (1996)
WWF 1500 x 247 (Girder 175)	175	5500	400	1400	25.4	8	1700	10X100	10X100	1397
WWF 1850x273 (Girder 225)	225	6000	400	1800	25.4	8	1900	10X100	12X100	1146
WWF 1850x257 (Girder 285)	285	7000	420	1800	25.4	6.4	2300	12.7 X100	16X120	549
WWF 985x71 (double and single weld)	196	2300	200	980	10	5	1000	N/A	75x50x5	233

Notes: 1. All dimensions are in millimeters

2. The bridge girders were designed using $F_y = 250$ MPa

3. 5 mm continuous welds were required throughout

Table 2.1 Nominal Geometric Properties for WWF 985x71 and Bridge Girders

(a) Reference axis

(b) End View of Specimen and Property Definitions

Figure 2.1 Geometric Property Definitions and Reference Orientation

Figure 2.2 Schematic Diagram of Test Set-up

Figure 2.3 Cross-Section of Reaction Frame

Figure 2.4 View of Lateral Support System

Figure 2.5 Plan View of Watt Mechanisms and Test Specimen

Figure 2.6 Typical Watt Mechanism

Figure 2.7 Schematic Diagram of a Typical Watt Mechanism

(a) Strain Gauge Layout for Flanges (all test set-ups)

(b) Numbering sequence used for flanges on all tests

Figure 2.8 Strain Gauge Locations and Numbering for all Flanges

(a) Rosette Layout

Note: Even numbers are on other side of the web

(b) Strain Gauge Numbers

Figure 2.9 Strain Gauge Locations and Numbering Sequence for WWF 985x71 Webs

Figure 2.10 (a) Rosette Layout for Bridge Girders WWF 1450x247, WWF 1850x273a WWF 1850x257

Figure 2.10 (b) Rosette Numbering for Bridge Girders WWF 1450x247, WWF 1850x273a, WWF 1850x257

Figure 2.11 (a) Rosette Layout for Bridge Girder WWF 1850x273 (inverted)

Figure 2.11 (b) Rosette Numbering for Bridge Girder WWF 1850x273 (inverted)

Note : $c/2 = (50 \text{ mm})$

(a) LVDT locations on Web

(b) LVDT Numbering

Figure 2.12 LVDT Layout and Numbering Sequence for WWF 985x71

(a) LVDT Locations on Web

(b) LVDT Numbering

Figure 2.13 LVDT Layout and Numbering for Bridge Girders WWF 1450x247, WWF 1850x273a, WWF 1850x257

(a) LVDT Locations on Web

(b) LVDT Numbering

Figure 2.14 LVDT Layout and Numbering for Bridge Girder WWF 1850x273b

(a) Numbering Sequence and Set-Up for WWF 985x71

(b) Numbering Sequence and LVDT Set-Up for Bridge Girders

Figure 2.15 Numbering Sequence and LVDT Set-Up for All Shapes

Figure 2.16 Tension Coupon Definition

Figure 2.17 Tension Coupon Identification for WWF Sections

Chapter 3

Test Results

3.1 Geometric Properties

The cross-sectional dimensions, defined in Figure 3.1 (a), were measured and are given in Table 3.1 in terms of the mean values and coefficients of variation. According to CAN/CSA G40.20-M87, which defines allowable fabrication tolerances, the tilt of the flanges must be less than 6 mm for a web depth over 300 mm. The warping of the flanges was determined by measuring the depths (d_1 and d_2) from flange tip to flange tip and comparing them with the depth at the web. The largest average difference was found to be approximately 9.5 mm for WWF 985x71 (b) with two fillet welds. The average measured tilt for each flange (top and bottom) is 4.75 mm, which is less than the 6 mm tolerance.

The out-of-straightness for the web plates of each shape was determined at seven locations along the length of the WWF girder from measurements made using the imperfection measuring device illustrated in Figure 3.1 (b). The average out-of-

straightness given in millimeters at depths of $0.1h$, $0.3h$, $0.5h$, $0.7h$ and $0.9h$, where the distance to the point on the web is measured from the top of the top flange, and the average maximum allowable out-of-straightness (web flatness) specified in CAN/CSA G40.20-M87 of $1/150$ of the shape depth are given in Table 3.2. With the exception of the WWF 985x71 sections which exceeded this value by two millimeters, all others meet this fabrication tolerance criteria. This excessive out-of-straightness for the WWF 985x71 section was the result of imperfections induced during fabrication that were not corrected. As for the bridge girders, the largest out-of-plane imperfection was 9.9 mm over 1800 mm, which is 0.825 times the allowable tolerance.

The measured geometric properties and out-of-straightness web plate imperfections are to be used in supporting finite element analyses, the determination of test to predicted bearing resistance ratios from loads predicted using the proposed Clause 15.9 (based on the equivalent column concept), and the corresponding statistical analysis for determining the appropriate resistance factor.

3.2 Material Properties

A representative sample of tension coupons cut from the webs and flanges of each of the test specimens were tested in accordance with ASTM standards E8M-91 (1991) to determine the material properties of the test specimens as specified in section 2.4. Eleven

coupons, six from the flanges and five from the web, were taken from the WWF 985x71 section (as shown in Figure 2.17); ten coupons from each web and flange plate used to fabricate the bridge girders (five coupons aligned and five coupons perpendicular to the rolling direction); and five coupons from each stiffener plate thickness (aligned in the rolling direction). The steel grade for all plates were specified as 350W with the exception of the web plate for the WWF 1800x257 which was made with 300W grade steel as a 350W plate of that size was not available.

The stress-strain curves for coupons can be characterized by the two typical curves illustrated in Figure 3.2. The corresponding material properties evaluated from these curves are listed in Tables 3.3 to 3.11 for each plate. Curve Type I was the most common but the 6.4 mm plate which consistently exhibited Curve Type II. The characteristic material properties include the modulus of elasticity, yield strength, static yield stress, yield strain, ultimate strength, ultimate strain, final stress and final strain. The final stress was evaluated using the natural log of the original area divided by the necked down area, and the final strain was calculated using the final length divided by the original length.

The measured material properties were used as input for the strain data interpretation program, Benichou (1994), to calculate the principal stresses and directions and are to be used subsequently in the determination of test-to-predicted bearing resistance and the corresponding resistance factor.

3.3 Web Behaviour

Out-of-plane web displacements, flange rotations about the longitudinal axis of the beam and strains were measured at discrete locations (refer to section 2.2) to obtain a description of the behaviour of the slender webs of the WWF 985x71's and the bridge girders when subjected to concentrated in-plane loads. The data are presented in two sections entitled load versus out-of-plane web displacements and strain distributions. Graphs illustrated the load versus out-of-plane web displacements for the LVDT with the maximum displacement; the deflected shape through the depth at the centre of the load and at the edge of the load plate for the ultimate load and additional load levels where necessary. Figures give principal strains and orientations on both sides of each web for load levels coinciding with the progression of yielding. The strain distributions for the WWF 985x71 test are given in Prabha (1996).

A summary of the tests gives information about the location and number of lateral supports (Watt mechanisms), the failure load and mode, the maximum out-of-plane web displacement and location, and other relevant observations or comments are given in Table 3.12.

3.3.1 Load vs. Out-of-Plane Web Behaviour

The characteristic behaviour of the web is best described by load versus out-of-plane web displacement curves which are illustrated for the WWF 985x71 (a) and (b) sections for interior loads and end reactions in Figure 3.3 to 3.6, and for the bridge girders in Figures 3.8 to 3.11. The displacements shown are those of the LVDTs which give the maximum out-of-plane displacements. The location of that LVDT with respect to the loaded flange, and key load levels that coincide with the progression of yielding, are also indicated. The initial slope gives an indication of the effects of the initial out-of-straightness of the web and is large for small imperfections as can be seen, for example, in the upper panel of the WWF 1450x247 in Figure 3.8. The change in slope represents the reduction in stiffness which results from either the formation of yield bands in the web of the upper panel under consideration, or in the adjacent panel which provides support (as illustrated for the WWF 1800x273 (a) in Figure 3.9), or from second order effects of the load acting on the deformed web that has significant out-of-plane displacements of the order of magnitude of the web thickness, as illustrated for the end reaction tests for the WWF 985x71, in Figures 3.5 and 3.6, where the web was still elastic. There is also the possibility that a combination of both occurred.

The location of the maximum out-of-plane displacement of the web for the WWF 985x71 sections was found to be at about $0.25h$ from the loaded flange for both interior

and end reaction tests. For the bridge girders, the maximum displacements occurred at approximately $0.1d_1$ and $0.4d_2$ of the top and bottom panels, respectively, with the exception of the inverted WWF 1800x273 (b), where it occurred at $0.6d_2$, (see Figures 2.13 and 2.14 for layout).

All tests failed by inelastic web buckling. For the WWF 985x71 section tests, the flange and surrounding web offered little or no post-buckling restraint, and load decreased soon after the ultimate load was reached without further appreciable web displacements. The post buckling restraint provided to the upper panel for the WWF 1450x247, WWF 1850x273, and WWF 1850x257 is evident from the long plateau at the ultimate load level of the load deformation curve for this panel. The erratic displacement readings illustrated for LVDT #2 in Figure 3.10 (a) and 3.11 (a), are associated with very small out-of-plane web displacements that are masked due to the LVDT tip sliding over the web as the web height decreases. Hand measurements show this vertical decrement to be as much as 5 mm. The vertical flange tip displacements for the WWF 985x71 (b), given in Figure 3.7, have been included for completeness of test data presentation and are to be used elsewhere for comparison with the results of the corresponding finite element analysis, Prabha (1996).

The displaced shapes of the web directly under the middle of the loaded length and at the edge of the loaded length, are drawn from the displacements measured by the

LVDTs at ultimate load for WWF 985x71 (a) and (b) in Figures 3.12 and 3.13, and for the WWF 1450x247 and WWF 1850x257 in Figures 3.14 and 3.16. The displaced shapes of the web for WWF 1850x273 (a) and (b) are given at a series of load levels as yielding progresses, in Figure 3.15 and 3.16. For the bridge girders, the location of the longitudinal stiffener is indicated. The displaced shapes for the WWF 985x71 sections are compared with finite element results by Prabha (1996). The displaced shapes for the bridge girders clearly indicate that the longitudinal stiffener restrains the web from the lateral displacement but not from rotation. The webs on either side behave similarly to two columns with pinned-fixed end conditions that buckle interactively. For the WWF 1850x273 (b), the longitudinal stiffener effectively reduces the web depth to $0.8h$. The displaced shapes give an indication of restraint provided to the web plate within the equivalent column width from the flanges and surrounding web plate. These displaced shapes are to be used as a comparison for the finite element results and for further development and adaptation of the equivalent column concept and design equations for slender webs of bridge girders with longitudinal stiffeners. Based on hand measurements of displacements of the web, it was concluded that the LVDT support frame functions as designed and effectively eliminated any misleading displacements that occurred previously due to flange rotation (see Benichou (1994) and Kirkhope (1995)).

3.3.2 Web Strain Distribution

All the measured strain data were converted into principal strains with their corresponding orientations, by using the strain interpretation program developed by Benichou (1994) and is illustrated at each discrete location for either side of the web, at different load levels as yielding progressed (as indicated by the load deflection curves), for the WWF 985x71 sections in Figure 3.18 (a) and (b) and 3.19 (a) and (b); and for the bridge girders in Figures 3.20 to 3.23. Three values appear beside each rosette location indicated by the symbol 'T' which graphically illustrates the orientation of the principal strains. The first value corresponds to the principal strain that is most closely aligned with the bar or flange of the 'T', the second to the stem of the 'T' and the third value in brackets gives the orientation of the stem in degrees measured positively counter-clockwise from the vertical. For the WWF 985x71 sections, the diagrams are marked 'near side' and 'far side' where 'near side' indicates the side that buckled out of the page. For the bridge girders, the buckled shape and location of the longitudinal stiffener are shown. The yield strain determined for the web from the corresponding tension coupon tests is also indicated on the figures. Note, the web is biaxially stressed and yielding at a given location is loosely interpreted as the exceedance of the uniaxial yield strain value shown. Strain gradients through the web indicate out-of-plane bending and are easily identified by comparing values on either side of the web. Suspect strain data is indicated by '**' on the appropriate figure. These data are not used any further.

Transformation of the inelastic strain data into stresses and the comparisons with the finite element analyses for the WWF 985x71's is discussed in Prabha (1996) and is not discussed further.

As generally illustrated in Figures 3.21 to 3.23, the orientation of the principal stresses are mirrored on opposite sides of the web as would be expected. A comparison of the principal strains with the yield strains at all loads gives an indication of the formation and location of the yield bands which form a mechanism in the web and define the ultimate load. A summary of this is presented in Table 4.2. A yield band is fully formed when one side is yielded in compression and the other yields in tension as a result of large out-of-plane bending of the web. Until this strain condition is achieved over the entire length of the horizontally oriented yield band on the web near the flange web junction and the circular arc yield band that forms in the web, the web is able to sustain more load with increasing out-of-plane web deformation. Other than stating the obvious, where the magnitude of the strains diminish with the distance from the load point (approaching zero average vertical strain on the web near the unloaded flange along midspan, as would be expected), little else can be said until the finite element comparisons are made.

The displaced buckled shape of the bridge girders indicates that the longitudinal stiffener acts as lateral support creating an inflection point along its length. The rotation of the longitudinal stiffeners is consistent with the buckled shape in which the panel can buckle in either direction. As previously described, the longitudinal stiffener creates two web panels with fixed-pinned end conditions that buckle interactively and are defined by the location of the stiffener.

Section	w		b		t		d		d ₁		d ₂		a ₁	
	mm		mm		mm		mm		mm		mm		mm	
985x71 (a) (single weld)	Average	4.98	199.64	9.70	981.5	981.5	981.5	0.71	981.75	981.75	979.42	979.42	97	97
	Std. Dev.	0.07	0.59	0.16					0.97	0.97	1.24	1.24	1.56	1.56
985x71 (b) double weld	Average	5.26	198.80	9.65	984.5	984.5	984.5	3.53	975.0	975.0	980.46	980.46	96.5	96.5
	Std. Dev.	0.21	0.62	0.10					1.60	1.60	1.56	1.56	1.60	1.60
WWF 1450x247 b/w=175	Average	8.03	400.33	25.59	1450	1450	1450	0.0	1449.06	1449.06	1448.17	1448.17	195.5	195.5
	Std. Dev.	0.35	0.48	0.16					0.54	0.54	0.66	0.66	1.07	1.07
WWF 1850x273 (a) b/w=225	Average	8.16	400.00	25.46	1849.5	1849.5	1849.5	0.71	1848.0	1848.0	1848.9	1848.9	196.13	196.13
	Std. Dev.	0.76	0.56	0.13					0.94	0.94	0.99	0.99	0.83	0.83
WWF 1850x273 (b) b/w=225 (inverted)	Average	8.04	400.17	25.55	1849	1849	1849	0.0	1848.42	1848.42	1848.17	1848.17	195.75	195.75
	Std. Dev.	0.32	0.54	0.08					1.24	1.24	0.83	0.83	0.89	0.89
WWF 1850x257 b/w=285	Average	6.36	419.854	25.57	1849	1849	1849	0.0	1848.08	1848.08	1847.5	1847.5	209.46	209.46
	Std. Dev.	0.54	0.67	0.16					0.90	0.90	0.52	0.52	0.93	0.93

Table 3.1 Cross-Sectional Geometric Properties for Test Specimens

Section Designation	Maximum web plate out-of-straightness at 5 even intervals throughout the depth					
	depth 1 0.1h (mm)	depth 2 0.3h (mm)	depth 3 0.5h (mm)	depth 4 0.7h (mm)	depth 5 0.9h (mm)	max. allow. out-of- straightness (mm)
WWF 985x71 (a) (single weld)	3.97	6.44	7.16	6.12	3.71	6.4
WWF 985x71 (b) double weld	5.31	7.61	8.2	7.11	4.37	6.4
WWF 1450x247 b/w=175	1.29	3.34	3.44	4.28	2.72	9.3
WWF 1850x273 (a) b/w=225	0.3	2.18	5.41	7.56	5.61	12.0
WWF 1850x273 (b) b/w=225 (inverted)	1.74	4.15	5.75	6.28	2.95	12.0
WWF 1850x257 b/w=285	0.68	4.09	8.42	9.9	6.81	12.0

Table 3.2 Measured Web Plate Out-of-Straightness

Table 3.3 Characteristic material properties determined from uniaxial stress strain curves for the WWF 985x71

Coupon No.	E MPa	σ_y MPa	$\sigma_{0.2}$ MPa	σ_u MPa	σ_t MPa	$\epsilon_y \times 10^{-4}$	$\epsilon_u \times 10^{-4}$	$\epsilon_t \times 10^{-4}$	Est MPa
1	213000	387.4	389.3	538.1	478.3	2200	11651	160000	996.9
2	221000	413.8	416.4	570.5	578.0	3923	6438	132000	811.7
3	215000	381.3	372.3	536.8	642.8	2074	10703	170000	968.4
4	228000	386.2	377.0	538.9	520.6	3679	7365	162000	952.8
5	*	395.1	393.5	541.9	513.5	*	*	135000	*
6	208000	400.5	394.5	541.1	537.8	3347	8242	170000	1103.4
7	210000	376.8	365.0	512.6	497.0	3053	3340	185000	1233.7
8	221000	370.7	363.4	516.9	512.4	3430	4166	145000	917.5
9	209000	388.8	378.2	548.8	490.3	2037	3439	176000	1014.4
10	213000	414.5	406.3	596.4	580.2	3707	5434	121000	609.0
11	218000	393.2	367.6	545.3	531.1	4651	5450	160000	869.7

* insufficient data available

Table 3.4 Characteristic material properties determined from uniaxial stress strain curves for the 6.4 mm web plate in the direction longitudinal to rolling.

Coupon No.	E MPa	σ_y MPa	$\epsilon_y \times 10^{-6}$	σ_{ny} MPa	$\epsilon_n \times 10^{-6}$	σ_{un} MPa	$\epsilon_{un} \times 10^{-6}$	σ_f MPa	$\epsilon_f \times 10^{-6}$	E_n MPa
L1	192000	352.1	3996	*	*	503.0	139303	943.6	980735	1115
L2	188000	355.0	3887	*	*	503.5	140000	932.0	1075187	1091
L3	218000	356.1	3214	*	*	509.6	150000	932.3	967750	1045
L4	201000	360.4	3878	*	*	504.9	156000	946.7	999746	951
L5	196000	359.7	3671	*	*	507.0	134000	938.2	748592	1127
Avg.	199000	356.7	3729	*	*	505.6	144000	938.6	954402	1066
Std. dv.	11500	3.4	311	*	*	2.7	12700	6.6	122346	71.4
ν	0.058	0.0096	0.083	*	*	0.0054	0.061	0.022	0.13	0.067

* There was no definite yield plateau evident for any of the specimens, therefore yield stress was calculated using a 0.2% offset

Table 3.5 Characteristic material properties determined from uniaxial stress strain curves for the 6.4 mm web plate in the direction transverse to rolling.

Coupon No.	E MPa	σ_y MPa	ϵ_y $\times 10^{-4}$	σ_y MPa	ϵ_u $\times 10^{-6}$	$\sigma_{0.2}$ MPa	$\epsilon_{0.2}$ $\times 10^{-6}$	σ_f MPa	ϵ_f $\times 10^{-6}$	E_u MPa
T1	200000	359.5	3701	*	*	483.2	150000	908.7	760343	845
T2	190000	340.7	3831	*	*	480.1	144000	908.7	755040	992
T3	194000	355.1	3738	*	*	503.0	150000	946.9	701287	1011
T4	204000	353.7	3913	*	*	498.2	145000	942.0	757897	1024
T5	200000	358.5	3859	*	*	505.0	124000	947.4	668973	1215
Avg.	198000	353.5	3808	*	*	493.9	143000	930.7	728708	1017
Std. dv.	5648	7.5	87	*	*	11.5	10607	20.2	41431	131
ν	0.029	0.021	0.023	*	*	0.023	0.074	0.022	0.057	0.13

* There was no definite yield plateau evident for any of these specimens, therefore yield stress was calculated using a 0.2% offset

Table 3.6 Characteristic material properties determined from uniaxial stress strain curves for the 8 mm web plate in the direction parallel to rolling.

Coupon No.	E MPa	σ_y MPa	ϵ_y $\times 10^{-6}$	σ_{xy} MPa	ϵ_{ux} $\times 10^{-6}$	σ_{ux} MPa	ϵ_{ux} $\times 10^{-6}$	σ_t MPa	ϵ_t $\times 10^{-6}$	E_{xt} MPa
L1	209000	397.0	2028	394.3	10000	539.5	175000	751.5	1080103	840.7
L2	197000	399.4	2215	397.6	13000	542.9	159204	766.2	1127087	934.4
L3	221000	401.6	1913	401.2	13000	543.5	144278	769.5	1060283	1006
L4	205000	401.5	2204	400.2	9500	543.6	164179	768.3	1132542	885.4
L5	204000	401.9	2204	396.2	10940	558.3	175000	769.3	1107385	943.9
Avg.	207400	400.28	2112.8	397.9	11288	545.56	163532.2	764.96	1101480	922.07
Std. dv.	8892	2.08	136.19	2.83	1646.1	7.32	12775.9	7.64	30861	62.52
ν	0.043	0.005	0.065	0.007	0.2	0.013	0.078	0.009	0.028	0.068

Table 3.7 Characteristic material properties determined from uniaxial stress strain curves for the 8 mm web plate in the direction transverse to rolling.

Coupon No.	E MPa	σ_y MPa	$\epsilon_y \times 10^{-6}$	σ_{xy} MPa	$\epsilon_d \times 10^{-6}$	σ_{ux} MPa	$\epsilon_{ux} \times 10^{-6}$	σ_t MPa	$\epsilon_t \times 10^{-6}$	E_a MPa
T1	226000	425.7	2193	422.3	8042	*	*	*	*	*
T2	231000	419.5	1794	415.7	7256	554.5	175000	757.4	726055	810.8
T3	229000	414.2	4364	411.5	12750	548.4	164179	740.1	808202	859.6
T4	223000	418.1	1874	414.1	10000	553.5	165000	758.1	794229	857.5
T5	216000	407.8	2015	400.8	9950	538.9	149253	737.3	766062	938.2
Avg.	225000	417.1	2248.0	412.9	9599.5	548.8	163358	748.2	773637	866.5
Std. dv.	5970	6.6	1081.8	7.8	2128.9	7.1	10612.2	11.1	36240.8	52.8
v	0.027	0.016	0.042	0.019	0.34	0.013	0.065	0.15	0.047	0.061

* Data could not be obtained due to improper specified load rate which caused the specimen to fail too quickly

Table 3.8 Characteristic material properties determined from uniaxial stress strain curves for the 25 mm flange plate in the direction parallel to rolling.

Coupon No.	E MPa	σ_y MPa	$\epsilon_y \times 10^{-6}$	σ_y MPa	$\epsilon_u \times 10^{-6}$	σ_{un} MPa	$\epsilon_{un} \times 10^{-6}$	σ_t MPa	$\epsilon_t \times 10^{-6}$	E_{xt} MPa
T1	209000	399.4	2590	395.3	13826	606.2	135678	231.7	1031412	1626.6
T2	217000	400.4	2466	400.4	12006	612.8	120603	229.9	760501	1854.3
T3	248000	397.5	2665	*	*	604.2	130000	234.6	718017	*
T4	215600	393.1	2583	388.6	9015	604.9	125628	232.2	632147	1800.0
T5	205000	392.8	3805	394.4	11048	600.6	130000	225.8	716864	1663.5
Avg.	218000	396.6	2812.8	394.7	10317.6	605.7	128381.8	230.9	771788.2	1736
Std. dv.	17500	3.5	559.4	4.8	1856.1	4.5	5625	3.3	152429.9	108.5
v	0.080	0.009	0.200	0.012	0.048	0.007	0.044	0.014	0.190	0.063

* T3 had no definite yield plateau

Table 3.9 Characteristic material properties determined from uniaxial stress strain curves for the 25 mm flange plate in the direction transverse to rolling.

Coupon No.	E MPa	σ_y MPa	$\epsilon_y \times 10^{-4}$	σ_y MPa	$\epsilon_y \times 10^{-4}$	σ_{ult} MPa	$\epsilon_{ult} \times 10^{-4}$	σ_t MPa	$\epsilon_t \times 10^{-4}$	E_u MPa
L1	205000	389.9	2087	386.2	9956	594.9	140000	235.0	1277794	1574.9
L2	203000	391.2	2178	392.4	12609	598.6	140000	232.7	829952	1551.9
L3	199000	391.1	2014	389.9	9023	*	*	*	*	*
L4	203000	389.9	1941	391.7	10000	594.2	140000	229.8	1215662	1574.9
L5	209000	385.5	2797	386.8	10000	594.4	140000	230.1	1173808	1563.2
Avg.	203900	389.48	2203.4	389.4	10317.6	595.53	140000	231.90	1124302.7	1566.2
Std. dev.	3634	2.42	343.25	2.81	1347.1	2.07	0.00	2.44	200830.5	11.03
v	0.18	0.006	0.16	0.007	0.064	0.004	0.001	0.011	0.179	0.007

* L3 was unable to test until failure

Table 3.10 Characteristic material properties determined from uniaxial stress strain curves for the 10 mm stiffener plate in the direction parallel to rolling.

Coupon No.	E MPa	σ_y MPa	$\epsilon_y \times 10^{-4}$	σ_{xy} MPa	$\epsilon_{xy} \times 10^{-4}$	σ_{ult} MPa	$\epsilon_{ult} \times 10^{-4}$	σ_t MPa	$\epsilon_t \times 10^{-4}$	E_m MPa
L1	210000	381.0	1795	380.4	13000	533.4	153465	631.1	1024679	1008.7
L2	212000	385.9	1817	383.0	15000	535.6	159204	634.1	957719	969.4
L3	219000	388.7	1773	388.4	14925	537.9	169154	640.6	1003675	893.0
L4	238000	387.4	1642	383.6	12500	534.5	150000	635.3	1001312	1017.3
L5	211000	390.1	1866	387.2	10000	535.9	160000	645.0	1120855	940.4
Avg.	218000	386.6	1778.6	384.5	13085	535.46	158364.6	637.2	1021648	965.7
Std. dev.	11800	3.51	83.78	3.25	2056.7	1.68	7310.3	5.54	60570	51.1
ν	0.054	0.009	0.047	0.008	0.04	0.003	0.046	0.009	0.06	0.05

Table 3.11 Characteristic material properties determined from uniaxial stress strain curves for the 12.7 mm stiffener plate in the direction parallel to rolling.

Coupon No.	E MPa	σ_y MPa	$\epsilon_y \times 10^{-6}$	σ_{xy} MPa	$\epsilon_{xy} \times 10^{-6}$	$\sigma_{\alpha\alpha}$ MPa	$\epsilon_{\alpha\alpha} \times 10^{-6}$	σ_t MPa	$\epsilon_t \times 10^{-6}$	E_{α} MPa
L1	216000	391.5	1820	390.8	15162	542.1	170000	483.3	1009809	899.7
L2	221000	393.7	1782	393.6	11559	544.6	174129	484.3	1020071	876.2
L3	214000	390.4	1817	389.9	11349	540.4	155000	486.6	1024336	982.5
L4	206000	392.5	1894	393.2	13000	541.7	160000	482.0	1067649	939.3
L5	206000	395.6	1829	394.4	19214	538.7	150000	476.5	1095739	970.4
Avg.	212500	392.7	1828.4	392.5	14056.0	541.5	161825.8	482.5	1043520.8	933.6
Std. dv.	68900	2.01	40.8	2.1	3259.8	2.18	10099.0	3.77	36617.4	45.4
ν	0.032	0.005	0.022	0.005	0.021	0.004	0.06	0.007	0.035	0.049

Table 3.12 Test Histories

Test section and Test Type	Maximum Deflection and location	Ultimate Load, kN, and Failure Mode	Observations
WWF 985x71 (a) 1 side welded only interior loading test N=100 mm	11.2 mm at 0.5 h (LVDT #2)	298 kN inelastic web buckling	hand measured deflection approx.: 12mm @ 150 mm below top flange two Watt mechanisms provided sufficient stability no noticeable rotation of the top flange at the ends
WWF 985x71 (a) 1 side welded only end reaction test N=100 mm	55.6 mm at 0.38 h (LVDT #3)	159 kN inelastic web buckling	deflected shape encompasses the complete panel up to the vertical stiffener flange web junction remains perpendicular maximum deflection occurred 390 mm from the bottom flange upper flange started to rotate slightly at 150 kN

Location of deformation in terms of h is measured from the bottom flange.

Failure loads for the end reaction tests are determined using the recorded actuator force and the measured span geometry.

Table 3.12 (cont.) Test Histories

Test section and Test Type	Maximum Deflection and location	Ultimate Load, kN, and Failure Mode	Observations
WWF 985x71 (b) both sides welded interior loading test N=100 mm	12.7 mm at 0.5 h (LVDT #2)	270 kN inelastic web buckling	At 100 kN, minor rotation in the bottom flange at midspan of approximately 0.0006 radians occurred. As the yield bands formed, the maximum deformation crept upwards towards top flange.
WWF 985x71 (b) both sides welded end reaction test N=100 mm	55.0 mm at 0.5 h (LVDT #2)	169 kN inelastic web buckling	Deformation shape is sinusoidal. At 115 kN, the end rotation of the top flange at the end was approximately 0.0249 radians.

Location of deformation in terms of h is in reference to the bottom flange.

Failure loads for the end reaction tests are determined using the recorded actuator force and the measured span geometry.

Table 3.12 (cont.) Test Histories

Test section and Test Type	Maximum Deflection and location	Ultimate Load and Failure Mode	Observations
WWF 1450x247 Girder h/w=175 interior load test	12.3 mm @ LVDT #2 (upper panel) 11.0 mm @ LVDT #7 (lower panel)	1000 kN inelastic web buckling	at 755 kN, longitudinal stiffener starts to displace at 970 kN, compressive yielding started to occur in the upper panel at gauge #3
WWF 1850x 273 (a) Girder h/w=225 interior load test	18.2 mm @ LVDT #2 (upper panel) 11.0 mm @ LVDT #7 (lower panel)	894 kN inelastic web buckling	at 270 kN, top flange began to rotate all 6 Watt mechanisms were in use at 1500mm and 3000 mm either side of centre
WWF 1850x273 (b) Girder h/w=225 (inverted) interior load test	6.4 mm @ LVDT #7 (lower panel)	887 kN inelastic web buckling	the longitudinal stiffener was bent upwards a total of 8 mm at the end of the test. originally the largest displacement was located at the centre of the beam but moved progressively up towards the top flange as the test proceeded all 6 Watt mechanisms were in use at 1500 mm and 3000 mm either side of centre

Location of deformation in terms of h is in reference to the bottom flange.
No end test were performed on bridge girder sections.

Table 3.12 (cont.) Test Histories

Test section and Test Type	Maximum Deflection and location	Ultimate Load and Failure Mode	Observations
WWF 1850x257 Girder h/w=285 interior loading test	14.5 mm @ LVDT #2 (upper panel) 11.0 mm @ LVDT #7 (lower panel)	633 kN, web yielding	At 353 kN, rotation on op flange occurred due to compression. At 480 kN, the web started to exhibit non-linear behaviour.

Location of deformation in terms of h is in reference to the bottom flange.
No end test were performed on bridge girder sections.

(a) Recorded Dimensions and Geometry

(b) Schematic Diagram of Imperfection Measuring Device

Figure 3.1 Definitions of Measurements For Beam Geometry

Figure 3.2 Typical Uniaxial Stress Strain Curves from Tension Coupon Tests

Figure 3.3 Load vs. Maximum Displacement Curve for WWF 985x71 (welded one-side), Interior Load

Figure 3.4 Load vs Maximum Displacement Curve for WWF 985x71 (double-welded), Interior Load

Figure 3.5 Load vs. Out of Plane Web Deformation for WWF 985x71 (a) Endtest (welded one-side)

Figure 3.6 Load vs Out of Plane Web Deformation for WWF 985x71 (a) Endtest (double welded)

(a) Rotation of One Edge of Flange, LVDt #16

(b) Rotation of One Edge of Flange, LVDt #17

Figure 3.7 WWF 985x71 (b) Endtest, Rotation of Unloaded Flange

(a) Load vs. Maximum Displacement Curve for the Upper Panel, (0.2h)

(b) Load vs. Maximum Displacement Curve in the Lower Panel, (0.8h)

Figure 3.9 Load vs. Displacement Curves for WWF 1850x273 (a)

(a) Load vs. Maximum Displacement Curve for the Upper Panel, (0.2h)

(b) Load vs. Maximum Displacement Curve in the Lower Panel, (0.8h)

Figure 3.9 Load vs. Displacement Curves for WWF 1850x273 (a)

(a) Load vs. Displacement Curve at 0.11h

(b) Load vs. Maximum Displacement Curve at 0.5h

Figure 3.10 Load vs. Displacement Curves for WWF 1850x273 (b)

(c) Load vs. Displacement of Curve at 0.16 h

(d) Load vs. Displacement Curve at 0.3 h

Figure 3.10 (cont.) Load vs. Displacement Curves for WWF 1850x273 (b)

2 OF 2

PM-1 3 1/2" x 4" PHOTOGRAPHIC MICROCOPY TARGET
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(e) Load vs. Displacement Curve at 0.2h

Figure 3.10(cont.) Load vs. Displacement Curves for WWF 1850x273 (b)

(a) Load vs. Maximum Displacement Curve in the Upper Panel, (0.2h)

(b) Load vs. Maximum Displacement Curve in the Lower Panel, (0.8h)

Figure 3.11 Load vs. Displacement Curves for WWF 1850x257

(a) Maximum Out-of-Plane Web Deformation at Centreline

(b) Maximum Out-of-Plane Web Deformation at N/2

Figure 3.12 Out-of-Plane Web Deformation for WWF 985x71 (a) (single-welded), Interior Load

(a) Maximum Out-of-Plane Deformation at Centreline

(b) Maximum Out-of-Plane Web Deformation at N/2

Figure 3.13 Out-of-Plane Web Deformation for WWF 985x71 (b) (double-weld), Interior Load

(a) Maximum Out of Plane Web Deformation at Centreline

(b) Maximum Out of Plane Web Deformation at N/2

Figure 3.14 Out-of-Plane Web Deformation for WWF 1450x247

(a) Centreline Web Displacements at Different Load Levels

(b) Web Displacements at N/2 at Different Load Levels

Figure 3.15 Out-of-Plane Web Deformations for WWF 1850x273 (a)

(a) Centreline Web Displacements at Different Load Levels

(b) Web Displacement at N/2 at Different Load Levels

Figure 3.16 Out-of-Plane Web Deformation for WWF 1850x273 (b)

(a) Maximum Out of Plane Deformation at Centreline

(b) Maximum Out of Plane Deformation at N/2

Figure 3.17 Out-of-Plane Web Deformation for WWF 1850x257

Figure 3.18 (a) Strain Distribution for WWF 98.5x71 (a) (single-welded), 281 kN

Figure 3.18 (b) Strain Distribution at Ultimate for WWF 985x71 (a) (single-welded), 297 kN

Figure 3.19 (b) Strain Distribution at Ultimate for WWF 98Sx71 (b) (double-welded), 271 kN

Figure 3.20 (a) Strain Distribution and Deflected Shape for WWF 1450x247, 537 kN

Figure 3.20 (b) Strain Distribution and Deflected Shape for WWF 1450x247, 824 kN

.. Parabolic Reading Case No. 100

Figure 3.21 (a) Strain Distribution and Deflected Shape for WWF 1850x273 (a), 377 kN

Figure 3.21 (c) Strain Distribution at Ultimate and Deflected Shape for WWF 1850x273 (a), 894 kN

Figure 3.22 (a) Strain Distribution and Deflected Shape for WWF 1850x273 (b), 842 kN

Figure 3.22 (b) Strain Distribution at Ultimate and Deflected Shape for WWF 1850x273 (b), 887 kN

Figure 3.23 (a) Strain Distribution at

Figure 3.23 (b) Strain Distribution at Ultimate and Deflected Shape for WWF 1850x257, 633 kN

Chapter 4

Discussion and Application of Equivalent Column Concept

4.1 Equivalent Column Concept

To extend the application of the equivalent column concept and the proposed design equations to slender, customized WWF shapes and bridge girders with longitudinal stiffeners it is necessary:

1. to establish the probable range of shapes that are being built;
2. to test a representative sample to obtain test data related to internal strains and deformed shapes, material data, geometries, initial imperfections, failure modes and mechanisms and make the corresponding observations;
3. to interpret and present the data in a suitable format for comparison to finite element analysis;
4. to conduct and verify the finite element results with the test data;
5. to prepare a description of stresses and displacements for the finite element analyses to develop further the proper rational model. These descriptions should include the

effective width which is a function of the extent of yielding, a description of the distribution of internal forces, the deflected shapes, and the failure modes.

The scope of this thesis is limited to the presentation of data for comparison with the finite element analyses and any related observations that may be useful in the continued development of the equivalent column concept. The principal stress data for the WWF 985x71 tests and their general behaviour is presented in the finite element comparison by Prabha (1996) and is not given here.

4.2 Stress Distribution

Sixty-four strain gauges arranged in 26 rosettes on both sides of the web girders were used to determine the principal strains and their orientation at thirteen discrete locations. These data were then converted to stresses using a strain data (elastic and inelastic) interpretation program developed by Benichou (1994), and the corresponding material data from the tension coupon tests. First, the principal strains and orientation are determined from the measured strains using Mohr's circle for strains. These strains are then converted to principal stresses by the use of a bi-linear stress strain curve and traditional plasticity algorithms for finite element analysis. The bi-linear curve is defined by the modulus of elasticity, the yield strain calculated by dividing the yield stress by the modulus of elasticity and the strain hardening strain.

The characteristic material properties used to convert the strain data are given in Table 4.1. These same properties will be used in future finite element analyses.

Figures 4.1, 4.2, 4.3 and 4.4 show the principal stresses and orientation on each side of the web at load levels associated with the end of the elastic range, in some cases an intermediate load level, and at ultimate for the bridge girders, the WWF 1450x247, the WWF1850x273 (a) and WWF 1850x273 (b), and the WWF 1850x257, respectively. Also shown are the yield stress, the location of the longitudinal stiffener, the corresponding location of the load and the deflected shape.

The average of the principal stresses on the two sides, most closely aligned with the vertical direction at any load level, diminishes through the depth and with the horizontal distance from the centreline. The following particular observations are drawn from Figure 4.2 (a) to (c) for WWF 1850x273 (a) and are equally applicable to the other girders with the exception of the WWF 1850x273 (b) in which the load was applied on the flange located at $0.8h$ from the longitudinal stiffener. These exceptions will be noted where they occur.

The progression of yielding for all bridge girders is also shown in the strain distribution Figures 4.1 to 4.4, with one of the diagrams indicating the approximate

locations of yield bands as determined from measured strains. Table 4.2 presents the readings for specific gauges and specific load levels as yielding progressed.

For WWF 1850x273 (a), initial yielding began at a distance of $0.1h$ from the top flange (mid-height of the upper web panel) followed by yielding in the region near the fillet on the web. Yielding then propagated outward from the first yield band forming an arc about the load, extending as far as $N+16t$. With increasing out-of-plane deformation of the order of magnitude, t , the initial yield band propagated through the thickness of the web forming a progressive collapse mechanism at or just below the ultimate load.

The WWF 1850x273 (b) exhibits the same characteristic with the exception that the initial yield band forms at $0.05h$, the second forms at $0.15h$ and the yielding extends left and right to a value of at least $N+24t$, but lack of other strain gauge data prevents the exact determination of the distance. Based on the limited data, yield bands need not be fully formed (i.e. a true mechanism) at ultimate. The principal stresses indicate a series of waves forming along the length of the webs alternating from positive to negative to positive.

Prabha (1996) found that the general shape of the average normal stress distribution, at the ultimate load level, is similar to the shape of the normal stress distribution evaluated over any width within the effective column width. From the data available the average normal stress distributions were evaluated along the centerline for

each bridge girder and plotted in Figure 4.5 for the maximum load. These average normal stress distributions at each load level, determined from the strains measured on the surface, were calculated from the normal stress distribution through the thickness at the ultimate load by assuming a linear strain distribution through the thickness and a bi-linear stress strain curve. Also shown in Figure 4.5, is the best fit power function of the form of $\sigma = \sigma_{max}(z/h)^p$ for each distribution. The stress distribution for the WWF 1450x247, the WWF 1850x273 (a) and the WWF 1850x273 (b), are best described by a cubic function and the WWF 1850x257 by a ninth power function. The normal stress distribution does not seem to be influenced by the presence of the longitudinal stiffener. It has yet to be determined whether the longitudinal stiffener extends the effective column width or not. As Prabha (1996) showed for the WWF shapes, these stress distributions could be used in conjunction with the aspect ratio of the equivalent column concept to determine the plate buckling coefficient, c .

4.3 Deflected Shapes

The out-of-plane web displacements are illustrated along the length and through the depth for the WWF 1450x247, the WWF 1850x273 (a) and 1850x273 (b), and the WWF 1850x257, in Figures 4.6, 4.7, 4.8, and 4.9 respectively. Level 1 and Level 4 are those displacements measured by LVDTs 1, 10 and 16 and LVDTs 6, 15, and 21, respectively, whereas Levels 2 and 3 are constructed from four LVDT measurements

numbering 2, 11, 17 and 19 and then 4, 13, 18 and 20 respectively. Also shown in these figures is the assumed effective column half-width of $(N+10t)/2$ as determined by Prabha (1996).

The characteristic deflected shape gives an indication of the boundary conditions of the web plate within the effective column width. As indicated by the displaced shape of each girder at midspan, the top flange was restrained from lateral displacement and rotation about the longitudinal axis by the load and in fact approached fixed support conditions. The bottom flange was restrained against lateral displacement and rotation by the lateral and torsional stiffness of the flange and again approached fixed support conditions. The web was restrained against lateral displacement by the longitudinal stiffener. The rotational stiffness that the longitudinal stiffener provides to the web is very small, and the web plate may be treated as being simply supported by the stiffener.

The out-of-plane web displacements along the length of the web for the upper and lower panels indicate that the boundary conditions along the assumed column width and the displacement functions for the effective column width are difficult to define. Hence, the application of a classical solution to this type of buckled shape may be difficult.

In the application of the effective column concept it is necessary to calculate the buckling coefficient, c , which is a function of the normal stress distribution, boundary conditions and the deflected shape. The latter two are based on experimental evidence

and require care in their application. It is possible that the buckling coefficient may only be obtained from a verified finite element analysis as indeed was the case in Prabha's (1996) solution. It is also possible that a buckling coefficient and load may be determined for each panel and then combined to determine the interactive buckling capacity.

The predicted bearing resistance for the girders, as calculated from the proposed Clause 15.9 by Prabha, (1996), are shown in Table 4.3. The mean value of the test-to-predicted ratio of 0.817, which is considerably different from 1.000, and the large value of the coefficient of variation of about 0.18, indicate that the direct application of this proposed clause is inappropriate. This was not unexpected as the proposed clause, (Clause 15.9), is tailored for the effective length, normal stress distribution, deflected shape and boundary conditions characteristic of WWF shapes. Modifications to Clause 15.9 need to be made to cover slender built-up sections.

Section	E	σ_y	ϵ_y	$\sigma_{0.2}$	ϵ_u	σ_{uL}	ϵ_{uL}	σ_f	ϵ_f	E_m
WWF 1450x247 h/w=175										
WWF 1850x273 (a) h/w=225	225000	417.1	0.001835	412.9	0.01	548.8	0.163	748.2	0.774	866.5
WWF 1850x273 (b) h/w=225										
WWF 1850x257 h/w=285	199000	356.7 ¹	0.001792 ²	*	0.012 ²	505.6	0.0144	938.6	0.954	1066

* Value not available

¹ The yield stress was determined by the 0.2% offset method, as the curves did not exhibit any definite yield plateau

² Value used in strain interpretation program determined graphically

Note: All stresses are in MPa

All strains are in mm/mm

Table 4.1 Material Properties used in Strain Interpretation Program

Table 4.2 Formation of Yield Bands on Webs of Bridge Girders

Section	WWF 1450x247 b/w=175	WWF 1850x273 (a) b/w=225	WWF 1850x273 (b) b/w=225 (inverted)	WWF 1850x257 b/w=285
yield strain	2248 microstrain	2248 microstrain	2248 microstrain	3729 microstrain
<div> <div>Progression of yielding</div> <div>with increasing load</div> <div>→</div> </div>	gauge #8 reaches yield P = 701.65 kN @ .70P _y	gauge #11 reaches yield P = 807.5 kN @ .90P _y	gauge #8 reaches yield P = 569.7 kN @ .64P _y gauge #7 reaches yield P = 788.25 kN @ .89P _y	gauge #3 reaches yield P = 589.2 kN @ .93P _y
	gauge #9 reaches yield P = 884.6 kN @ .87P _y	gauge #4 reaches yield P = 867.2 kN @ .97P _y	gauge #15 reaches yield P = 827.88 kN @ .93P _y	gauge #12 reaches yield P = 618.3 kN @ .97P _y
	gauge #12 reaches yield P = 898.1 kN @ .90P _y	gauge #12 reaches yield P = 885.36 @ .99P _y	gauge #16 reaches yield P = 877.3 kN @ .98P _y	
	gauge #3 reaches yield P = 989.29 kN @ .99P _y		gauge #5 reaches yield P = 881.3 kN @ .99P _y	
Ultimate Load	998.4 kN	893.8 kN	887.2 kN	633.4 kN

*gauges #3 and #4 are 25mm below the bottom of the top flange

*gauges #5, #7 and #8 are at 0.05 b

*gauges #9, #11 and #12 are at .1b

*gauges #15 and #16 are at .15b

specimen	Ultimate Load kN	Br kN	Test-to-predicted ratio
WWF 1450x247	998	1033	0.966
WWF 1850x273a	894	1070	0.836
WWF 1850x273b	887	1036	0.856
WWF 1850x257	633	1034	0.612
Mean			0.817
Standard Deviation			0.148
v			0.182

Table 4.3 Test-to-Predicted Ratio Using Proposed Clause 15.9

Figure 4.1(a) Stress Distribution and Deflected Shape for WWF 1450x247, 537 kN

Figure 4.1(c) Stress Distribution at Ultimate and Deflected Shape for WWF 1450x247, 998 kN

Figure 4.1 (d) Yield Bands at Ultimate and Deflected Shape for WWF 1450x247, 998 kN

Figure 4.2 (b) Stress Distribution and Deflected Shape for WWF 1850x273 (a), 746 kN

Figure 4.2 (d) Yield Bands at Ultimate and Deflected Shape for WWF 1850x273 (a), 894 kN

Figure 4.3 (a) Stress Distribution and Deflected Shape for WWF 1850x273 (b), 842 kN

Figure 4.3 (c) Yield Bands at Ultimate and Deflected Shape for W1850x273 (b), 894 kN

Figure 4.4(a) Stress Distribution and Deflected Shape for WWF 1850x257, 557 kN

Figure 4.4 (b) Stress Distribution at Ultimate and Deflected Shape for WWF 1850x257.633 kN

Figure 4.4 (c) Yield Bands at Ultimate and Deflected Shape for WWF 1850x257, 633 kN

Figure 4.5 Normal Stress Distribution and Fitted Curves for Bridge Girders

Figure 4.6 Out of Plane Web Displacements at Various Levels for WWF 1450x247

Figure 4.7 Out of Plane Web Displacements at Various Levels for WWF 1850x273 (a)

• Displacements taken at Ultimate

Figure 4.8 Out of Plane Web Displacements at Various Levels for WWF 1850x273 (h)

Chapter 5

Conclusions, Summary and Recommendations

5.1 Conclusions and Summary

Over 660 tests world wide have been carried out to determine the bearing resistance of webs under concentrated loads. Few of these tests are representative of the shapes currently being manufactured in North America. As well, the loaded lengths used in many cases were too small and led to empirical equations that did not give a clear physical interpretation of the behaviour nor a unified approach to determine the bearing resistance. The three studies at Carleton University (Benichou, 1994, Kirkhope, 1995, and Prabha, 1996) covered the range of W and WWF shape produced by Canadian steel mills and have led to the development of the equivalent concept and appropriate design equations. The mean test-to-predicted ratios and coefficients of variation were 1.064 and 0.073 for WWF shapes with interior loads and 1.012 and 0.053 for end reaction tests; and 1.002 and 0.029 for W shapes with interior loads and 1.171 and 0.205 for end reaction tests. A statistical evaluation of the test data and the test-to-predicted ratios determined a value for the resistance factor to be 0.9, which resulted in a reliability index of 3.0 or more for each shape.

This study advances the work by providing an experimental data base of a representative sample of bridge girders and customized built-up shapes, with and without longitudinal stiffeners. The bearing resistance of the webs of such girders is of prime importance for the design and construction of bridge girders. The data will be used to verify finite element models and for the continued development and adaptation of the equivalent column concept.

Four tests were completed on thin-webbed bridge girders with longitudinal stiffeners subjected to concentrated in-plane loads at midspan. Two tests on the WWF 1850x273 bridge girders, with the longitudinal stiffeners at $0.2h$ and $0.8h$, model the critical load condition on the positive and negative moment regions of a bridge girder that result during launching. Tests were done on two thin-webbed WWF 985x71 shapes to investigate the behaviour due to concentrated interior loads and end reactions. One of the sections had one-sided web-to-flange welds and the other had two-sided welds. The test data has been processed and presented in such a form as to enable the precise modeling of the specimens by finite element analyses to be carried out; Prabha (1996).

To conduct these tests, six Watt mechanisms were designed and fabricated to provide lateral restraint to the beam while allowing in-plane displacements to occur. Ancillary tests include fifty-one tension coupon tests to determine the material properties for all plates. All relevant geometry and initial imperfections were measured. With

this information, finite element modeling and statistical evaluation can be carried out as the next phase of this program.

In summary,

1. all tests failed by inelastic web buckling;
2. the longitudinal stiffener provides lateral support and creates a nodal line resulting in two web panels behaving essentially as though having fixed-pinned boundary conditions that buckle interactively;
3. measurements of the deformed shape indicate that the out-of-plane web displacements extend beyond the $N+10t$ equivalent column width range. This had been established previously by Kirkhope (1995) and Prabha (1996);
4. preliminary calculations of the normal stresses distribution show the shape of the normal stress distribution with depth are a function of $(z/h)^p$ where p is equal to 3 for most bridge girders but equal to 9 for WWF 1850x257;
5. the presence of the longitudinal stiffener appears to have a localized effect on the normal stress distribution. The maximum normal stress is found to be σ_y at the loaded

flange and zero at the bottom flange. This distribution is similar to tests without a longitudinal stiffener.

5.2 Further Work

1. Finite element analyses of the bridge girders that take into account all experimental data should be carried out to replicate the tests, in so far as possible. The finite element mesh should be sufficiently refined (representative of local yielding phenomena), to verify the modeling procedure. Subsequently, the finite element analyses can be applied with confidence to other boundary and loading conditions to develop the equivalent column concept for these shapes, along with any adaptations that may be required;
2. Further review and investigation of box girders with and without inclined webs, may be investigated analytically to determine whether or not the design rules developed for this work are satisfactory. These, as they represent a significant extension of this work, should be confirmed by large scale tests.

Chapter 6

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