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New Development of Section Classification for Composite Girders with Low Steel Yield Strength

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Abstract: The Composite steel/concrete girder is one of the main structural systems used in bridges and buildings. Steel element mainly located in tension zone and a concrete element located in compression zone. Full integration by the shear connectors used to simulate as one section without any slippage between the two materials. The classification requirements for steel sections and composite sections in most specifications were originally derived from experimental and analytical studies based on the theory of elasticity. The compression concrete slab restrain the buckling of the top flange and the compressed part of the web. Steel plates behave plastically up to failure. By considering concrete slabs connected to the steel compression elements, the section may be placed in a better class in terms of slenderness. The present study focuses on the evaluation of section classifications using ANSYS (FE- software). An extensive parametric study using the calibrated FE modeling procedures was preferred to predict the modified relaxed equation. A new released equation and new classification limits have been developed between compact and non-compact considering the fixity effect of the concrete slab. In this study, yield strength of the steel material is 2.4 ton/cm2 and concrete strength 0.40 ton/cm2.

Index Terms— **Analysis, Composite, flexural; Finite, Nonlinearity.**

I. INTRODUCTION

The steel-concrete composite girder is one of the main most common supper-structural types. Composite action improves the structural behavior by combining the two structural elements to develop a single composite section. The steel section is mainly located in tension region and the concrete slab located in the compression region, connected by metallic devices called shear connectors. **ECP-LRFD (2012)[1]** stated that full composite action is developed when the reinforced concrete slab and shear connectors are designed to avoid splitting of the concrete slab and the plastic moment strength of the composite section could be achieved while the partial composite action is developed when the shear strength of connectors governs the strength capacity of the partially composite beam[1]. **Sanker and Jacob (2013)[2]** concluded that buckling is one of the most important failure modes in steel structures is the failure which is a critical sophisticated phenomena in structures under compression or bending loads. Buckling strength depends on materials nonlinearity, type of acting loads, type of supports, imperfections and affected also by thermal loads. **Gupta et al. (2006)[3]** noticed that most available codes' formulas are based on linear experimental techniques not accounting the material or geometric nonlinearities and not considering for the effect of the concrete slab in composite sections.

II. MATERIALS AND METHODS

Codes Classification

ECP-LRFD (2012)[1] and **AASHTO (2005)[4]** specification classifies steel sections to three types, compact, non-compact and slender sections. **Error! Reference source not found.**(a), while according to **EUROCODE (2001)[5][5]** the composite sections are classified into four categories **Error! Reference source not found.**(b) depending on the local buckling behavior of the web in compression and accounting for the stress-gradient effect. **EUROCODE (2001)** allows using plastic design method only for Class 1 and 2 sections while **AASHTO (2005)** and **ECP-LRFD (2012)** allow it for the compact section only. Section classifications are shown in [Table 1].where parameters as defined in the relevant codes, Figs. 4(a)-(b), M_y, M_p and M_{max} are yield, plastic and ultimate moments respectively and b_w, t_w and ε , Ψ and α are web height, web thickness, maximum strain, ratio between upper to lower flanges stresses and ratio between the location of axis of bending to the web height respectively.

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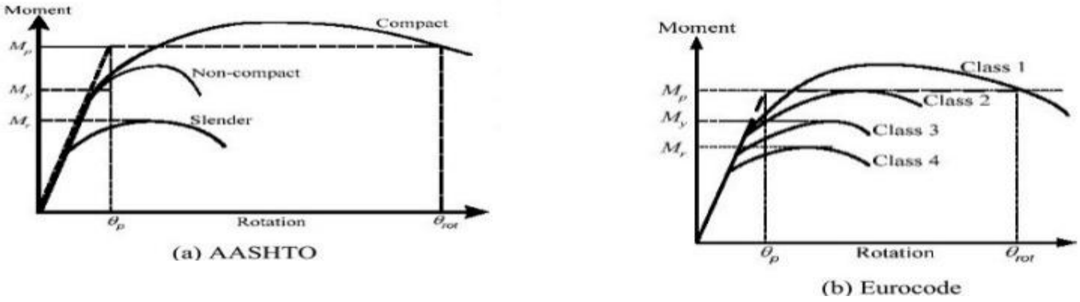

Fig. 1-Section Classification (a) AASHTO (2005), (b) EUROCODE (2012)

Fig. 2- Local buckling of (a) open section, (b) closed sections [6][6]

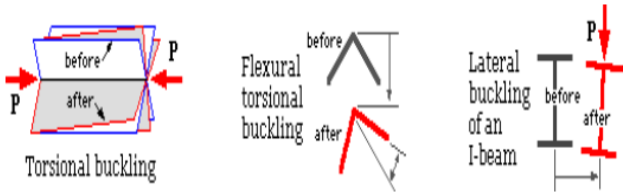

Fig. 3- Types of Overall Buckling[7] Subramani (2012)[7]

Buckling in structures and material plastic behavior

Owens (1994)[6] showed in **Fig. 2** the types of local buckling behaviors of the steel elements. These sections can be regarded as a combination of individual plate elements connected together to form the required shape. ~~[7]~~ **Subramani** and **Sugathan (2012)**[7]showed in **Fig.3** different types for overall buckling due to thin walled shear behavior. Yield stress is achieved when the extreme top or bottom fibers reach the yield stress, the moment corresponding to this state is called the yield moment M_y as shown in Error! Reference source not found.**(b), 6(a)**. This does not imply failure as the beam can take additional load until failure. When the load continues to increase, more fibers at the section reach yield stress and the stress distributions modified**.** Eventually, when the whole of the cross section's fibers reaches the maximum yield stress, the moment at this stage is called plastic moment M_p as shown in Error! Reference source not found.**Fig. 4(a),6(c)**. The ratio of plastic moment to the yield moment (M_p/M_y) is known as the shape factor since it depends on the shape of the cross section. For plate girders consisting from three plates, considering (b) as plate maximum dimension (length) and (t) as plate minimum dimension (thickness) while (β) is a variable value and referring to Error! Reference source not found.**–6** sections of steel structures can be classified to:

- Plastic sections can reach its full-plastic moment Mp and allow rotation at or after the plastic moment.
- Compact cross-sections, can reach its full-plastic moment Mp but the rotation could not be developed.
- Non-Compact cross-sections, local buckling prevents the section from reaching its full-plastic moment Mp.
- Slender cross-sections in which the local buckling prevent ultimately the reaching the yield web slenderness is one of the most important influence on flexural strength of composite girder.

III. LITERATURE REVIEW

Taleb et al. (2015)[8] concluded that this theoretical simplicity of supports is fulfilled by certain dimensions of flanges. Concrete slab at the composite section provide fixation support which lead eliminate the web local buckling length at least in the direction of loading. **Musa (2016)[9]** concluded that the in-plane deformation

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happens axially before transverse buckling and shear deformation[9], concrete slab also provides practically sufficient in plane resistance for the axial deformation compared with the steel section only. Several researchers focused their effort in studying the behavior of composite girders.

Lui et al. (2016)[10] concluded that some of the empirical methods could be more accurate to predict the flexural capacity of simply supported composite beams considering the degree of shear connection and the codes may be more conservative while evaluating the partial composite action. The partial integration is used mainly for the composite girders on buildings where the structures mostly still in elastic zones and not extended to plastic zone .**Prakash et al. (2012)[11]** studied the effect of variable high strength steel shear connectors' densities and shear capacities on the ultimate moment of resistance of steel and concrete composite girders under monotonic load, he recommended that the maximum ultimate load to be limited to 1.5 times the loads relevant to the plastic moment. **Gupta et al. (2007)[12]** concluded that for the full integration composite (Full Shear) compact with high yield strength steel section the failure can occurs due to concrete crushing or steel plastic failure, while for partial integration composite the failure most probably happens due to shear failure of the connectors itself or due to slippage of the concrete slab. For compact sections in which ductile failure takes place, the ultimate flexural strength is given by its full plastic moment capacity. However, for compact sections with higher steel yield strengths, crushing of the concrete slab may take place prior to reaching the full plastic moment capacity of the sections as observed by **Gupta et al. (2006)[3]**, All the girders were designed to verify the ability of the ultimate flexural strength equations provided by the current such as AASHTO (2005) and **EUROCODE (2001)** specifications to predict the flexural strength of sections with D_p/D_t as shown in **Fig. 4(a)** in the linear range, where D_t is the total section depth. Gupta et al (2007)[12] selected girders having D_p/D_t in the range 0.15 to 0.4. Both experimental and analytical results show that the existing strength equations are conservative. A less conservative estimation of an equation to reduce the ultimate flexural strength was developed. The proposed equation is expressed as a function of D_p/D_t ratio. Duc and **Okui (2014) [13]**and **Duc, D.V., Matsuno, [14]**, studied FE models to verify the influence of using composite behavior with SBHS500 and SBHS700 (high strength steel with ultimate strength 500 N/mm^2 and 700 N/mm^2 respectively) on the web slenderness limits for section classification and concluded that applying this high performance steel to both homogeneous and hybrid sections can extend significantly the web slenderness limits of section classification. Compact and non-compact web slenderness limit boundary for homogenous section is about by 70% greater than that of AASHTO (2005) [4] and 50% of that of the EUROCODE (2001). **Patil and Shaikh (2013)[15]** used ANSYS [16], finite element modeling program to simulate six specimens to investigate the impact of interaction (full or partial) type between the two materials of the composite girders. The results conclude that height of the shear connectors does not influence much the deflection of the composite beam. For all these reasons, concrete strength was considered in this study as a factor to produce new equation and classifications limits.

IV. OBJECTIVE OF THE STUDY

The main targets of this study is to achieve a good understanding for the types of predominate failure modes of Steel-Concrete composite girders with ultimate and yield strength equals 3.7t/cm^2 and 2.4 t/cm^2 respectively which is the lowest type of steel strengths used in the ECP-LRFD with lower concrete strength 0.40 t/cm^2 under positive (sagging) moments and provide a less strengthen design tendency and Applying these studies to **ECP-LRFD (2012) [1]**. The present study is focused on the behavior of buckling, yielding, plastic or crushing failure of the main elements of the compact composite and not extended to study of the connection failure or slipping of the concrete slab, full interaction composite.

V. METHODOLOGY

In order to accomplish the study the famous commercial finite element software **ANSYS [16]**, which is able to simulate the overall non-linear plastic behavior of simply supported composite beams subjected to loads including buckling of the steel elements and cracking of the concrete slab.

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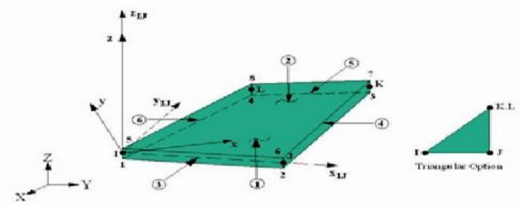
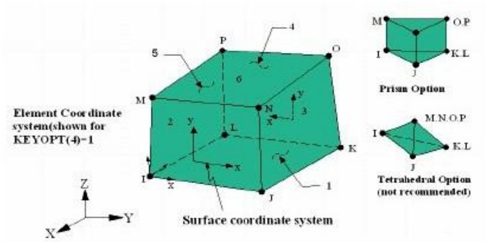


A-SHELL43 Geometry

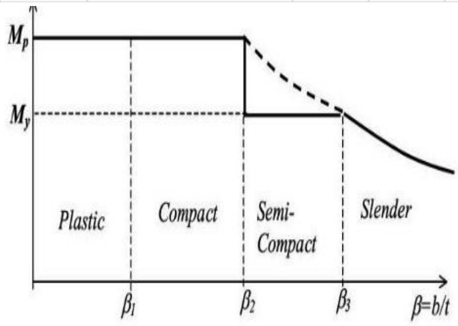


b - 3D SOLID65– Reinforced Concrete

Fig. 4- ANSYS Structural Elements ANSYS[16]

Table 1 - Section Classifications AASHTO[4], EUROCODE[5] and ECP-LRFD[1]

Design Code	Section Class	Definitions	Web Slenderness Limits
AASHTO	Compact	Mmax. > Mp	$2Dcp/tw \leq 3.76\sqrt{Es/Fy}$
	Non-Compact	Mp > Mmax. >My	$2Dcp/tw < 5.70\sqrt{Es/Fy}$
	Slender	Mmax. > My	Other Than The Above
Eurocode	Class -1	Mmax. ≥ Mp Sufficient Rotational Capacity	$bw/tw = 36\varepsilon/\alpha \quad \alpha \leq 0.50$ $bw/tw = 396\varepsilon/(13\alpha-1) \quad \alpha > 0.50$
	Class -2	Mmax. ≥ Mp Limitted Rotational Capacity	$bw/tw = 41.50\varepsilon/\alpha \quad \alpha \leq 0.50$ $bw/tw = 456\varepsilon/(13\alpha-1) \quad \alpha > 0.50$
	Class -3	Mmax. ≥ My	$bw/tw = (42\varepsilon/((0.67+0.33\Psi)) \quad \Psi > -1.00$ $bw/tw = 62\varepsilon(1-\Psi)\sqrt{(-(\Psi)} \quad \Psi \leq -1.00$
	Class- 4	Mmax.<My	Other Than The Above
ECP-LRFD	Compact	Mmax. > Mp	$bw/tw = ((699/\sqrt{fy})/((13\alpha-1)) \quad \alpha \leq 0.50$ $bw/tw = (63.6/\alpha)/(\sqrt{fy}) \quad \alpha > 0.50$
	Non-Compact	Mp > Mmax. >My	$bw/tw = ((111(1-\Psi)\sqrt{(-\Psi)})/((\sqrt{fy})) \quad \Psi > -1.0$ $bw/tw = (222/\sqrt{fy})/((2+\Psi)) \leq -1.00 \quad \Psi \leq -1.00$
	Slender	Mmax. > My	Other Than The Above



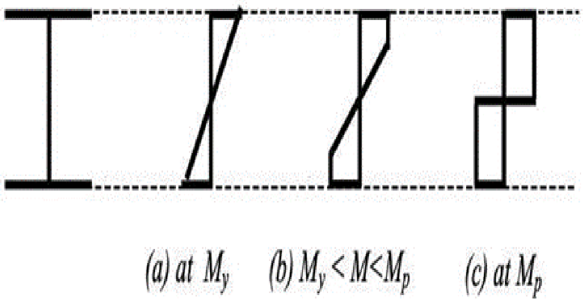


Fig. 5- Moment capacities of sections Owens P. (1994)[6] **Fig. 6- Plastification of cross-section under bending Owens P. (1994)[6]**

The reliability of the FE model of composite beams is validated by comparison with experimental study. FE is a very good solution when it is difficult to achieve traditional buckling theories and when there is no closed form solutions. Three-dimensional four-node shell element, SHELL43 **Fig. 5(a)** were used with three translations in x, y and z in each node to achieve the compatibility condition with translation in x, y and z in adjacent brick element to it. For this purpose, The element has plasticity, creep, stress stiffening, large deflection, and large strain capabilities.

An eight-node solid element, Solid65, **Fig. 5(b)** was used to model the concrete with three degrees of freedom at each node–translations in the nodal x, y, and z directions. The element is capable of plastic deformation, cracking in three orthogonal directions, and crushing. It is recommended by **Gupta et al. (2006)[3]** to ignore the

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compressive reinforcement in the concrete. In the same time, the steel wires perpendicular to the longitudinal direction of the beam were also recommended to be ignored by **Omarn et al. (2009)[17]** to avoid any effect on the strength and the behavior of the steel girder. Two types of buckling analysis are supported by **ANSYS** ~~[16]~~; Eigen values analysis and Non-Linear Buckling analysis. Nonlinear buckling analysis which is a system not subject to super-position as linear systems is usually the more appropriated and is recommended approach for evaluation of buckling structures.. However, **Subramani and Sugathan (2012)[7]** concluded to that the analysis using eigenvalue buckling method always had conservative result values and it is not recommended to be used in recent engineering studies. Initial imperfection is required to initiate the non-linear bucking of the plate.

VI. VALIDATION OF THE EXPERIMENTAL METHOD

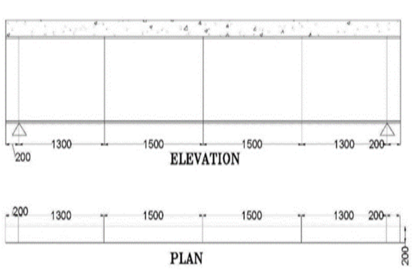

Fig.7-Composite girder experimental specimen

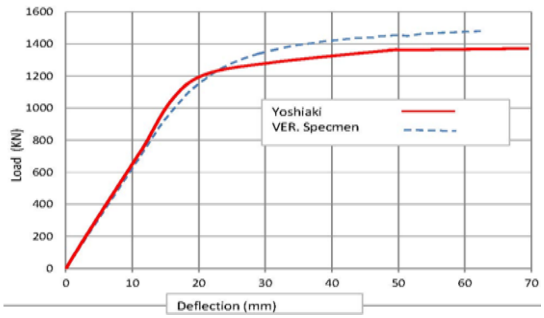

Fig. 8 - Load-Displacement curve experimental

Okui, Y. (2011)[18] [18][18], tested three composite girder specimens with different span lengths to investigate the shear capacity and interaction between bending moment and shear capacities. The development was done using **DIANA[19]**, a finite element software program **Fig. 8** showed one of the girder specimens with a span length of 5.6m loaded at the center of the specimen. The other two specimens have the same cross section but 7.0m and 9.0m span lengths in order to change the ratio between the bending moment and shear force. I shaped steel girders were designed with SM400A grade steel (Yield strength 300 MPa, Ultimate strength 450 MPa and 28% elongation) while the concrete strength of the slab is 45 MPa. [Table 2] showed a summary of the experimental results and an extracted output of Okui's study. A 9.00m span girder was used as a reference to validate the present ANSYS F.E. method followed in this study. The summary comparison of the outputs are shown in [Table 3] with the values of shear and bending moments. The comparison has shown better agreements with the F.E. Model include nonlinear buckling. **Fig. 9** is showing the Load – displacement curves of the Okui's experimental results relative to the same curve for the ANSYS verification model; the figure showed a good agreement of the two specimens except for that the displacement of experimental specimen extended about 12 mm after the plastic failure load.

VII. RESULTS AND DISCUSSIONS

Nine Finite element models were used to investigate the effect of the variables considered in the study,[Table 4] showed the specimens details. Moment capacity, yield and plastic moment for variable spans under positive moments were considered. [**Error! Reference source not found.** summarized the calculated moments and moments at failure for the F.E. specimens. Yield and plastic moments were calculated from section first principal, while M_{umax} and $M_{u\,comp}$ represent the FE's maximum moment at failure and, the moment at first concrete crushing at strain equal 0.0035. New status of the section is developed by comparing $M_{u\,comp\,to}$ M_y and M_p. As seen from [Table 5], the moment at failure increased due to considering the nonlinearities for both steel and concrete in addition to the stiffness of the concrete and accordingly the capacity of section to maintain loads up to failure extended, thus the codes equations are considered conservative.

The graphs shows the relation between α and b_w/t_w, **Fig. 10 to Fig. 12** showed the graphs for F.E. $\alpha = D_{cp}/t_w$ as shown on (**Fig. 4(a)** for compact) and $\alpha' = D_{cp}/t_w$ (**Fig. 4(b)** for Non-Compact outputs) for all the spans considered. **Figs. 13** showed the graphs for the three variable spans considered spans, relative to the slenderness limits of the other codes and also for the Gupta et al. (2006)[3][3]At each point of on the figures, there is an indication for the status of the section before/after using the composite FE study. The study lead to the following equation for limiting between compact and non-compact sections. Where F_y yield strength of web in N/mm^2 b_w web maximum dimension (length) in mm, t_w web minimum dimension (thickness) in mm and $\alpha = D_{cp}/b_w$

$$\alpha = (6e^{(0.0025*bw/tw)} + 0.058(\frac{bw}{tw})) * \frac{5/(bw/tw)}{\sqrt{Fy}} * 0.92$$

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Table 2- Experimental Verification Samples

	Experimental Results		Design Capacity	
	Bending Moment	Shear Force	Full Plastic Moment	Shear Capacity
	KN.m	KN	Mpl (KN.m)	Qu (KN)
Sp-1 (5.6m)	3298	1178	3061	1149
Sp-2 (7.0m)	3264	933	3033	1149
Sp-3 (9.0m)	3259	709	3127	1171

Table 3- ANSYS Verification Okui, Y.

	Shear Force KN	% of increase	Bending Moments KN.m	% of increase
Yoshiaki OKUI	709		3259	
Ver. Sample	741.2	104.5416	3480.4	106.7935

Table 4 – FE's Specimens Properties

		Length	Web		Flange					
		(m)	Dim (mm)	F_{yw} (t/cm^2)	Dim (mm)	F_{yf} (t/cm^2)	b_w (mm)	t_w (mm)	b_w/t_w	Status ECP-LRFD
Conc. Strength 0.40 Spec.	40A1	4.5	1000X20	3.6	200x20	2.4	1000	20	50	C
	40A2	4.5	1000X10	3.6	200x10	2.4	1000	10	100	NC
	40A3	4.5	1000X5	3.6	200x5	2.4	1000	5	200	S
	40B1	9	1000X20	3.6	200x20	2.4	1000	20	50	C
	40B2	9	1000X10	3.6	200x10	2.4	1000	10	100	NC
	40B3	9	1000X5	3.6	200x5	2.4	1000	5	200	S
	40C1	18	1000X20	3.6	200x20	2.4	1000	20	50	C
	40C2	18	1000X10	3.6	200x10	2.4	1000	10	100	NC
	40C3	18	1000X5	3.6	200x5	2.4	1000	5	200	S

Table 5- Output Results

		Composite Moments (ton.cm) (ton.cm)							
		M_y	M_{umax}	M_u	M_p	M_u/M_y	M_u/M_p	M_p/M_y	New Status
Concrete Strength 0.40 Spec.	40A1	21704.23	39060	38730	38340	1.78	1.01	1.77	C
	40A2	12606.23	17300	17300	24790.6	1.37	0.7	1.97	NC
	40A3	6926.078	2489	2489	16315.6	0.36	0.15	2.36	S
	40B1	21704.23	42603	38730	38340	1.78	1.01	1.77	C
	40B2	12606.23	22146.9	20440.5	24790.6	1.62	0.82	1.97	NC
	40B3	6926.078	14204	14204	16315.6	2.05	0.87	2.36	C
	40C1	21704.23	30210	30210	38340	1.39	0.79	1.77	N.C.
	40C2	12606.23	18585	18585	24790.6	1.47	0.75	1.97	N.C.
	40C3	6926.078	5620	5620	16315.6	0.81	0.34	2.36	S

VIII. FUTURE STUDIES

Other factors and influences. Using intermediate stiffeners and its influence on the span, width and thickness of the web from the economical point of view. The effect of residual stresses, especially when using hybrid welded sections to include the welding effect stress .The behavior of composite girders connected with flexible shear Connectors (partial integration) on the slenderness limits. This study is for steel with 2.4 t/cm^2 yield strength and concrete strength 0.40 t/cm^2. New studies are required to validate the proposed equation for higher slenderness classifications (between 50 and 100).

IX. CONCLUSIONS

A new released equations and new classification limit between compact and non-compact have been developed considering the effect of the concrete slab strength. The present study focused on using lower concrete strength equal 0.40 t/cm^2 with variable spans and concluded that the failure behavior of specimens and its slenderness were affected by their spans and concrete strength of composite girder. Lengths for limiting lateral unbraced for full plastic bending capacity(L_p), limiting lateral unbraced for inelastic lateral torsional buckling (L_r) and limiting for using plastic design(L_{pd}) have to be revised for the composite section This paper has offered a good preliminary approach with definite considered dimensions for steel and concrete sections, further research is still required on the implication of its recommendations, to enlarge the scope of application to cover wider range of sections' dimensions, and material specifications.

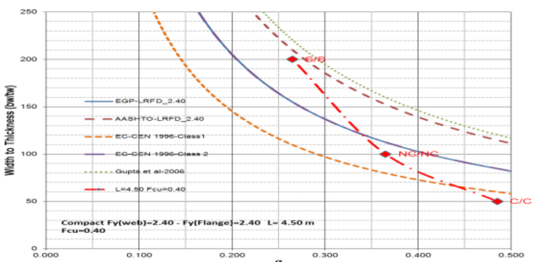

Fig. 9 Section Classification (L=4.50m)

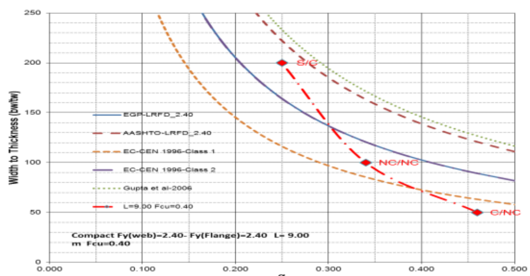

Fig. 10 Section Classification (L=9.00m)

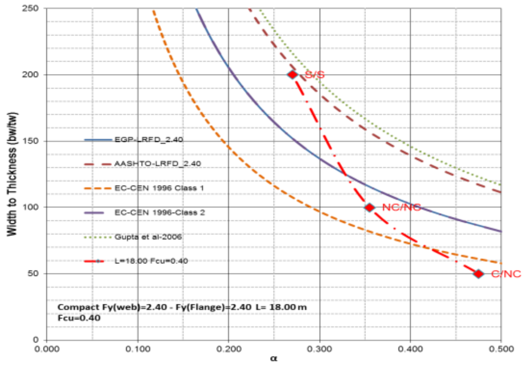

Fig. 11 Section Classification (L=18.00m)

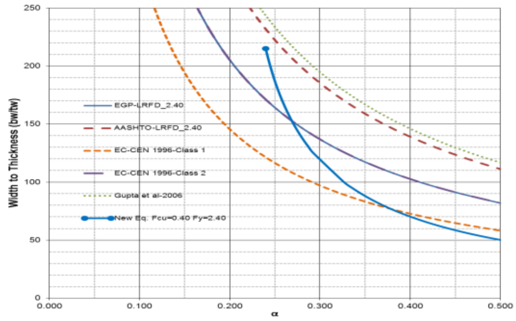

Fig. 12 Proposed Equation

REFERENCES

[1] ECP-LRFD (2012), Egyptian code of practice for steel construction-LRFD-205-First Edition, Housing and Building National Research Center, Cairo, Egypt.

[2] Sanker, M.B, Jacob P. A (2013), Comparison of Design Standards for Steel Railways Bridges, International Engineering of Research and Applications (IJERA) ISSN: 2248-9622 vol.3 Issue 2, March –April, 1131-1138.

[3] Gupta, V.K., Okui, Y. and Nagai, M. (2006), Development Of Web Slenderness Limits For Composite I-Girder Accounting For Initial Bending Moment, Structural Eng./Earthquake Eng., JSCE, Vol.23, No.2, October ,229-239.

[4] AASHTO (2005), American Association of State Highway and Transportation Official - LRFD; Bridge Design Specifications-2005-interim revisions, Washington D.C.

[5] EUROCODE (2001), Design of Composite Steel and Concrete Structures, N.4-85(2001), Common Unified Rules for Composite Steel and Concrete Structures, EUR 9886 EN., British Standards Institution, London.

[6] Owens P. (1994), Steel Designers manual', 5th edition, The Steel Construction Institute, ELBS Blackwell Scientific Publisher, chapter 8, local buckling, London.

[7] Subramani, T., Sugathan, A. (2012), Finite Element Analysis of Thin Walled-Shell Structures by ANSYS and LS-DYNA, International Journal of Modern Engineering Research (IJMER), Volume 2, Issue.4, July-Aug.,1576-1587, ISSN: 2249-6645.

[8] Taleb, C.E, Ammari, F. and Adman, R.(2015), Analytical Study of Buckling Profile Web Stability, Structural Engineering and Mechanics, An Int'l Journal Volume 53 Number 1.

[9] Musa I.A., (2016), Buckling of plates including effect of shear deformations: a hyperplastic formulation. Structural Engineering and Mechanics, An Int'l Journal Vol. 57 Number 6.

[10] Liu, J., Ding, F., Lui. X. and Yu, Z. (2016) , Study on Flexural Analysis of Simply Supported Steel-Concrete Composite Beam, Steel and composite structures, An int'l Journal Volume 21 No. 4,

[11] Prakash, A., Anandavalli, N., Madheswaran, C.K. and Lakashmanan, N. (2012), Experimental Investigation on flexural behavior of HSS Studs Connected Steel Concrete Composite Girders, Steel and composite structures, An int'l Journal Volume 13 No. 3.

[12] Gupta,V. K., Okui, Y. and Inaba, N. and Nagai, M. (2007); Effect of Concrete Crushing on Flexural Strength of the Steel-Concrete Composite Girder; Journal of Japanese Society of Civil Engineers JSCE (A),Volume 63, No.3 ,475-485.

[13] Duc, D.V., Okui, Y.(2014), Flexural Capacity of Composite Girders: Design Equation Accounting for Bridge High Performance Steel; Research Report of Department of Civil and Environmental Engineering, Saitama University, Volume 40.

[14] Duc, D.V., Matsuno, M.(2011), Influence of SBHS500 on Section Classification Criterion for Composite Girders. Japan Society for Civil Engineers, Annual Scientific Lecture 66th.

[15] Patil, P.S., Shaikh ,M.G. (2013), A study of Effect of Shear Connector in Composite Beam in Combined Bending and Shear by ANSYS. International Journal of Innovative Technology and Exploring Engineering (IJITEE) ISSN: 2278-3075, Volume 3, Issue-3, August.

ISSN: 2319-5967
ISO 9001:2008 Certified
International Journal of Engineering Science and Innovative Technology (IJESIT)
Volume 6, Issue 3, May 2017

[16] ANSYS , Swanson Analysis Systems, Finite Element Analysis (FEA) Version 9.0 and Documentation Theory Reference. 9th ed. Swanson Analysis Systems, Inc.

[17] Omarn, H.Y., Zangeneh, P and El-Hacha, R. (2009); Finite Element Modeling of Steel-Concrete Composite Beams Strengthened with Pre-stressed CFRP plate; The second Official International Conference of International Institute for FRP in Construction for Asia-Pacific Region; South Korea, December,187-192.

[18] Okui, Y. (2011), Recent Topics of Design Codes for steel and Composite Bridges, Proceedings of The Ninth China-Korea-Japan Symposium on Structural Steel Construction, China Steel Construction Society , China.

[19] DIANA, Acronym Displacement ANAlyser, Finite Element Analysis (FEA) solver TNO DIANA BV, 2002.

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