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FLEXURAL CAPACITY OF COMPOSITE GIRDERS ACCOUNTING FOR BRIDGE HIGH PERFORMANCE STEEL

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ABSTRACT: Elasto-plastic finite element analyses are used in a parametric study to investigate the positive flexural capacity of composite steel girders. The flexural capacity of steel-concrete composite girders using high-performance bridge steel (SBHS) is investigated with regard to web slenderness limits. Furthermore, an investigation is conducted regarding the impact of the initial (early) bending moment resulting from the unshored (unsupported) construction approach on the limits regarding web slenderness for the section classification. The findings from the composite girder's FE simulation model show that the flexural capacity of the composite bridge girders resisting a positive bending moment distribution, predicted by currently designed codes, is particularly conservative. The potential compact-noncompact limit is greater than that of AASHTO and Eurocode by around 50 and 70%, respectively. Many sections show acceptable flexural capacity as noncompact while being categorized as slender by current criteria.

Keywords: Bridge high-performance steel, Steel-concrete composite girders, Section classification, Web slenderness, Load-carrying capacity

1. INTRODUCTION

One of the most popular structural alternatives for medium-sized highway and railway bridge superstructures is the steel-concrete composite girder. By arranging the almost steel parts of the girder under tension and a concrete slab component under compression, this type of structure can combine steel and concrete elements for the maximum strength. The overall cost of composite building has recently changed because of the rise in labor costs in industrialized nations. Bridges with the composite girders have been developed with the tendency of simpler structures, for example, thicker web plates with the least quantity of stiffeners, to reduce the labor work. Therefore, it is important to take into account how the load-bearing capability of the steel girders is affected by the slenderness of the steel plates that make up the girder.

For girders that are simply supported, the mid-span zone is primarily affected by bending under vertical load types, the girder ends on support locations are mostly impacted by shear force due to reaction force from the bearings, and the zone nearby support locations is affected by a combination of bending moment and shear force. Based on experiments in [1], Nagai M., Inaba N., Okui Y., Miyashita T. and Hirayam came to the conclusion that the moment-shear interaction effect is minimal when the design of composite girders considers the bending and shear strength of composite girders independently. Due to the distribution feature of a strong bending moment and a low level of shear force, the flexural capacity of the mid-span zone should be taken into consideration when designing simply supported composite girders. According to Collings D., as mentioned in [2], the span-to-depth ratio for common composite girders falls between 18 and 20. With this scale, the girder can be categorized as a flexural dominant structure, and its flexural capacity can be seen as its critical strength.

Bridge high-performance steels (SBHS) are a new production of steel with 500 Mpa yield strength as a key advantage. Since 2008, it has been standardized as SBHS500 in the Japanese Industrial Standard (JIS) [3]. When compared to standard grade steels, the SBHS500 production shows advanced properties such as good weldability and high yield strength, but it also presents different inelastic behaviors, such as shorter ductility, almost no yield plateau, and larger yield ratio. Since 2014, the Japan Specification for Highway Bridges, a revision version 2012 [4], has permitted the use of this steel grade for the structure of bridges. According to the report of Ando R., Tanaka M., Takagi M., Homma K. in [5], this new steel grade has just so far been used in a small number of bridge projects, including the Nagata Bridge, Tokyo Gate Bridge, and Rinkai Chuo Bridge. However, the major load-carrying structures in the Tokyo Gate Bridge and Nagata Bridge projects are of the truss type, in which structural elements are mostly subject to longitudinal force. The use of SBHS500 steel grade in bridge structures has only been the subject of a small number of researches such as [6] and [7]; these studies, however, have focused on structures where local buckling behavior predominates.

Despite being approved for use in bridge structures, very few studies have been able to examine the flexural capacity of composite girders employing SBHS500 grade in order to design flexural composite girder bridges, so more thorough research need to be done.

The unshored approach has been used most frequently recently in the construction of steel-concrete composite girders for bridge superstructures. The key benefit is that there is no demand for a temporary support system. The steel girder must be able to withstand its own weight as well as the distributed load of wet concrete during the stage of concreting for the slab component. Before the composite behavior, the early bending moment in the steel girder is caused by the load component brought on by wet concrete. Due to the difficulty of accounting for the beginning bending moment on the steel girder alone, the effect of this bending moment was only taken into consideration in a small number of studies [8].

Table 1. Section classification based on Web slenderness limit specified by AASHTO [9] and Eurocode [10]

Section class	Flexural resistance	limit regarding web slenderness	
		AASHTO	Eurocode
Compact	$M_u \geq M_p$	$\frac{2D_{cp}}{t_w} \leq 3.76\sqrt{\frac{E_s}{f_y}}$	$b_w/t_w \leq \begin{cases} \frac{41.5\varepsilon}{\alpha} & \alpha \leq 0.5 \\ \frac{456\varepsilon}{13\alpha - 1} & \alpha > 0.5 \end{cases}$
Non-compact	$M_p \geq M_u \geq M_y$	$\frac{2D_c}{t_w} < 5.7\sqrt{\frac{E_s}{f_y}}$	$b_w/t_w \leq \begin{cases} \frac{42\varepsilon}{0.67 + 0.33\psi} & \psi \geq -1.0 \\ 62\varepsilon(1-\psi)\sqrt{-\psi} & \psi \leq -1.0 \end{cases}$
Slender	$M_y \geq M_u$	Otherwise	Otherwise

where M_u indicates maximum flexural capacity; Plastic moment M_p is defined as the moment at which the entire cross- section has attained its yield stress; Yield moment M_y is defined as the moment at which the top or bottom fiber of the girder's cross-section has reached its yield stress; Fig. 1 explains the symbols used in Table 1.

A steel-concrete composite girder's flexural strength can be predicted throughout the structural design phase using section classification. The flexural strength of the girder can be assessed using M_p and M_y values based on geometrical data of the girder section. Theoretically speaking, M_p and M_y represent the plastic and yield flexural strengths of a steel-concrete composite girder, explained in Fig.1. The geometrical information used to categorize the composite girder section in this study is the web slenderness. Fig. 1 explains the web slenderness with values for D_{cp}, D_c, and α. Table 1 provides a summary of the section classification in the existing design codes based on web slenderness.

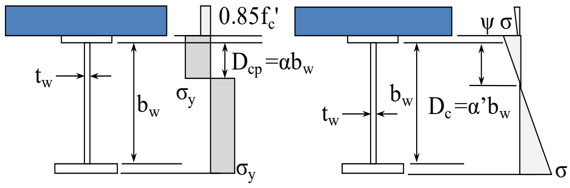

Fig.1 The use of geometrical parameters in existing design codes

A steel-concrete composite girder's flexural strength can be predicted throughout the structural design phase using section classification. The flexural strength of the girder can be assessed using M_p and M_y values based on geometrical data of the girder section. Theoretically speaking, M_p and M_y represent the plastic and yield flexural strengths of a steel-concrete composite girder, explained in Fig.1. The geometrical information used to categorize the composite girder section in this study is the web slenderness. Fig. 1 explains the web slenderness with values for D_{cp}, D_c, and α. Table 1 provides a summary of the section classification in the existing design codes based on web slenderness.

The distribution of the early bending moment, which is induced in the steel girder during the slab concreting stages, needs to be taken into consideration as a factor affecting flexural resistance when referring to the erection of the bridge superstructure using the un-shored solution. The influence of the early bending moment on the categorization of composite girders based on the web slenderness limit for section was presented by Gupta V.K., Okui Y., and Nagai M. in [11]. These authors discovered that the initial bending moment had a major impact on the noncompact-slender limit regarding the web slenderness. The research states that both ultimate flexural resistance M_u and yield moment M_y drop with an increase in the early bending moment M_I level brought on by the use of the unshored concreting process; however, the reduction in My value is greater than the reduction in M_u value. The Noncompact-Slender limit may significantly increase as a result of this occurrence. Despite the fact that it produced significant results on investigating the influence of begining bending moment on the noncompact-slender web slenderness limit the study published in [11] only took into account conventional steel grade SM490Y when it came to composite girders with the homogeneous steel section. Due to their aesthetic looks and cost-effectiveness, twin-I composite girder viaducts have become increasingly often used in construction. Due to the weight of the larger concrete slab being distributed to the lesser main steel girders, this superstructure style would result in a higher amount of early bending moment. The investigation of Gupta's group in [11] examined only at a maximum early bending moment level of 0.4 M_{ys}. Due to the wet concrete slab part's actual condition, this level of early bending moment might not be compatible with acting loading. Applying the ideas from [11] to forecast the flexural capacity of steel-concrete composite girders employing the SBHS500 grade would be questionable.

The investigation into the flexural capabilities of composite girders with respect to the new steel grade SBHS500 and the influence of the begining bending moment was documented in [12, 13]. Nevertheless, a general and brief report of the investigation results on the noncompact-slender and compact-noncompact limits regarding the web slenderness was given in [12]. Reference [13] provided and discussed the results for the compact-noncompact and noncompact-slender web slenderness limitations that only account for the begining bending moment level of M_l=0.4M_{ys}.

2. RESEARCH SIGNIFICANCE

This study examines how elevated early bending moments from the unshored construction method and the inelastic behavior of the new SBHS500 steel influence the section classification of steel–concrete composite girders, focusing on web slenderness limits. By developing structural simulation models with SBHS500 and conventional SM490Y steels in both homogeneous and hybrid girders, the study numerically evaluates flexural capacity and derives web slenderness limitations. These results are compared against AASHTO and Eurocode regulations. The research highlights the effects of early bending moments and inelastic steel properties on girder performance, offering insights for optimizing composite girder design with advanced steel grades..

3. NUMERICAL STRUCTURAL MODEL

3.1 Investigation structure

The structural model to investigate flexural capacity of the steel-concrete composite girder is presented in Fig.2. The girder model's length, web height of the steel girders and slab thickness are fixed at 9 meters, 3m and 0.3m respectively. As previously indicated, small-sized I-shaped steel transverse beams were used to connect the twin main I-girders at spacing of 5 to 10 meters; the 9 m length is approximately close to the range's maximum length. Targeting the twin, I-girder type, the concrete slab is only supported by two primary steel girders. In practical designs, deep steel girders and a 300mm thick slab are quite common. The size effects in concrete members can be avoided by scaling the structural model depending on the actual building design.

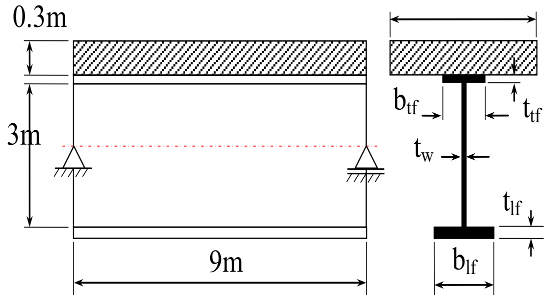

Fig.2 The analysis structure in the current study

In the current research, the limit quantities regarding the web slenderness for the section categorization can be accessed using two key values α'=D_c/b_w for Non-compact section and α=D_{cp}/b_w for Compact section. To consider different levels of compression depth in web plates under D_c and D_{cp}, the current research work analyzed hundreds of simulated composite girder structures while taking into account varied values of web thickness t_w, top and bottom flange width b_{uf} and b_{lf}, concrete slab width b_c, and top and bottom flange thickness t_{uf} and t_{lf}, respectively.

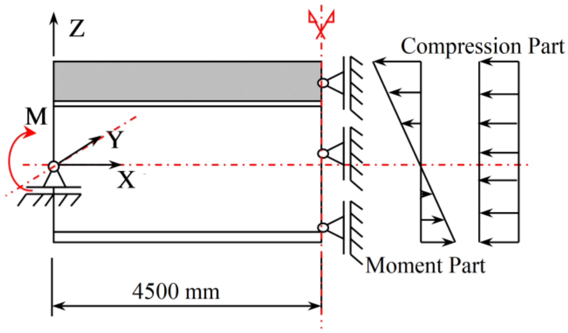

Fig.3 Boundary condition of the simulated structural model

Based on the symmetrical rule, a half of the structure would be taken into consideration, which reduced the process work with a lot of simulated structural elements. As a result, a corresponding boundary condition was set up as shown in Fig.3. The central surface of the model composite girder was designated for the symmetrical condition required as support following X axis and free displacement following Y and Z axes, as shown in the figures. The support at the girder end was designated to freely displace along the X axis in order to produce a pure bending at the support. Following this, bending moment and a reaction of compression happened on the central surface at an axis corresponding to the girder support. However, the reaction of volume compression did not occur due to the force balancing requirement, and the simulated composite girder structure only generated a pure bending moment.

The incremental load steps for the structural model are created using the displacement control method. As shown in Fig. 4, a rotation about the y axis at the support is required to incrementally create a pure bending moment.

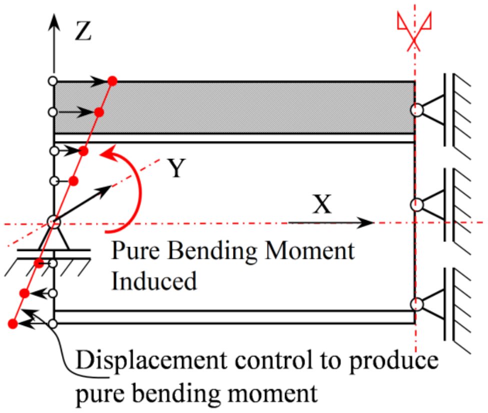

Fig.4 Pure bending moment induced by displacement control in the simulation model

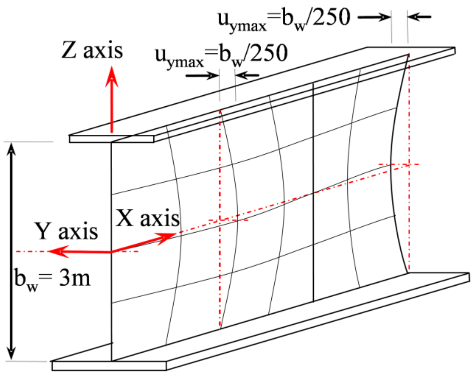

Fig.5 The initial geometric imperfection in the web plate

The simulation model for steel-concrete composite girders should take into account the residual stress distribution brought on by the high temperatures caused by welding. Nonetheless, the compressive range of the web plate created by loading is overlapped by the residual stress distribution's tensile strain range, which is located adjacent to the web's edge lines. The local buckling in the web plate caused by the compressive stress distribution level can be lessened by the combination of these processes. In order to maintain the safe side assumption when examining the web slenderness limits, the residual stress distribution in the web plate caused by the welding process is disregarded in the FE simulation model. The impact of residual stress on the flexural capacity is also reported by Oehlers and Bradford in [14].

The initial imperfection of the geometrical web out-of-plan deflection in the web plate is taken into consideration as a sinusoidal surface specified in Eq. 1 and seen in Fig. 5. Japanese Specification for Highway Bridges [4] provides a value of $b_w/250$ as the upper limit of initial out-of-plane web deflection.

$$u_y = \frac{b_w}{250}\sin\left(\frac{\pi}{b_w}z\right)\cos\left(\frac{\pi}{b_w}x\right) \tag{1}$$

where b_w, x, y, z are explained in Fig.5.

By employing separate analysis phases, the effects of the initial bending moment distributed along the steel girder were taken into consideration. Due to its own weight and the weight of the wet concrete, the steel girder section is only subject to pure bending moment action during the first analysis phase. The concrete slab component is activated in the second step to provide the composite behavior. Analysis of the upgraded structure is ongoing till ultimate state.

3.2 Non-linear elasto-plastic models

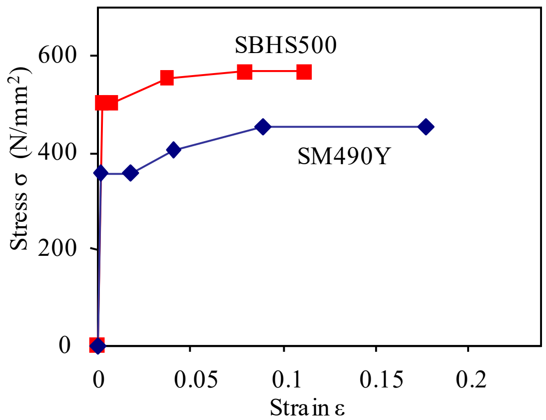

Fig.6 The idealized stress-strain relation curves of SM49Y and SBHS500 used in numerical model

To investigate the impact of early bending moment distribution in the steel girder due to unshored construction method on section classification of composite girders based on accessing the web slenderness limits, the current study applied the non-linear elasto-plastic FEM model proposed in [11]. Two material models have been employed in this work to estimate the ultimate state of the composite girders.

Steel materials' elasto-plastic behavior is modeled using the von Mises yield criterion and the Prandtl-Reuss equation. As a result of experimental findings, the uniaxial multi-linear stress-strain relation curves of steel grades SM490Y for web plates and SBHS500 for flange plates are depicted in Fig.6.

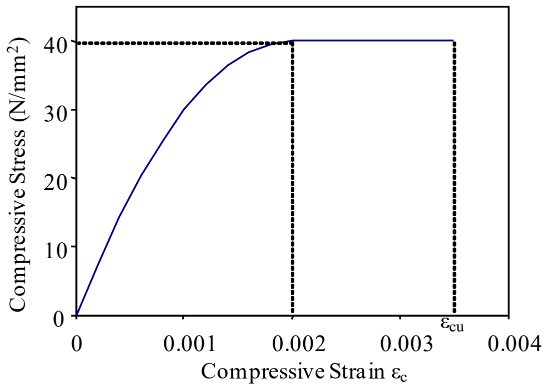

Fig.7 The idealized stress-strain curves of concrete material model

AbdElrahman A.M., Hussein M.M., Attia W.A.L. documented in [15] the negative effects of raising the concrete slab's compressive strength in relation to the ratios of M_u/M_p and M_u/M_y. However, current study has assigned a value of 40 MPa to the compressive strength of concrete material in order to deal with conservative trends and agree to high strength steel girders. The assumptions of plastic hardening material, Mohr-Coulomb yield criterion, and linear elastic characteristics are used to represent concrete under compression. Based on a proposal published in JSCE [16] the uniaxial stress-strain curve for concrete under compression is defined and shown in Fig 7.

The four-node quadrilateral isoparametric curved thin shell element (Q20SH) from the DIANA software package DIANA [17] was used to discretize the steel girder's web and flange components. The element with five degrees of freedom (3 translations and 2 rotations) per node agrees to model local buckling deformations, as presented in Fig 8, and the spread of plasticity effects.

Eight-node isoparametric solid element (HX24L) from the DIANA software package was applied to mesh the concrete slab component, with three degrees of freedom at each node to simulate inelastic deformation, cracking in three orthogonal directions, and crushing. Positive bending moments would cause concrete slab parts to collapse. Additionally, the flexural strength of the composite girder models is calculated using the concrete's compressive strain, which has a range of 0.0035. As a result, modeling of the reinforcing in the concrete slab component can be ignored.

Phase analysis was used to take the early bending moment into consideration. As seen in Fig.9, the initial phase just included the steel girder model component, which represented the steel girder's own weight as well as the weight of wet concrete and construction loads. Activating solid elements, which were presenting the hardened concrete material, in the second phase indicated the beginning of the composite action. To identify the flexural capacity of the composite girder including the concrete slab part, the displacement control would be kept gradually increasing.

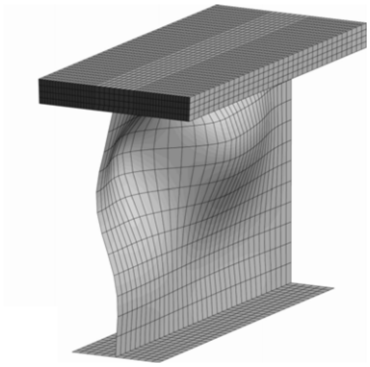

Fig.8 Local buckling mode in the simulation model

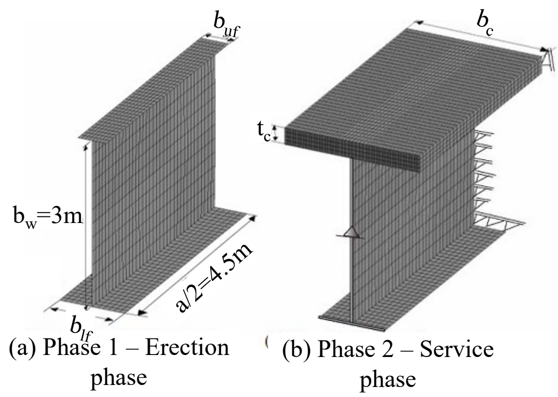

Fig.9 FE models for phase analysis

4. NUMERICAL APPROACH

In this study, comparing the ultimate flexural capacity M_u to yield moment M_y and the plastic moment M_p constitutes the basis of the current study's investigation into the section classification of composite girders. If $M_u \geq M_p$, the section is considered as compact. If $M_p > M_u \geq M_y$, the section is categorized as noncompact; and if $M_u < M_y$, the section is classified as slender.

As previously indicated, buckling has a significant impact on the flexural strength of steel-concrete composite girders. Additionally, it can be estimated employing the girder's section classification. The modes of local buckling at the top flange and global buckling of the girder under positive bending moment distribution can be disregarded once the concrete material has begun to harden because a thick and solid concrete slab prevents deflection of the steel girder's upper flange and maintains a transversal rigidity. The web slenderness limit is the only criteria used to classify sections in the service stage since the flexural capacity of the girder is primarily dependent on the local buckling mode of the compressive region of web plates. Table 1 shows the section classification criteria used by the current AASHTO and Eurocode, where M_u stands for the composite girder's flexural capacity and M_y and M_p stand for the yield and plastic moments, respectively. Regarding the M_p quantity, Roik E.h.K., Bergmann R., Haensel J. and Hanswille G. stated in [18] that for the girders with compact class sections, the impact of the beginning bending moment can be omitted because the ultimate loads acting on the girder with concrete slab constructed by supported and un-supported method are similar. According to the study's report in [11] the beginning moment was taken into account in simulated composite girder models to determine how it would affect the section classification's web slenderness limit.

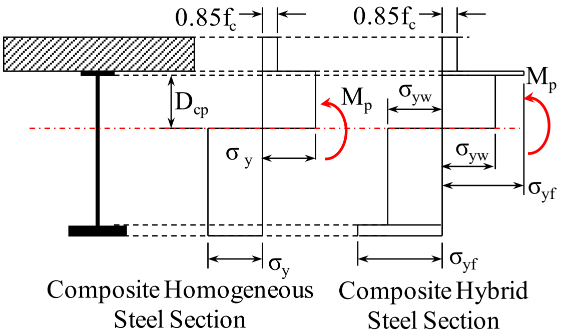

Fig.10 The stress distribution assumption for homogeneous and hybrid steel sections

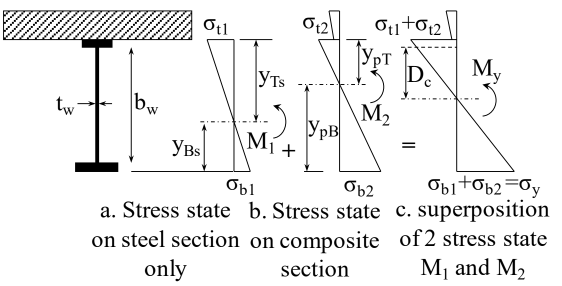

Fig.11 Yield bending moment of composite section with considering the initial bending moment

The plastic moment M_p is a theoretical quantity with the assumptions that the strain of the entire cross-section area has attained its yield level and the inelastic behavior of steel material complies with the assumption of being perfectly plastic.

The idealized stress distribution for hybrid and homogeneous sections is shown in Fig. 10 in accordance with the theoretical assumption. The yield level of the corresponding steel grade in a homogeneous section is denoted by the symbol σ_y, whereas the symbols for the corresponding steel grades in web plates and flanges are denoted by f_{yw} and f_{yf}, respectively. The compression height in a web plate is known as D_{cp}. The concrete material's intended compressive strength is f'_c.

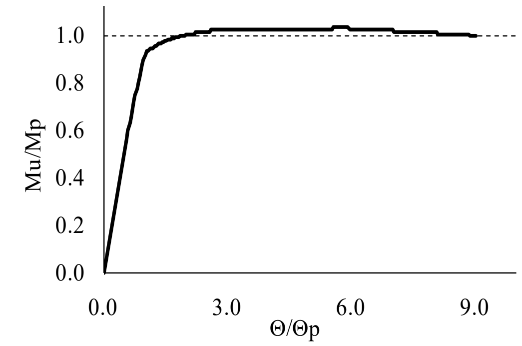

Fig.12 Section classified as compact

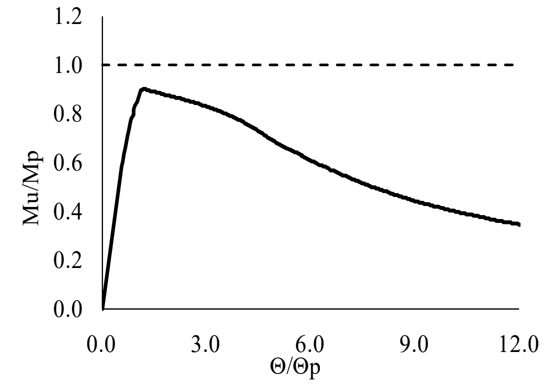

Fig.13 Section classified as non-compact

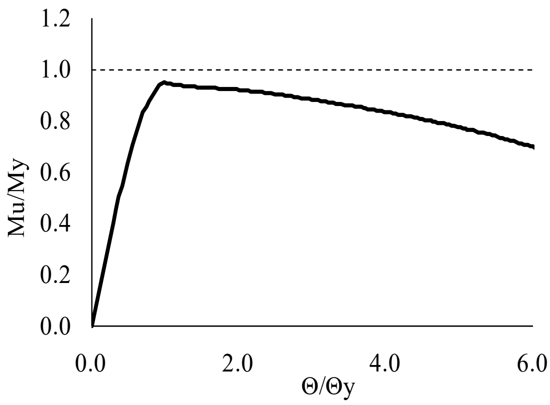

Fig.14 Section classified as slender

Yield moment M_y is calculated with the assumption that the yield value began to reach at either extreme fiber of the flange. The initial bending moment's effect is considered when calculating the yield moment M_y. The labels σ_b and σ_{t1} present the stress levels induced by the early flexural moment M_1 in the bottom and top flanges, respectively. The stress values σ_{t2} and σ_{b2} are produced by M_2 for the top and bottom steel section of the composite girder. Through the superposition of the two stress distributions shown in Fig. 11 the image shows that $\sigma_{b1} + \sigma_{b2} = \sigma_y$, as in practical design, the yield stress is considered to be reached at the extreme bottom flange of the steel girder. D_c value presents the compressive depth of a web plate when composite behavior begins in the girder. The current research also used the FE simulations since it was a more reasonable choice according to the potential cost compared to the experimental method. The findings of the ultimate bending moment M_u, compression parameters α and α', and width-to-thickness ratio b_w/t_w are collected from a sufficient number of girder models in order to estimate the web slenderness limit of composite girders with application of new steel grade SBHS500.

The compact, non-compact, and slender classes are each represented in Figures 12, 13, and 14, respectively. In the pictures, Θ_p and Θ_y represent theoretical rotations that correspond to elastic behavior while taking into account plastic and yield moments, respectively.

5. WEB SLENDERNESS LIMITS

5.1. The limits of compact-noncompact sections

5.1.1. Girder with SBHS500 homogeneous sections

Figure 15 shows the results of FE simulation for composite girders with the SBHS500 homogeneous steel section with the limits regarding web slenderness set by Eurocode, AASHTO, and the results reported in [11], where *α and* b_w/t_w stand for the compression region of the web plate and the width-thickness ratio, respectively. In the Fig. 15, the red symbols represent noncompact sections while the black symbols represent simulation results that have been verified as compact sections. According to Gupta, the compact-noncompact limit considered for the section classification is unaffected by the first bending moment, hence the beginning bending moment was not considered in this study.

Comparing the analysis results of composite girders with SBHS500 homogeneous steel section to those of Eurocode, AASHTO, and those suggested in [11], it can be shown that the web slenderness limit is significantly higher. The primary explanation appears to be the inelastic behavior of SBHS500 steel. In comparison to ordinary steel, SBHS500 can create a member with stronger local buckling resistance in the compressive zone of the web plate due to its higher yield level and the smaller plateau range. The web slenderness limit of compact-noncompact class of the composite girder with SBHS500 homogeneous steel section is roughly 25% higher than that of the composite section with SM490Y steel and the AASHTO specification, and roughly 50% higher than that of the Eurocode specification.

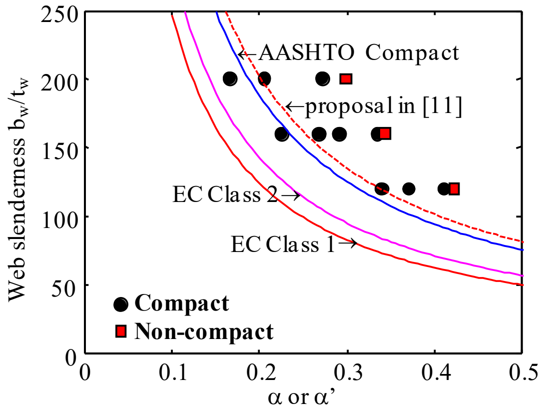

Fig.15 The limits of Compact-noncompact homogeneous SBHS500 steel sections

5.1.2. Girder with SBHS500-SM490Y hybrid steel sections

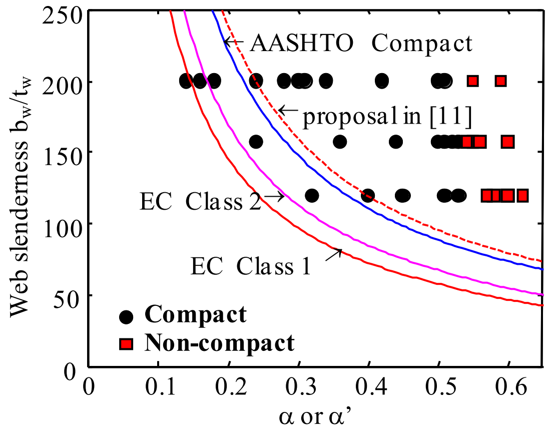

Fig.16 Compact-noncompact limit of hybrid SBHS500-SM490Y sections (M_1=0)

Plotted beside the limit equations of the compact-noncompact slenderness regulated by Eurocode and AASHTO in Fig. 16 are the results of FE simulation of composite girders applying the SBHS500-SM490Y hybrid sections. The findings show a greater variety of the compact-noncompact limits than those set by Eurocode and AASHTO. The limit for the hybrid sections is also higher than that of the homogeneous SBHS500 sections, as shown by the comparison in Figs. 15 and 16. Additionally, as demonstrated in those figures, the web slenderness border area regarding the composite girder with SBHS500-SM490Y hybrid steel sections exhibits a steeper general tendency than that of the compact-noncompact limits and the area regarding those of the girders with SBHS500 homogeneous steel sections.

The results shown in Fig. 16 also indicate that the maximum web slenderness regarding compact sections of the SBHS500-SM490Y hybrid composite girders varies around the value 0.5 and that the vertical neutral axis of the plastic section is around the central location. As a result, the steel has reached its final state of plasticity in the zones near both of the web plate's edge sides. The ultimate state can then be regarded as being related to the local buckling mode of a rectangular plate with three clamped and a free edge which can be understood as an area that prematurely yields adjacent to the edge ends of the web. This might be the primary factor behind the steeper tendency of the web slenderness compact-noncompact limit regarding the girder with the hybrid SBHS500-SM490Y sections as compared to that of the girders with homogeneous SBHS500 sections and ones as prescribed by AASHTO, Eurocode, and the Gupta proposal, in which the flexural capacity of the composite girder is associated with the local buckling mode of a rectangular plate with four clamped edges. The acquired results further show that the steel SBHS500 in the bottom flange of the girder, which has an essentially low plateau range, can reach a hardening range that is much higher than the yield level. Due to this behavior, the FE simulation composite girders can achieve flexural strengths of M_p level even though sections do not fully develop plasticity as is assumed in theory. In addition, the girder's ultimate flexural strength is substantially higher than the M_p value.

5.2. The limit of noncompact-slender sections

The evaluation limit regarding the web slenderness quantity, which is based on the values of the criteria M_u/M_y, must also take into account the variation in M_u and M_y values brought on by the initial bending moment M_1. With application of SM490 steel and an early bending moment M_1 that varied between 0 and 0.4 steel section's yield moment M_{ys}, research in [11] reported that M_1 had a significant impact on the noncompact-slender limit. In this work, the increase in compressive depth D_c, which favors local buckling behavior more, is used to explain why the maximum flexural moment M_u decreased.

In order to investigate the limit of a noncompact-slender with early bending moments of M_1 = 0, 0.2, 0.4, and 0.6M_{ys}, respectively, numerical results are presented in Fig. 17, 18, 19, and 20. The black symbols in these figures indicate composite girder simulation results that were categorized as noncompact sections, while the red symbols correspond to thin sections. The noncompact-slender limit of the girders with the SBHS500 homogeneous sections is approximately 20% greater than that of the girders with the homogeneous SM490Y steel sections and approximately 50% greater than that of AASHTO and Eurocode for the simulation FE girders, in which the early bending moment is not taken into account (fig. 17). The general noncompact-slender limit tendency for the girders with homogeneous SBHS500 sections is significantly different from that of the girders with SM490Y homogeneous steel sections, which has a steeper trend according to AASHTO and Eurocode.

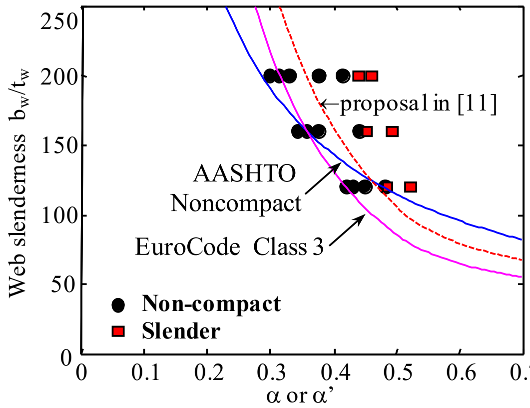

Fig.17 Limit of Noncompact-slender regarding homogeneous SBHS500 sections (M_1=0)

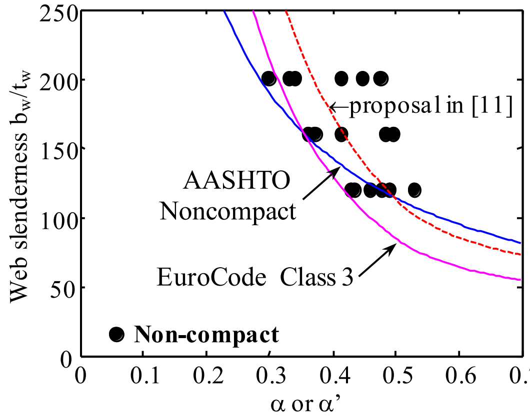

Fig.18 Limit of Noncompact-slender of girders with homogeneous SBHS500 sections (M_1=0.2M_{ys})

The analysis's findings show that, compared to the situation when M_1 = 0.2 Mys, the noncompact-slender limit regarding the composite girders with SBHS500 homogeneous sections is significantly lower. Two simulation girders are marked as slender as the initial bending moment is increased (from M_1=0.2M_{ys} to M_1=0.4M_{ys}). The opposing tendency, as observed in [11], is the behavior of decreasing the limit regarding web slenderness while increasing the beginning bending moment from M_1=0.2M_{ys} to M_1=0.4M_{ys}. For the early bending moment M_1=0.4M_{ys}, the limit for the girders with homogeneous SBHS500 sections is around 15% greater than that of the girders with homogeneous SM490Y sections. The feasible limit zone obtained from investigative simulative results of the composite girders with homogeneous SBHS500 sections with M_1=0.4M_{ys} is about 50% and 35% greater, respectively, than the noncompact-slender limits established by Eurocode and AASHTO.

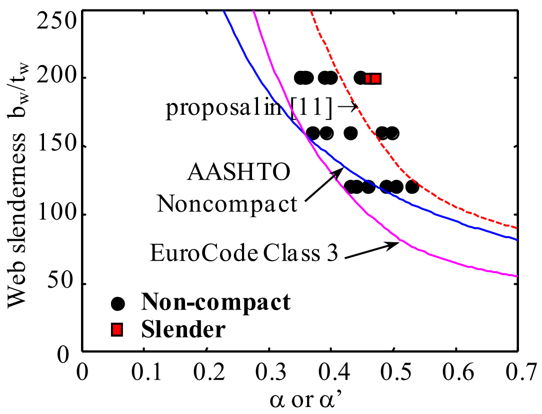

Fig.19 Noncompact-slender limit of homogeneous SBHS500 sections (M_1=0.4M_{ys})

None of the simulation girders exhibit slender classification for M_1 = 0.2M_{ys}. In this situation, the M_u and M_y results show that the yield moment M_y results are reduced more dramatically than the corresponding ultimate flexural moment M_u. It is possible to draw the conclusion that the web slenderness limit significantly increases with this level of the beginning bending moment (M_1=0.2M_{ys}) as compared to the scenario when the first bending moment is not taken into account.

According to the analysis's findings (Fig. 20), the limit increased by roughly 10% for M_1= 0.6M_{ys} in comparison to M_1= 0.4M_{ys}. This limit increase is consistent with the trend noted in [11]. Additionally, the noncompact-slender limit for the case of the girders with homogeneous SBHS500 sections for M_1=0.6M_{ys} is quite similar to the limit for the homogeneous SM490Y steel sections as reported by authors in [11].

In contrast to what authors stated in [11], the reduction in the noncompact-slender limit regarding web slenderness when the early bending level M_1 is increased from 0.2 to 0.4 M_{ys} in the numerical composite girders using SBHS500 homogeneous steel sections exhibits the reverse tendency. The following could be used to explain this phenomenon. The ultimate state of the girders with the homogeneous SM490Y sections is associated with local buckling of the rectangular plate with three simply supported edges and one free edge in the connection zone between the web and top flange plates due to the considerable plateau range of the steel material. For the girders with SBHS500 steel sections, the connection zone is more rigid in the ultimate state because of the very short plateau range and significant slope in the hardening range. Hence, for $M_1 \leq 0.4M_{ys}$, the ultimate state of the girders with homogeneous SBHS500 sections is associated with buckling of the 4-edge-simply-supported rectangular plate, obtaining greater strength than that of the plate with three simply supported edges and one free edge. For $M_1 > 0.4M_{ys}$, the steel material in the top flange and the connection zone both reached the inelastic range; the "free edge" of the representative rectangular plate of the girders with the SBHS500 section is still more rigid than that of the girders with SM490Y, hence inducing the better ultimate flexural strength.

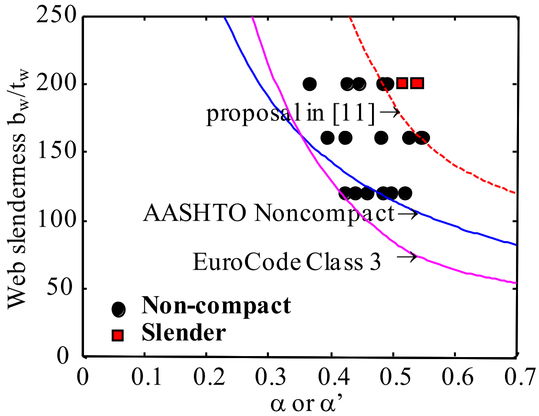

Fig.20 Noncompact-slender limit of homogeneous SBHS500 sections (M_1=0.6M_{ys})

The α and α' values for the girders designed in practice are typically lower than the value of 0.5. Nearly all composite sections with the SBHS500 homogeneous steel parts are categorized as noncompact within this range of α and α' values and for the degree of the beginning bending moment M_1>0.2M_{ys}.

The yield moment M_{yf} of the composite girders with hybrid steel section is defined by the following function: R_h*M_y, where M_y is the corresponding homogeneous section and Rh is the relevant hybrid factor. In [19] Schilling however, based only on study of the hybrid steel girder section, developed the hybrid factor that is currently employed by AASHTO. Furthermore, Schilling's derivation overlooked the impact of the initial bending moment as a result of the usual un-shored building technique. Another scientific publication will present the hybrid factor R_h suggestion that takes the early bending moment into account.

6. CONCLUSION REMARKS

The compact-noncompact and noncompact-slender limits for section classification can both be greatly increased by using SBHS500 steel in composite girders with homogeneous steel sections. Based on simulation results from the current study, the potential compact-noncompact web slenderness limit zone is greater than that of Eurocode and AASHTO by around 70 and 50%, respectively. The noncompact-slender limits observed from FEM data show the increase as well when the increasing starting bending moment is taken into account; this is a tendency that is similar to that found by authors reported in [11] but 15% to 20% more pronounced.

The compact-noncompact limit zone regarding web slenderness observed from analytical simulation results is considerably greater than that specified in AASHTO, Eurocode, and the proposal in [11] for the case of investigating the composite girders with hybrid SBHS500-SM490Y steel sections. For investigating the compact-noncompact web slenderness limit, the limit zone of the composite girders with hybrid SBHS500-SM490Y steel sections is larger and steeper than that of the girders with homogeneous SBHS500 steel sections.

In practice α and α' values for the level of early flexural moment M_1>0.2M_{ys}; nearly all composite sections with SBHS500 homogeneous steel parts are categorized as at least noncompact.

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