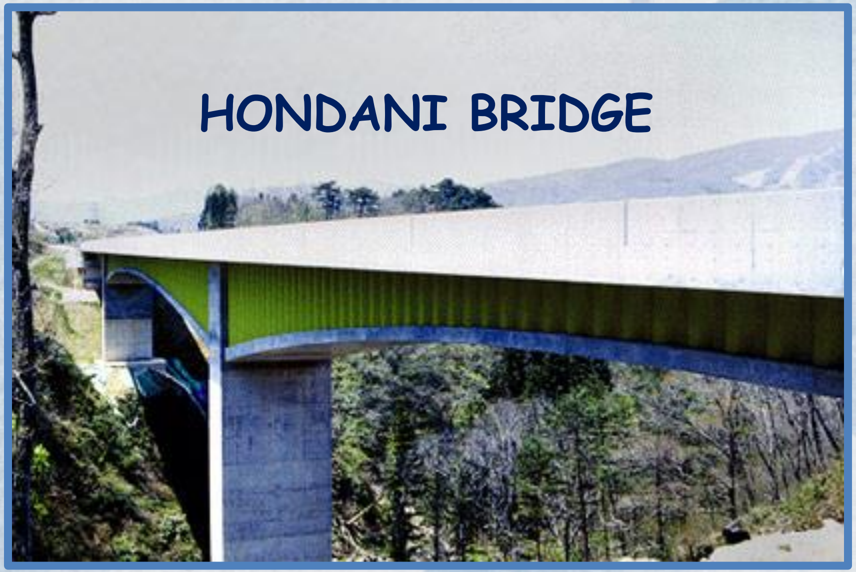
ADVANCES ON SHEAR STRENGTH AND BEHAVIOR OF BRIDGE GIRDERS WITH STEEL CORRUGATED WEBS



Dr. MOSTAFA FAHMI HASSANEIN
Associate Professor

Department of Structural Engineering; Faculty of Engineering; Tanta University, Tanta; Egypt

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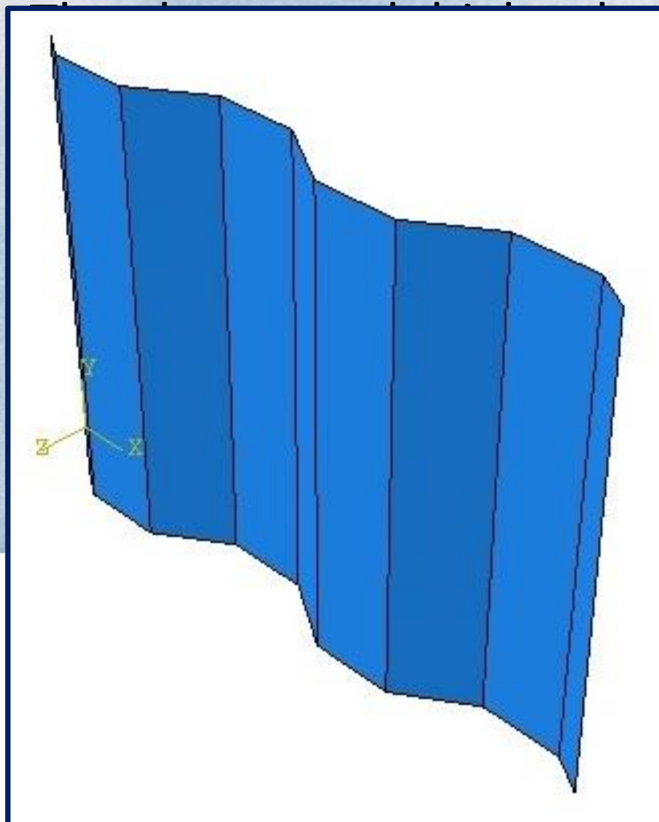
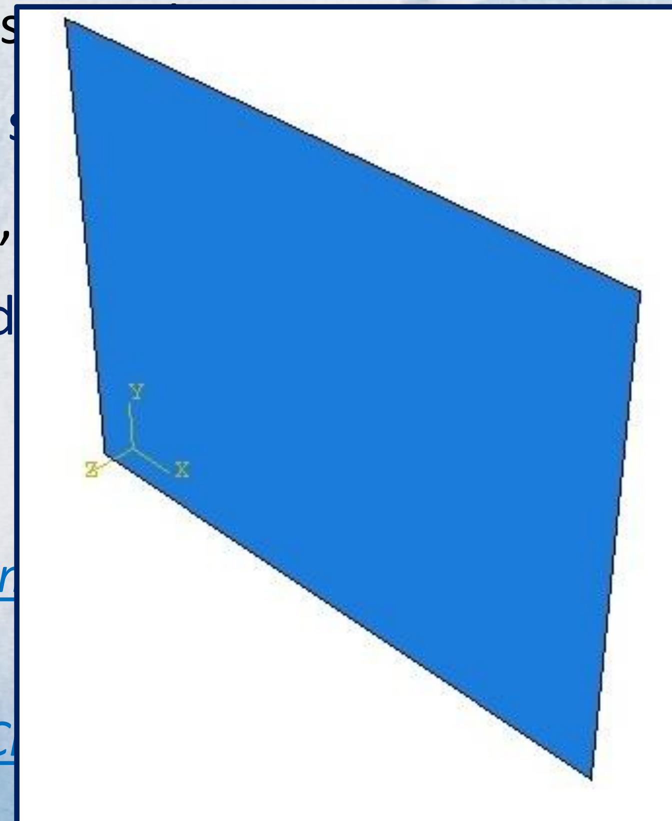
Contents

•*Introduction*

1. Advantages of corrugated web plates

- ➢ …ckling s…
- ➢ …plane s…
- ➢ …feners, …
- ➢ …tly red…





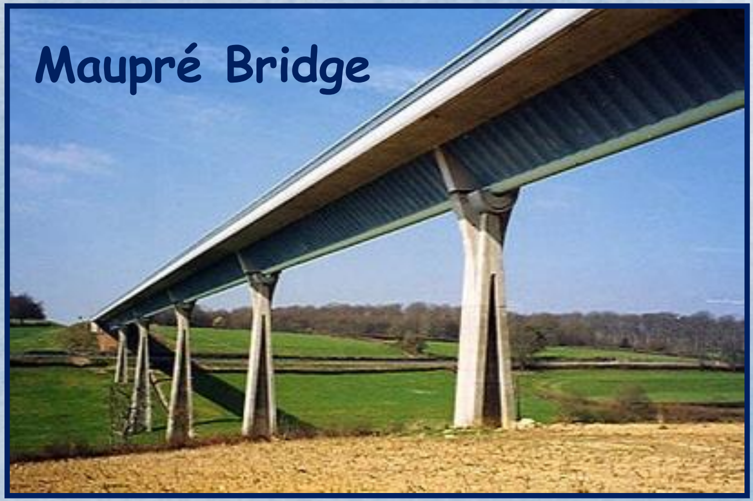
Har…

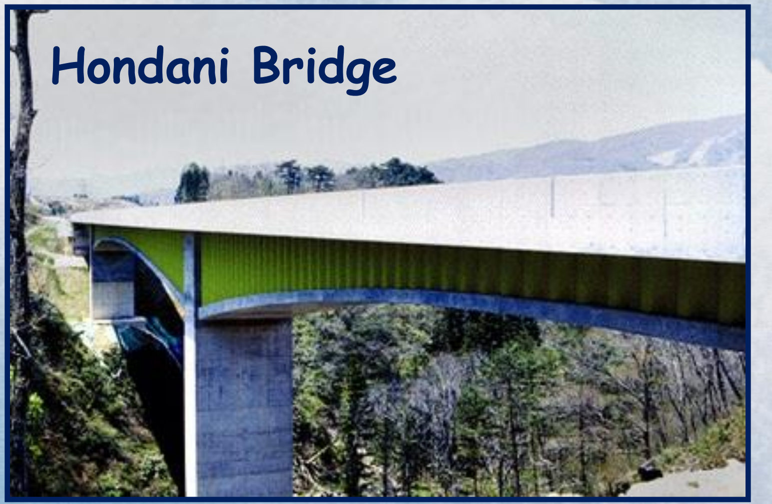
Cl…

•*Introduction*

1. Advantages of corrugated web plates

- They have much higher buckling strengths.
- The own significant out-of-plane stiffness.
- They act as continuous stiffeners, so no need to use stiffeners.
- Their thickness is significantly reduced.



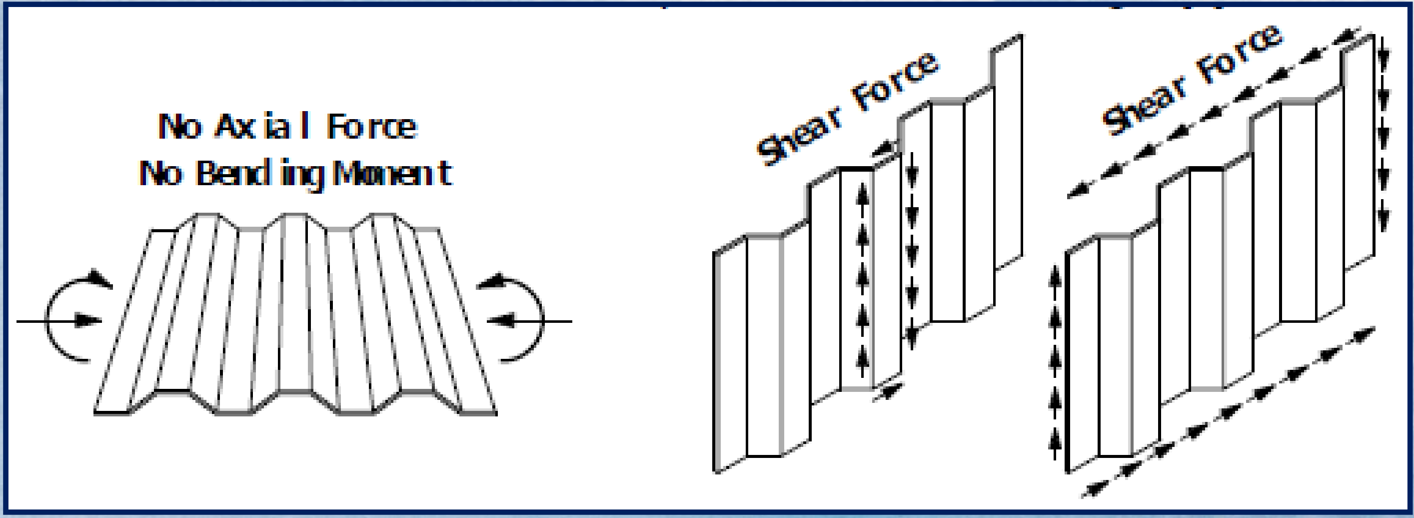


- **Structural efficiency**
- **Aesthetical appearance**

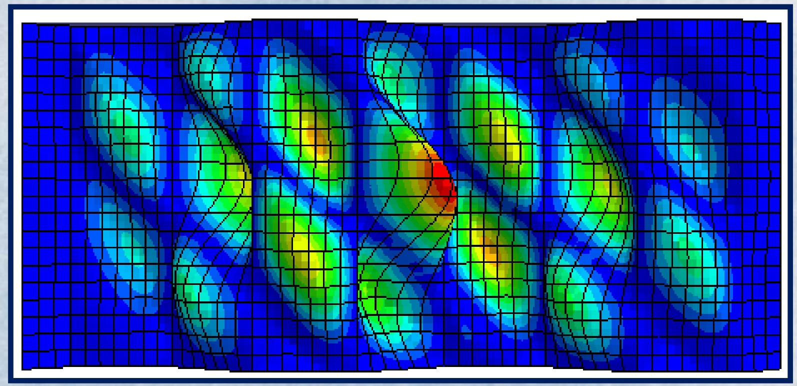
•Introduction

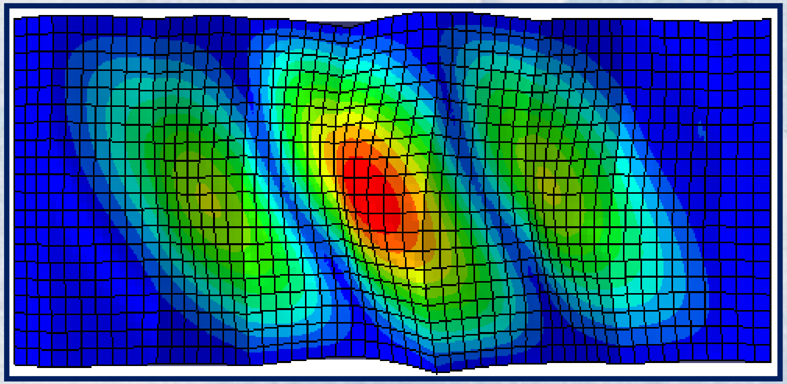
2. Interaction between shear and flexural behaviors

- Corrugated webs do not carry significant longitudinal stresses from the primary flexure of the girders and, consequently, the bending moment can reasonably be assumed to be carried totally by the flanges. Hamilton (1993) and Driver et al (2006)
- This is called the “accordion effect” and it is characterized by negligible axial stiffness.
- Therefore, the shear is carried entirely by the webs.



3. Elastic shear buckling



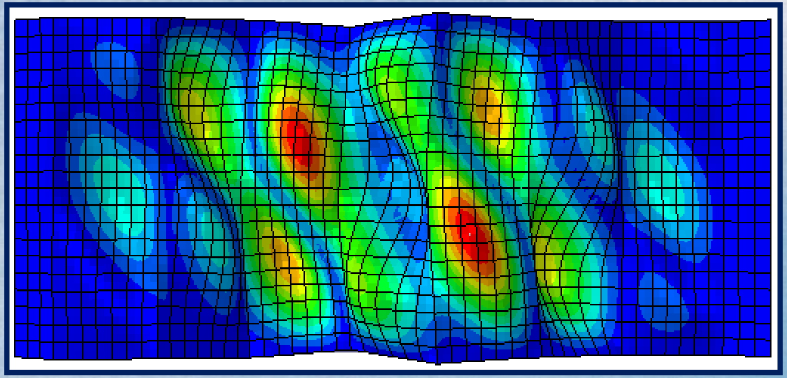


Local buckling $\tau_{cr,L}$

Classical plate buckling theory

Global buckling $\tau_{cr,G}$

Orthotropic-plate buckling theory



Interactive buckling $\tau_{cr,I}$

$$\frac{1}{(\tau_{cr,I})^n} = \frac{1}{(\tau_{cr,L})^n} + \frac{1}{(\tau_{cr,G})^n}$$

Lindner and Aschinger (1988)

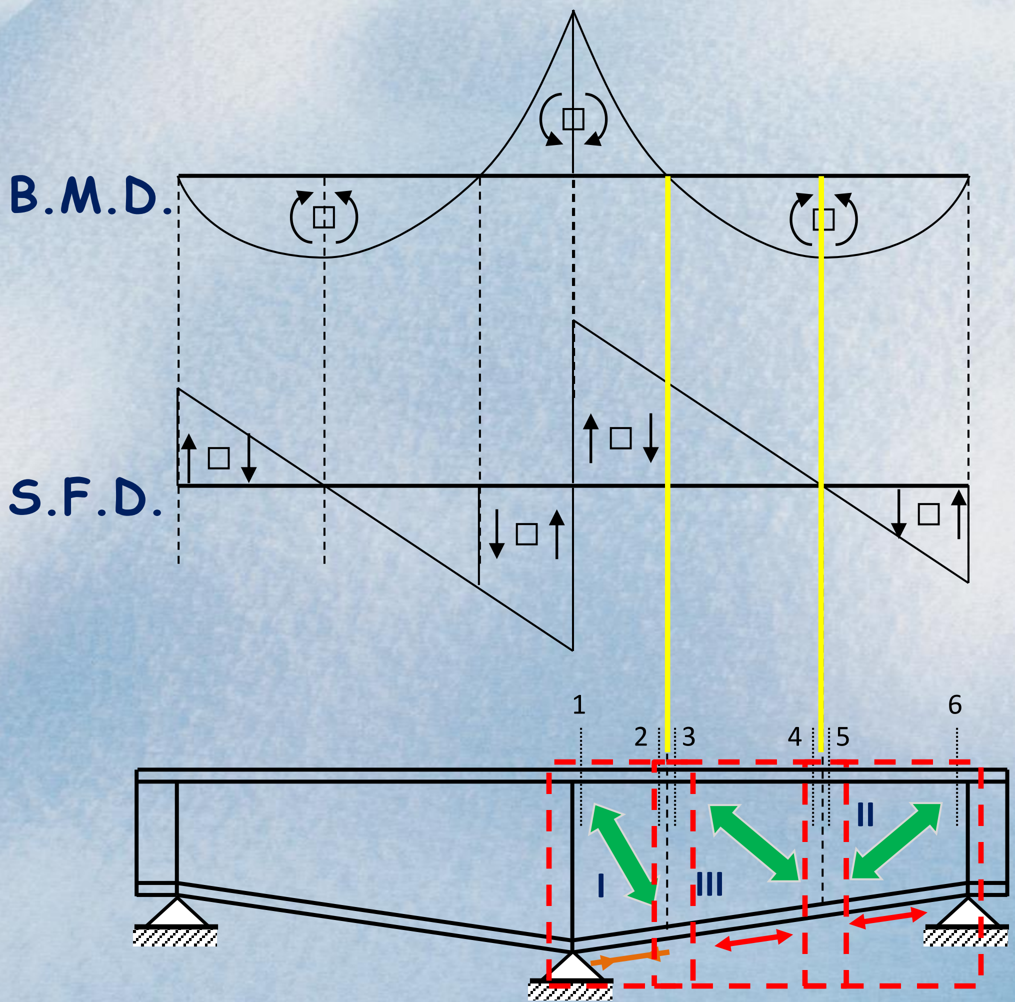
4. Interactive shear buckling stresses

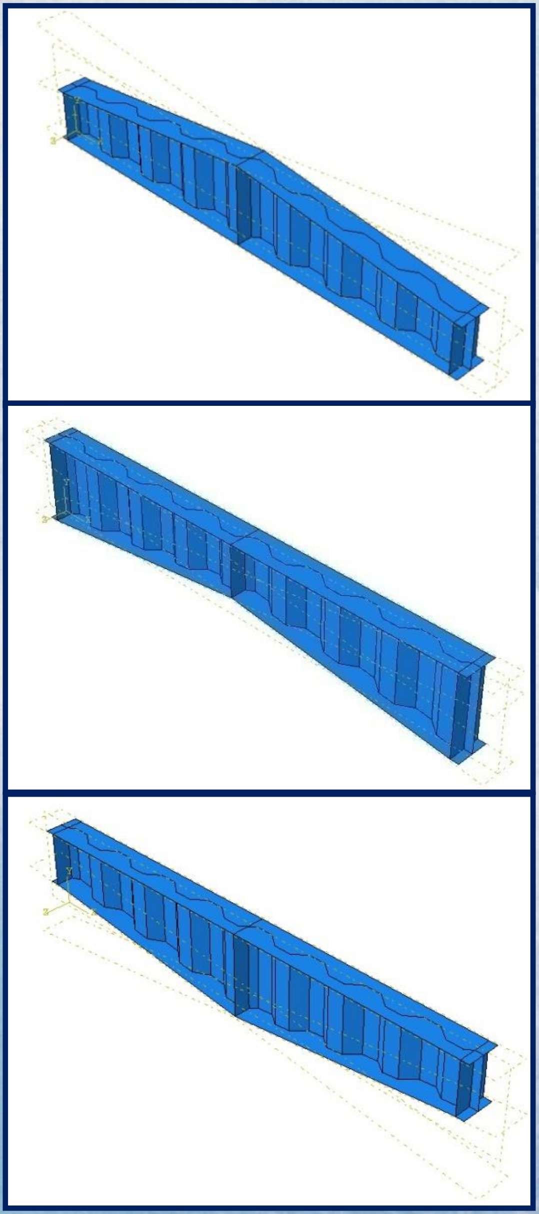
Several researches were conducted to find the best exponent n

Paper	Year	Interactive shear buckling stress predictions
Bergfelt and Leiva	1984	$\frac{1}{(\tau_{cr,I})} = \frac{1}{(\tau_{cr,L})} + \frac{1}{(\tau_{cr,G})}$
Yi et al.	2008	
El-Metwally	1998	$\frac{1}{(\tau_{cr,I})^2} = \frac{1}{(\tau_{cr,L})^2} + \frac{1}{(\tau_{cr,G})^2} + \frac{1}{(\tau_y)^2}$
Abbas et al.	2002	$\frac{1}{(\tau_{cr,I})^2} = \frac{1}{(\tau_{cr,L})^2} + \frac{1}{(\tau_{cr,G})^2}$
Hiroshi	2003	$\frac{1}{(\tau_{cr,I})^4} = \frac{1}{(\tau_{cr,L})^4} + \frac{1}{(\tau_{cr,G})^4}$
Sayed-Ahmed	2005	$\frac{1}{(\tau_{cr,I})^3} = \frac{1}{(\tau_{cr,L})^3} + \frac{1}{(\tau_{cr,G})^3} + \frac{1}{(\tau_y)^3}$

•*Introduction*

5. Types of tapered web panels





Direction of Tension Field

Tension on tapered flange

Compression on tapered flange

Contents

• ***Objectives***

1. Research objectives

is to provide additional data to engineers and the scientific community about the shear strength and behavior of:

- *Prismatic* bridge girders with corrugated webs.
- *Tapered* bridge girders with corrugated webs.

In detail, to

- find the real behavior at the juncture between the corrugated web and the flanges of *Prismatic* girders. Never been studied
- provide new critical stress formula.
- suggest a more suitable strength than those available in literature.

• ***Objectives***

1. Research objectives: Continue

is to provide additional data to engineers and the scientific community about the shear strength and behavior of:

- *Prismatic* bridge girders with corrugated webs.
- *Tapered* bridge girders with corrugated webs.

In detail, to

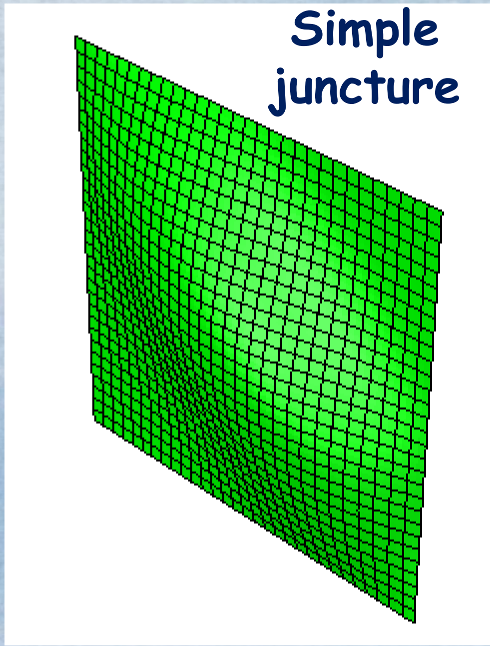
- Investigate the effect of different junctures between the corrugated web and the flanges of *Tapered* girders.
- provide new critical stress formulas for different tapered typologies.
- get the appropriate design strengths of *Tapered* girders.

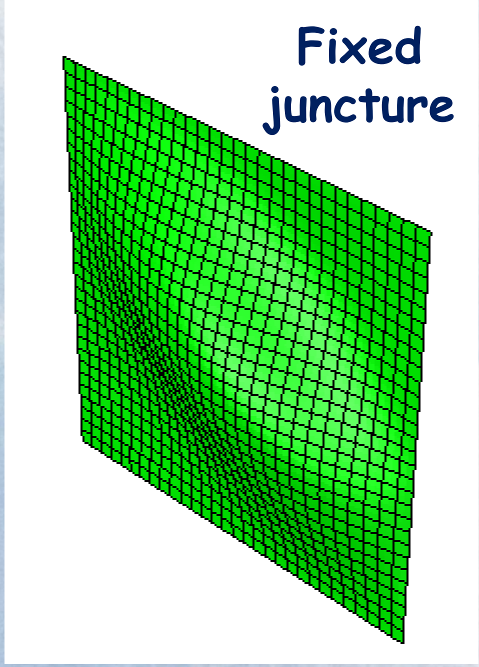
Contents

• *Validation of FEM*

1. Flat web plates

$$\tau_{cr} = \frac{k\pi^2 E}{12(1-\nu^2)}\left(\frac{t_w}{h_w}\right)^2$$





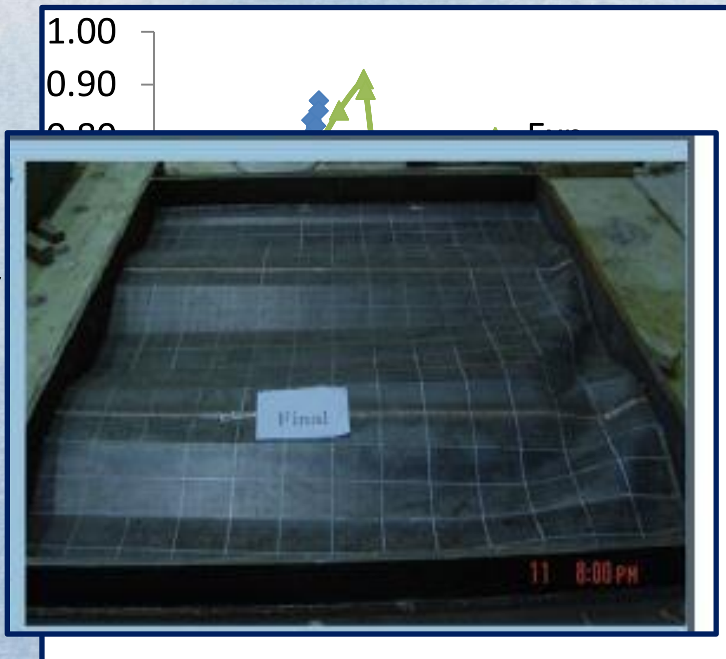
$$k_{ss} = 9.34$$

$$k_{sf} = 8.98 + \frac{5.61}{(a/h_w)^2} - \frac{1.99}{(a/h_w)^3} \; if \; a/h_w \geq 1.0$$

• ***Validation of FEM***

2. Prismatic girders with corrugated webs

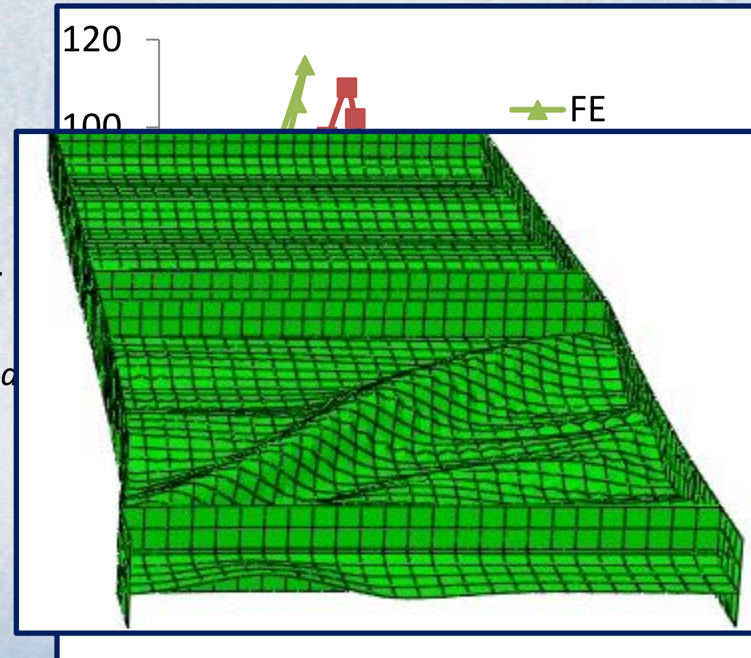
τ_{FE}/τ_y



Deflection (mm)

T

[Mpa

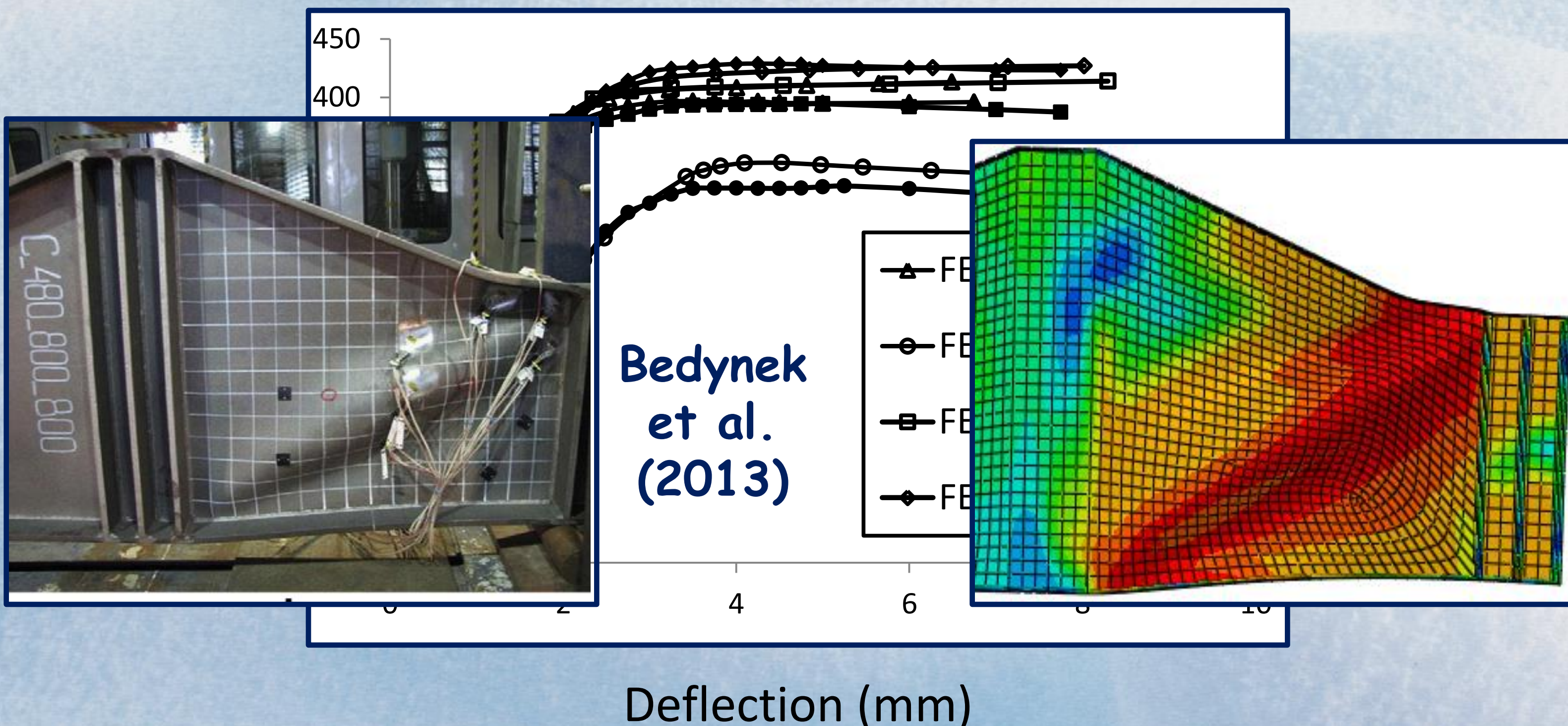


Deflection (mm)

M12: Moon et al. (2009)

• *Validation of FEM*

3. Tapered girders with flat webs



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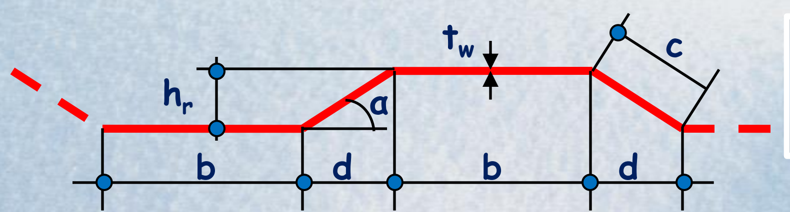
Parametric study

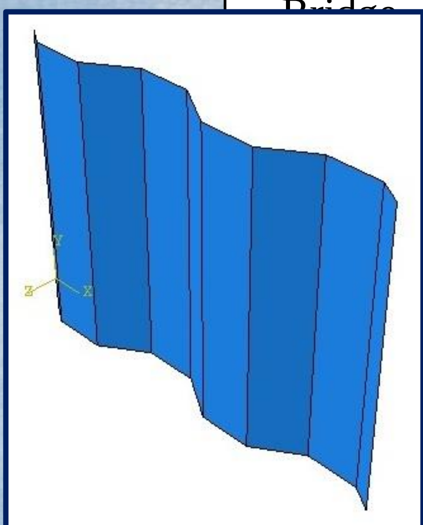
Prismatic

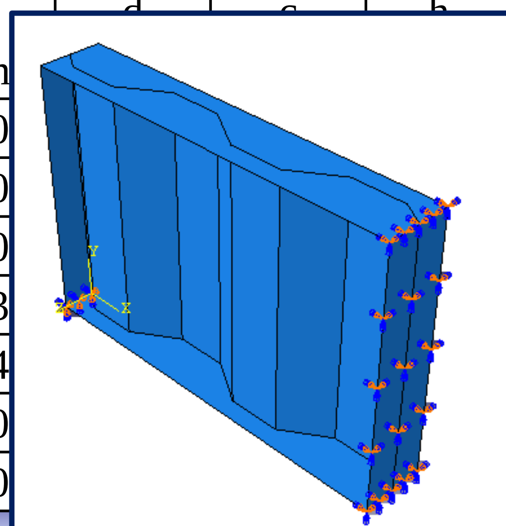


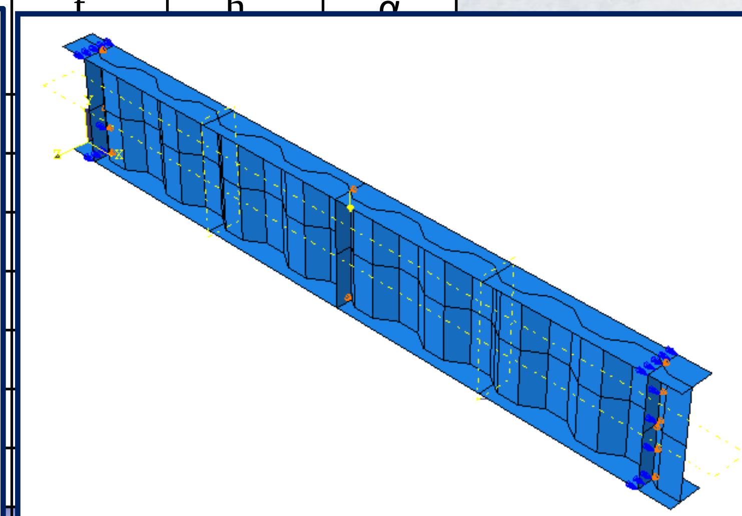
Tapered

Plates | **Panels** | **Girders**

Bridge	b [mm]	d	c	h	t	h	α
	250						
	300						
	330						
	353						
	284						
	430						
	330						
Average	325	284	334	174	10	2281	32



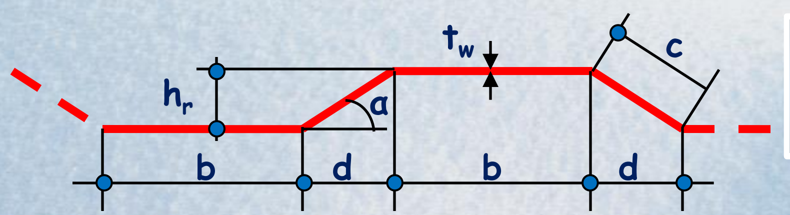




Parametric study

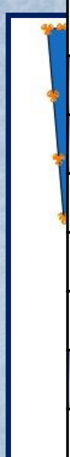
Prismatic

Tapered

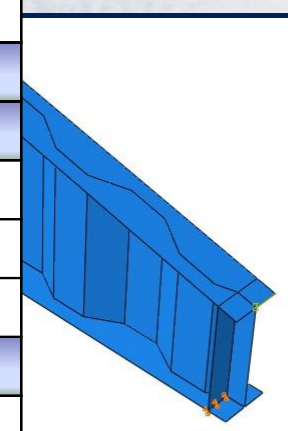


Plates

Girders



Bridge name	b [mm]	d [mm]	c [mm]	h_r [mm]	t_w [mm]	h_w [mm]	α [º]
Shinkai	**250**	**200**	**250**	**150**	**9**	**1183**	**36.9**
Matsnoki	**300**	**260**	**300**	**150**	**10**	**2210**	**30.0**
Hondani	330	270	336	200	9	3315	36.5
Cognac	353	319	353	150	8	1771	25.2
Maupre	284	241	284	150	8	2650	31.9
Dole	**430**	**370**	**430**	**220**	**10**	**2546**	**30.7**
IIsun	330	330	386	200	18	2292	31.2
Average	**325**	**284**	**334**	**174**	**10**	**2281**	**32**



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- Introduction
- Objectives
- Validation of finite element models
- Parametric study
- Results and discussion
- Recommendations

•*Results and discussion*

Prismatic - Plates

1. Effect of simple and fixed boundary conditions

No.	h_w [mm]	t_w [mm]	Buckling mode		$\tau_{cr,FE}$ [N/mm²]		[10]/[11]
			S	F	S	F	
[1]	[2]	[3]	[8]	[9]	[10]	[11]	[12]
1	1000	6	L	L	366	377	0.97
2	1000	8	I	I	612	636	0.96
3	1000	10	I	I	889	932	0.95
4	1000	12	I	I	1184	1255	0.94
5	1000	14	I	I	1489	1579	0.94
6	1000	16	I	I	1782	1904	0.94
7	1000	18	I	I	2085	2235	0.93
15	1400	6	I	I	351	354	0.99
16	1400	8	I	I	578	590	0.98
17	1400	10	I	I	818	853	0.96
18	1400	12	G	I	1070	1119	0.96
19	1400	14	G	I	1329	1391	0.96
20	1400	16	G	I	1594	1673	0.95
21	1400	18	G	I	1864	1962	0.95
						Ave	**0.96**
						COV	**0.014**

•***Results and discussion***

Prismatic - Plates

2. Comparison with the available interactive critical stress (n=1.0)

No.	h_w [mm]	t_w [mm]	Simple $\tau_{cr,FE}/\tau_{cr,I,1}$		Fixed $\tau_{cr,FE}/\tau_{cr,I,1}$		Fixed $\tau_{cr,FE}/\tau_{cr,I,0.6}$	
			k_G=36	k_G=31.6	k_G=68.4	k_G=59.2	k_G=68.4	k_G=59.2
[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]	[9]
1	1000	6	1.12	1.13	0.68	0.69	0.83	0.85
2	1000	8	1.08	1.09	0.66	0.67	0.85	0.87
3	1000	10	1.04	1.05	0.64	0.65	0.85	0.87
4	1000	12	0.99	1.01	0.61	0.63	0.84	0.87
5	1000	14	0.95	0.97	0.59	0.60	0.82	0.86
6	1000	16	0.90	0.92	0.56	0.57	0.80	0.84
7	1000	18	0.86	0.89	0.54	0.55	0.78	0.82
15	1400	6	1.13	1.14	0.67	0.68	0.88	0.90
16	1400	8	1.10	1.12	0.66	0.67	0.90	0.94
17	1400	10	1.05	1.08	0.64	0.65	0.91	0.95
18	1400	12	1.01	1.04	0.61	0.63	0.90	0.94
19	1400	14	0.98	1.02	0.59	0.61	0.89	0.93
20	1400	16	0.95	1.00	0.57	0.60	0.88	0.93
21	1400	18	0.94	0.98	0.56	0.59	0.87	0.92
		Ave	**1.12**	**1.17**	**0.67**	**0.70**	**0.99**	**1.05**
		COV	**0.130**	**0.149**	**0.066**	**0.076**	**0.131**	**0.150**

$$n = 0.6$$

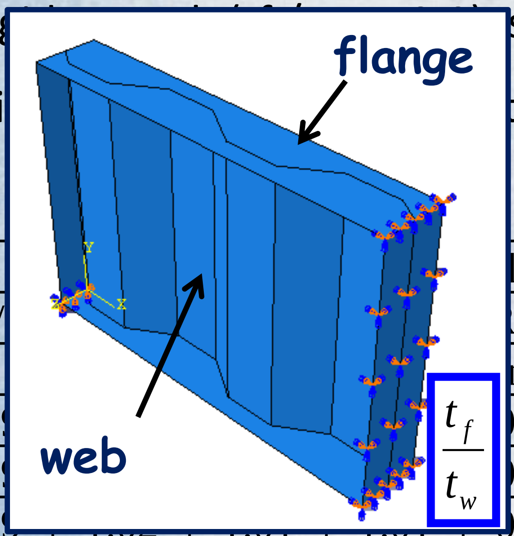
$$\tau_{cr,I,0.6} = \frac{\tau_{cr,L}.\tau_{cr,G}}{\left((\tau_{cr,L})^{0.6} + (\tau_{cr,G})^{0.6}\right)^{\frac{1}{0.6}}}$$

Prismatic - Panels

3. Limit of fixed juncture

➢ when the flanges are ri[illegible] shear failure mechanisms occurs.

➢ if they are relatively ri[illegible]trength becomes controlled by the deformation of flanges.



t_w [mm]	t_f/t_w	[illegible] stresses									
		RF/S	RF/[illegible]	[illegible]	[illegible]	[illegible]	[illegible]F/F	RF/S	RF/F	RF/S	RF/F
		h_w=1.6m		[illegible]		[illegible]	[illegible]m	h_w=2.2m		h_w=2.4m	
6	3	1.00	0.9[illegible]	[illegible]	[illegible]	[illegible]	[illegible].99	1.00	0.99	1.01	0.99
6	4	1.00	0.9[illegible]	[illegible]	[illegible]	[illegible]	[illegible].99	1.00	0.99	1.01	0.99
6	5	1.00	0.9[illegible]	[illegible]	[illegible]	[illegible]	[illegible].99	1.03	1.01	1.01	1.00
8	3	1.02	1.00	1.03	0.99	1.05	1.00	1.04	1.00	1.04	1.01
8	4	1.02	1.00	1.04	1.00	1.05	1.01	1.05	1.01	1.05	1.02
8	5	1.03	1.01	1.05	1.01	1.05	1.01	1.05	1.01	1.06	1.02
10	3	1.04	1.00	1.04	1.00	1.04	1.00	1.04	1.00	1.04	1.01
10	4	1.05	1.00	1.05	1.01	1.06	1.01	1.06	1.02	1.06	1.02
10	5	1.06	1.01	1.07	1.02	1.07	1.02	1.07	1.03	1.07	1.04
	Ave	**1.04**	**1.00**	**1.05**	**1.01**	**1.04**	**1.01**	**1.04**	**1.01**	**1.04**	**1.01**
	COV	**0.024**	**0.012**	**0.018**	**0.011**	**0.020**	**0.014**	**0.018**	**0.012**	**0.019**	**0.013**

➢ The realistic support condition at the juncture is nearly fixed for the case of (t_f / t_w ≥ 3.0).

•*Results and discussion*

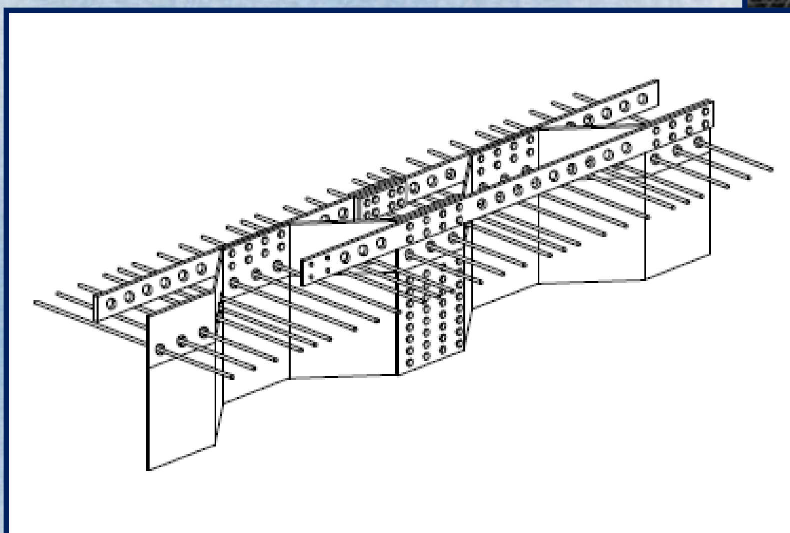
Prismatic - Panels

4. Recommended validity limit of the proposed formula

- For cases of **girde**… …ing (t_f / t_w ≥ 3.0).
- In **composite gir**… …r connections.

$$\tau_{cr,I,0.6} = \frac{\tau_{cr,L} . \tau_{cr,G}}{\left((\tau_{cr,L})^{0.6} + (\tau_{cr,G})^{0.6} \right)^{\frac{1}{0.6}}}$$

Embedded connections



Ikeda and Sakurada (2005)

Kurobe Bridge

•*Results and discussion*

Prismatic - Girders

5. Comparison with available shear strengths

Moon et al.

$$\frac{\tau_{n,M}}{\tau_y} = \begin{cases} 1.0 & : \lambda_s \le 0.6 \\ 1-0.614(\lambda_s - 0.6) & : 0.6 < \lambda_s \le \sqrt{2} \\ \frac{1}{\;} & : \sqrt{2} < \lambda_s \end{cases}$$

Driver et al.

$$\tau_{n,D} = \sqrt{\frac{(\tau_{cr,L} . \tau_{cr,G})^2}{(\tau_{cr,L})^2 + (\tau_{cr,G})^2}}$$

C bridges

eel webs

Sause and Braxtan

$$\lambda_s = 1.05\sqrt{\frac{\tau_y}{\;}}\left(\frac{h_w}{t_w}\right)$$

$$\tau_{n,S} = \tau_y \left(\frac{1}{(\lambda_{I,3})^6 + 2}\right)^{1/3}$$

•*Results and discussion*

Prismatic - Girders

5. Comparison with available shear strengths

Girder (h_w - t_w)	Failure modes	$\frac{\tau_{FE}}{\tau_y}$	$\frac{\tau_{n,M}}{\tau_y}$	$\frac{\tau_{n,D}}{\tau_y}$	$\frac{\tau_{n,S}}{\tau_y}$	$\frac{\tau_{n,M,0.6}}{\tau_y}$	$\frac{\tau_{n,D,0.6}}{\tau_y}$	$\frac{\tau_{n,S,0.6}}{\tau_y}$	$\frac{\tau_{n,S,0.6}}{\tau_{FE}}$
1600-6	I	0.85	0.86	0.71	0.63	0.91	1.80	0.77	0.91
1600-8	I	0.92	1.00	0.71	0.63	1.00	2.86	0.79	0.85
1600-10	I	0.79	1.00	0.71	0.63	1.00	4.02	0.79	1.00
1600-12	G	0.85	1.00	0.71	0.63	1.00	5.27	0.79	0.93
1800-6	I	0.90	0.86	0.71	0.63	0.90	1.70	0.77	0.86
1800-8	I	0.91	0.97	0.71	0.63	0.99	2.66	0.79	0.87
1800-10	I	0.80	1.00	0.71	0.63	1.00	3.71	0.79	0.99
1800-12	G	1.01	1.00	0.71	0.63	1.00	4.81	0.79	0.78
	Ave	**0.85**	0.95	0.71	0.63	0.97	3.00	**0.78**	**0.93**
	COV	**0.077**	0.066	0.000	0.000	0.051	1.150	**0.012**	**0.090**

$$if\,\tau_{el} > 0.8\tau_y$$

$$\tau_{inel} = \sqrt{0.8\tau_y\tau_{el}} \le \tau_y$$

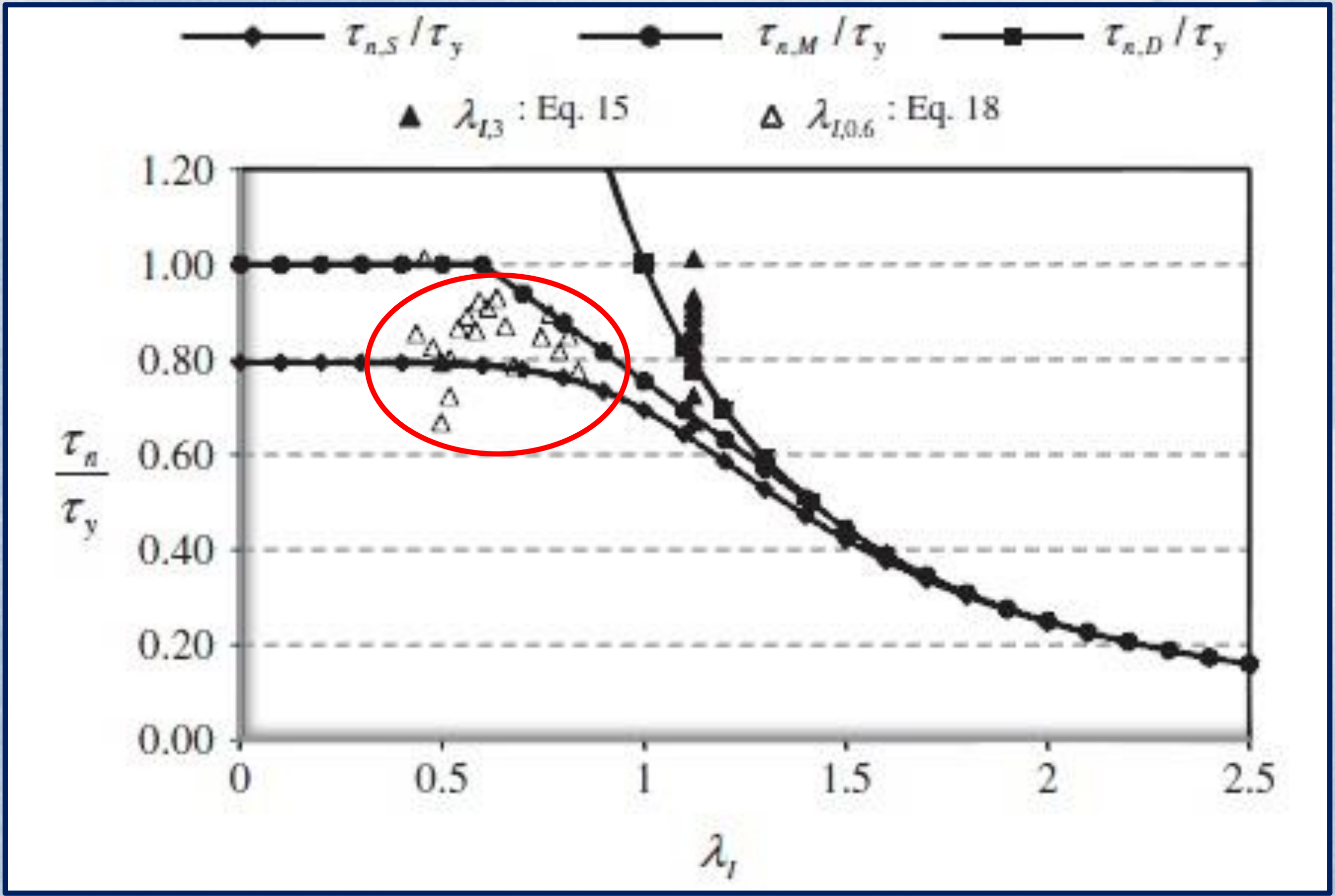
Elgaaly et al. (1996)

$$Ave\frac{\lambda_{I,0.6}}{\lambda_{I,3}} = 0.54$$

•*Results and discussion*

Prismatic - Girders

5. Comparison with available shear strengths



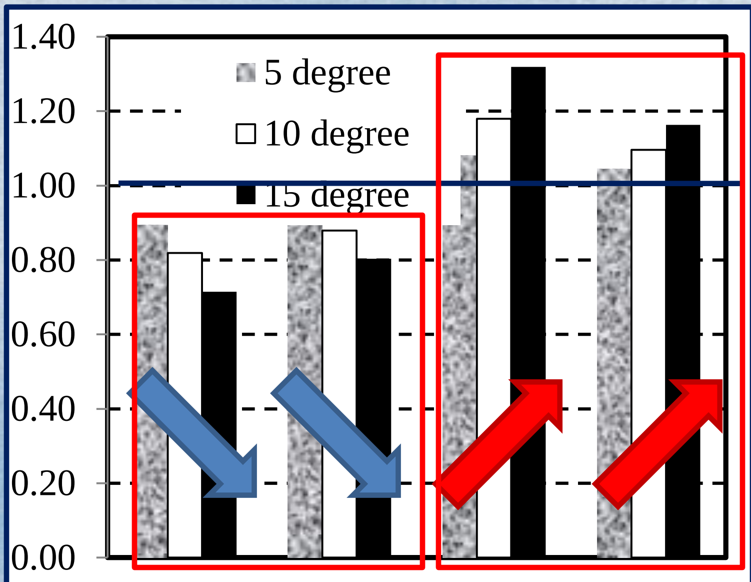
•*Results and discussion*

Tapered - Plates

1. Effect tapered web typology on interactive critical stress

$$\frac{\tau_{cr,FE}}{\tau_{cr,FE,P}}$$

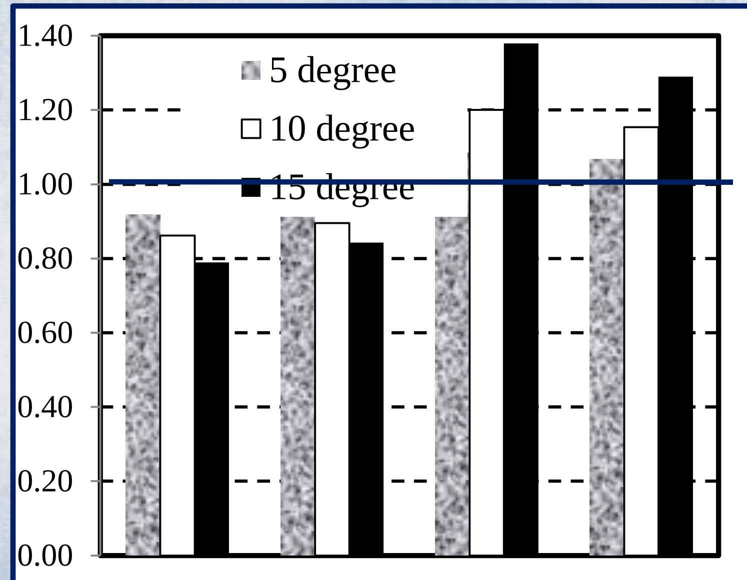
$t_w = 6mm$



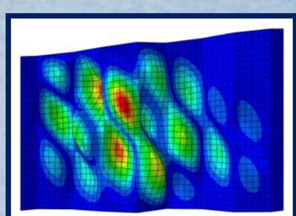
Case I Case II Case III Case IV

$$\frac{\tau_{cr,FE}}{\tau_{cr,FE,P}}$$

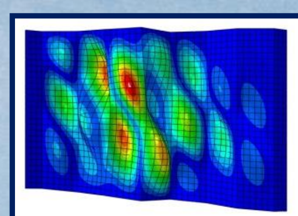
$t_w = 14mm$



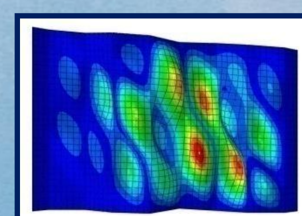
Case I Case II Case III Case IV



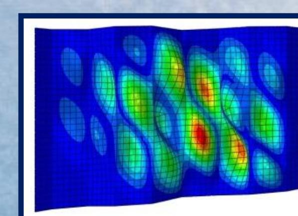
Case I



Case II



Case III



Case IV

•*Results and discussion*

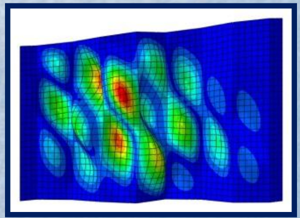
Tapered - Plates

2. Proposed interactive critical stress of tapered webs

The recommendation of the current design code for plated structural elements *EN 1993-1-5 (2007)* to determine the ultimate shear resistance of tapered plate girders with flat web plates as prismatic ones CANNOT BE USED.

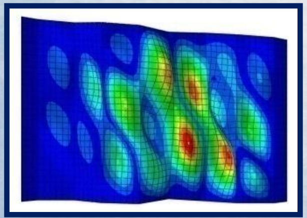
Applicable for different boundary conditions

$$\tau_{cr,\mathrm{Prop}} = \tau_{cr,FE,P} /(1+\tan\gamma)$$



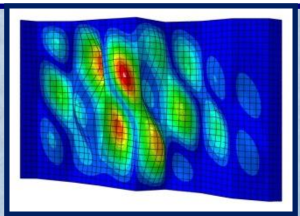
Case I

$$\tau_{cr,\mathrm{Prop}} = \tau_{cr,FE,P} /(1-\tan\gamma)$$



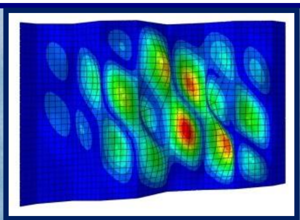
Case III

$$\tau_{cr,\mathrm{Prop}} = 1.04\tau_{cr,FE,P} /(1+\tan\gamma)$$



Case II

$$\tau_{cr,\mathrm{Prop}} = 0.94\tau_{cr,FE,P} /(1-\tan\gamma)$$



Case IV

Tapered - Girders

3. Nonlinear strengths – Parametric study results

Type	t_w [mm]	λ_s	$V_{ul,FE}$ [kN]	Buckling mode	$\frac{\tau_{ul,FE}}{\tau_y}$
Case I	6	0.683	944	I	0.96
	8	0.533	1318	G	1.00
	10	0.445	1693	G	1.03
	12	0.386	2144	G	1.09
	14	0.346	2619	G	1.14
Case II	6	0.696	1097	I	1.12
	8	0.544	1619	G	1.23
	10	0.453	2010	G	1.23
	12	0.394	2147	G	1.09
	14	0.352	2295	I	1.00
Case III	6	0.602	652	I	0.66
	8	0.470	917	I	0.70
	10	0.392	1146	G	0.70
	12	0.341	1450	G	0.74
	14	0.305	1686	G	0.73
Case IV	6	0.624	634	I	0.64
	8	0.487	894	I	0.68
	10	0.406	1136	G	0.69
	12	0.353	1397	G	0.71
	14	0.316	1666	G	0.73

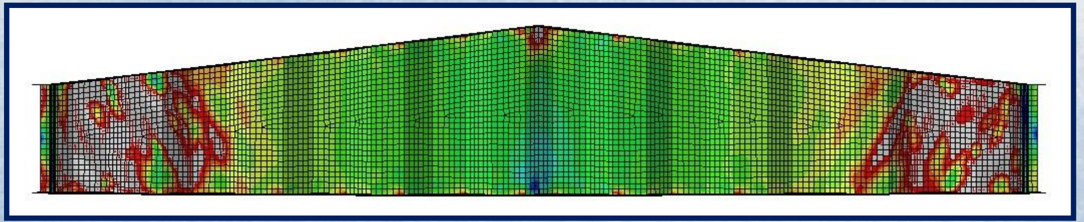
$$\tau_{ul,FE} / \tau_y \geq 1.0$$

This means that **neither** the type of the axial force in the inclined flange (tension or compression) **nor** the direction of the developed tension field (short or on the long web diagonal) **has any effect on the strength of these girders (Cases III and IV)**

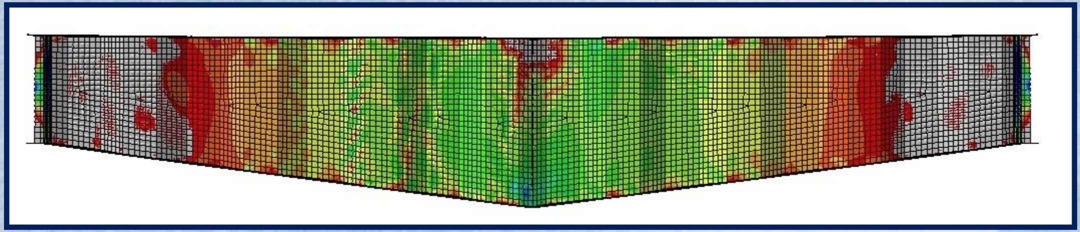
•Results and discussion

Tapered - Girders

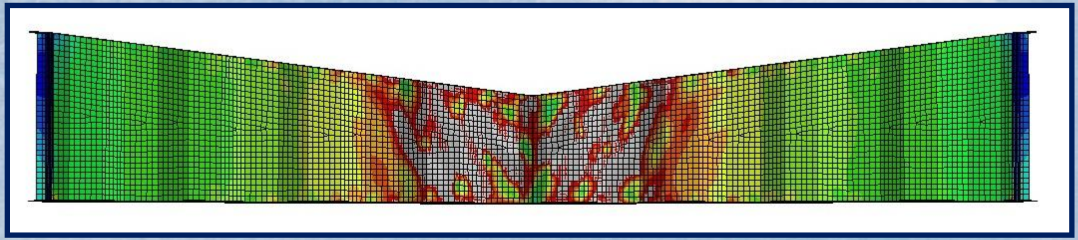
4. Failure modes and stress distributions



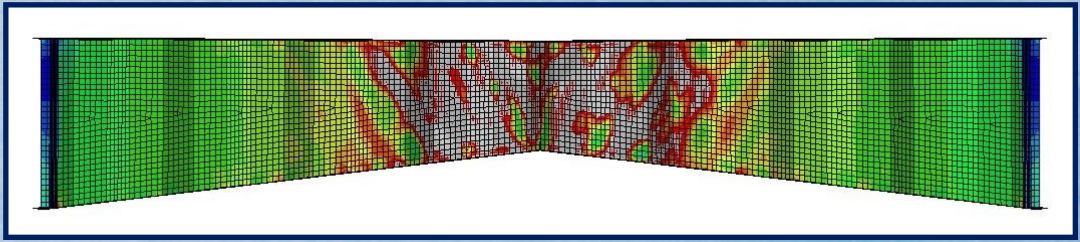
Case I



Case II



Case III



Case IV

•*Results and discussion*

Tapered - Girders

5. Comparison with available shear strengths

Moon et al.

$$\frac{\tau_{n,M}}{\tau_y} = \begin{cases} 1.0 & : \lambda_s \le 0.6 \\ 1 - 0.614(\lambda_s - 0.6) & : 0.6 < \lambda_s \le \sqrt{2} \\ \dfrac{1}{\lambda_s^2} & : \sqrt{2} < \lambda_s \end{cases}$$

Design manual for PC bridges with corrugated steel webs

$$\lambda_s = 1.05\sqrt{\frac{\tau_y}{k_I E}}\left(\frac{h_w}{t_w}\right)$$

Sause and Braxtan

$$\tau_{n,S} = \tau_y \left(\frac{1}{(\lambda_{I,3})^6 + 2}\right)^{1/3}$$

•*Results and discussion*

Tapered - Girders

5. Comparison with available shear strengths

Type	t_w [mm]	λ_s	$V_{ul,FE}$ [kN]	Buckling mode	$\frac{\tau_{ul,FE}}{\tau_y}$	$\frac{\tau_{ul,M}}{\tau_{ul,FE}}$	$\frac{\tau_{ul,S}}{\tau_{ul,FE}}$
Case I	6	0.683	944	I	0.96	0.98	0.81
	8	0.533	1318	G	1.00	1.00	0.79
	10	0.445	1693	G	1.03	0.97	0.77
	12	0.386	2144	G	1.09	0.92	0.72
	14	0.346	2619	G	1.14	0.88	0.69
Case II	6	0.696	1097	I	1.12	0.89	0.71
	8	0.544	1619	G	1.23	0.81	0.64
	10	0.453	2010	G	1.23	0.81	0.64
	12	0.394	2147	G	1.09	0.92	0.72
	14	0.352	2295	F	1.00	1.00	0.79
Case III	6	0.602	652	I	0.66	0.97	1.20
	8	0.470	917	I	0.70	0.91	1.13
	10	0.392	1146	G	0.70	0.91	1.13
	12	0.341	1450	G	0.74	0.86	1.07
	14	0.305	1686	G	0.73	0.88	1.08
Case IV	6	0.624	634	I	0.64	0.97	1.22
	8	0.487	894	I	0.68	0.94	1.16
	10	0.406	1136	G	0.69	0.93	1.14
	12	0.353	1397	G	0.71	0.90	1.11
	14	0.316	1666	G	0.73	0.88	1.08

h_{wo} / h_{w1}

Tapered - Girders

6. Recommended design shear strengths

$$\frac{\tau_{ul,\mathrm{Prop}}}{\tau_y} = C_T \begin{cases} 1.0 & : \lambda_s \le 0.6 \\ 1 - 0.614(\lambda_s - 0.6) & : 0.6 < \lambda_s \le \sqrt{2} \\ \dfrac{1}{\lambda_s^2} & : \sqrt{2} < \lambda_s \end{cases}$$

$$C_T = \begin{cases} 1.0 & \text{Case I} \quad \text{Case II} \\ h_{wo} / h_{w1} & \text{Case III} \quad \text{Case IV} \end{cases}$$

Contents

Recommendations

- New experimental results on girders with corrugated webs with real bridge dimensions should be conducted.
- New work should be made on checking the available buckling and design equations on all available bridge corrugated web profiles. *An MSc is under preparation now by Elkawase, A.A. at Tanta University*.
- The real behaviour of continuous tapered girders with corrugated webs , containing different web typologies, should be carried out by means of experimental tests. *A thesis is under preparation now by Hassanein and El Hadidy at Tanta University*.
- Concentration on the behaviour of tapered girders' behaviour should be made. *A paper is under review now by Zevallos et al. (Zevallos, E., Hassanein, M.F., Real, E. Mirambell, E.) as a collaboration between Tanta University and Universitat Politècnica de Catalunya, UPC of Spain*.

Recommendations

- Composite girders with corrugated webs in negative bending moment zones should be checked by using fibre reinforced polymers. *An MSc is under preparation know by Elshinrawy, T. at Tanta University*.

For more information:

- **Hassanein, M.F.**, Kharoob O.F., (2015), "Linearly Tapered Bridge Girder Panels with Steel Corrugated Webs near Intermediate Supports of Continuous Bridges", ***Thin-Walled Structures***, Vol. 88, pp. 119-128.
- **Hassanein, M.F.**, Kharoob O.F., (2014), "Shear buckling Behavior of Tapered Bridge Girders with Steel Corrugated Webs", ***Engineering Structures***, Vol. 74, pp. 157-169, 2014.
- **Hassanein, M.F.**, Kharoob O.F., (2013), "Behavior of Bridge Girders with Corrugated Webs: (II) Shear Strength and Design", ***Engineering Structures***, Vol. 57, pp. 544-553.
- **Hassanein, M.F.**, Kharoob O.F., (2013), "Behavior of Bridge Girders with Corrugated Webs: (I) Real Boundary Conditions at the Juncture of the Web and Flanges", ***Engineering Structures***, Vol. 57, pp. 554-564.

Thank you for listening

Contact details:

Dr. MOSTAFA FAHMI HASSANEIN

Department of Structural Engineering; Faculty of Engineering; Tanta University, Tanta; Egypt

E-mail: mostafa.fahmi@yahoo.com
mostafa.fahmi@f-eng.tanta.edu.eg
Mobile Tel: +201228898494
Fax: +20403315860