

Article

Influence of an Innovative Corrugated High-Strength Steel Profile on Soil–Steel Composite Bridges

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Abstract

Composite soil–steel corrugated bridges, which are widely used in road, railway, and civil engineering, are recognized as durable, sustainable, and cost-effective structures. Due to their interactions with the surrounding soil, relatively thin corrugated steel plates are usually used in these bridges. Larger spans are associated with larger cross-sections, and deep corrugations with a 500 mm pitch and a 237 mm depth are already in use worldwide. However, the behavioral benefits of high-strength steel and additional strengthening elements for CSS structures have rarely been investigated with regard to local buckling in the straight regions of the corrugation. This study analyzed the influence of high-strength steel and innovative corrugated cross-sections strengthened with circular steel pipes on the utilization ratio of steel plates in composite soil–steel structures. Two-dimensional numerical models of three bridges with spans of 26 m, 17.5 m, and 12 m and surrounded by soil were developed to identify internal forces from permanent and temporary actions. Plate utilization was designed according to the Swedish, Canadian, and American methods, considering local buckling in the 500×237 mm and 381×140 mm corrugation profiles. It was found that the use of higher-strength steel material, as well as the introduction of steel pipes, significantly reduced the plate thickness of regular corrugations. The results show that the use of higher-strength steel reduced the cross-section area of regular and innovative corrugations by 30–40%. Moreover, the cross-section area of the innovative profile was 5% to 36% lower than that of the regular corrugation profile. Nevertheless, the results show that the local buckling approach proposed by the Swedish design method could be considered conservative and should be revised. In addition, the method of preventing local buckling by reducing the plastic moment capacity could be neglected when using thicker plates and lower steel grades.

Keywords: corrugated steel structure; soil–steel structure; composite bridge; innovative corrugation; local buckling; high-strength steel



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1. Introduction

Corrugated soil–steel (CSS) composite structures applied in road, railway, and civil engineering are characterized by a light weight, a high load-bearing capacity, durability, cost-effectiveness, and sustainability [1–6]. The steel plates of these structures are usually relatively thin due to the mechanical properties of the corrugation and the interactions with the surrounding soil. Larger corrugation cross-sections enable thinner plates, wider spans, or a higher load-bearing capacity in CSS bridges. The use of deep corrugations with a 500 mm pitch and a 237 mm depth enabled the construction of the largest structure, with

a span of 32.4 m [7]. High-strength steel and additional strengthening techniques provide benefits that could allow this limit to be increased; however, studies on their impacts are scarce.

In general, CSS structures and their responses to dead and live loads have been intensively monitored and investigated with the aim of understanding the limitations of existing design approaches [3]. Many numerical investigations have compared the results obtained with such approaches [8–14], and full-scale destructive tests have been performed to validate the data and improve applicable calculation methods [15–20]. Various strengthening techniques for CSS structures have been proposed and studied to achieve specific stiffness values and potentially better behavior in interactions with the surrounding soil [7,21–25].

Maleska and Beben [25] studied whether extra stiffening elements are needed in soil–steel bridge design. They modeled stiffening steel ribs filled with concrete and compared the numerical results with field measurements. Finite element internal forces were also compared with outcomes from relevant standards and design methods. Beben and Stryczek [23] presented a numerical analysis of a corrugated steel plate bridge with a reinforced concrete relieving slab under static loads. They studied the slab's effect on shell deformations and compared the results with experimental tests. Displacements were reduced by 53–66% and stresses by 73–82%. Embaby et al. [7] examined the performance of lateral steel mesh reinforcement and circumferential steel stiffeners in the world's largest CSS structure, with a 32.40 m span. A three-dimensional finite element model was validated against field measurements. The results showed that steel meshes reduced induced strain actions by up to 50% and that the circumferential steel stiffeners could keep crown vertical deformations below 0.5%. Embaby et al. [24] also examined the beveling slopes of structure ends. The results showed notable longitudinal bending moments in steel plates that were not supported by the full ring. However, it was found that the use of steel meshes around the CSS structure decreased circumferential axial stresses by up to 30%. Bareikis [22] examined how a rational layout of steel mesh affects the behavior of a large-span, deep corrugated steel structure. Finite element results showed that steel meshes can be effectively used to limit CSS structure deformations and internal forces. The study found that crown vertical displacement was reduced by 45% during the peaking stage. It also concluded that steel meshes can increase the structure's load-bearing capacity. Chen et al. [26] and Sun et al. [27] studied local buckling effects of the straight region of the corrugation of CSS structures. Width-to-thickness ratios were proposed to prevent local buckling. Chen et al. [26] validated suggested limitations on pipe–arch-type structures. Sun et al. [27] provided strength design recommendations for deeply buried circular corrugated steel arches with various rise-to-span ratios, for both axial compression alone and combined axial compression and bending.

The use of a higher steel grade is associated with fatigue and local buckling issues in the design of conventional girder bridges and structures. Fatigue is not usually a decisive factor in the design of CSS structures because of the impact on soil cover and there being no welded parts. However, larger corrugation wave cross-section dimensions impact the resistance to local buckling in the straight region of the corrugation, especially when using higher steel grades. Regarding production technology and standard procedures of plate corrugation and rolling, it is almost impossible to stiffen corrugation using stiffeners. According to the results presented by Bareikis [28], increasing the plate thickness does not always provide a rational design either. Other strengthening techniques, for example, the application of hollow-section circular steel pipes, increase not only the load-bearing capacity of the structure but also the cross-section resistance to local buckling by reducing the buckling length of the straight region of the corrugation [21].

Local buckling is addressed in the Swedish design method (SDM) of corrugated soil–steel composite bridges [29]. Although it notes that commonly used systems employ relatively thick corrugated steel plates for which local buckling is not a concern, it still requires the corrugation to be examined for local buckling. In comparison, the CHBDC [30] and AASHTO [31] methods do not include any rules regarding local buckling prevention during the design of corrugated soil–steel composite systems. Considering this, an analysis was conducted to examine how the results are impacted by the local buckling relationship presented in the SDM [29]. It is important to understand the sensitivity of the results to the corrugation cross-section parameters, variables such as plate thickness, and the length of the straight region of the corrugation (tangent or buckling length).

In the current study, a numerical investigation was conducted on three low-profile, two-radius arches with spans of 26 m, 17.5 m, and 12 m and buried in soil. Their responses to temporary and permanent loads, as well as the impact of high-strength steel and two types of regular and innovative corrugation profiles reinforced by steel pipes, were analyzed. In addition, the slippage between the corrugated plates and circular pipes was evaluated, and the results are presented in this article. The corrugated steel plate utilization ratio was designed according to the Swedish [29], Canadian CHBDC [30], and American AASHTO [31] methods, and the results were compared. An analysis of the relationship between the plate utilization ratio and local buckling sensitivity was conducted according to the width-to-thickness ratio proposed by Sun et al. [27] and the Eurocode 3 Part 2 [32] standard.

2. Description of the Structure Dimensions

Three two-radius, low-profile arches were selected for this investigation, and their dimensions are presented in Figure 1. Such profiles are commonly used in practical engineering due to their rational performance and clearance box requirements as per national regulations. Each structure sat on 10 m long, 800 mm diameter reinforced concrete piles spaced 1.0 m apart. Solid pile heads that were 1.5 m wide and 2.25 m high were placed on top of the piles. Concrete of class C30/37 was used for the foundations to carry the vertical and horizontal loads transferred by the CSS structure. A soil cover of 1.40 m above the crown of the structure and live load model LM1 according to Eurocode 1 Part 2 [33] standard requirements were assumed in the analysis. Following standard installation rules for CSS structures, the backfill chosen comprised a sand–gravel mix with less than 5% fines, a unit weight of 21.50 kN/m³, and an effective friction angle of 33°. Detailed backfill envelope and soil parameters are presented in Section 3.

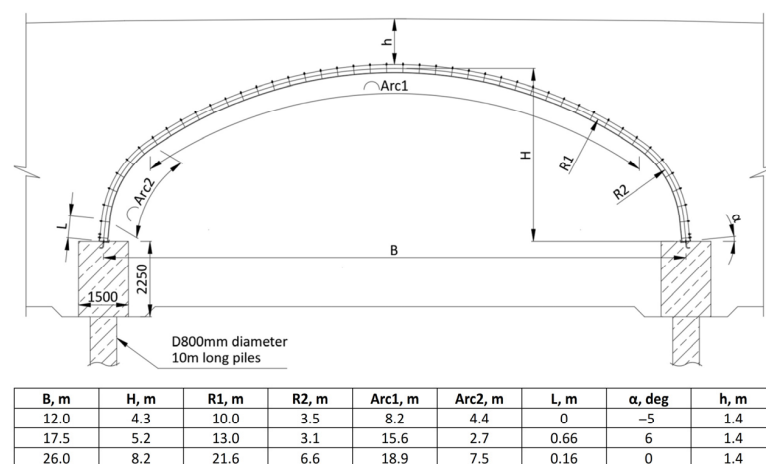


Figure 1. Transverse section dimensions of the two-radius, low-profile arches investigated in this article.

Two types of regular and innovative corrugation cross-section profiles were chosen to present comparative results, and their dimensions are presented in Figure 2. Although the length of the straight region of the corrugation mt somewhat depends on the plate thickness, only the axial tangent length behavior was studied. The diameter of the circular pipes for the innovative corrugations was selected considering the reduction in the buckling length of the straight region of the corrugation mt . It must be noted that, in this study, the corrugated plate and circular hollow-section steel pipe were assumed to have a fixed connection.

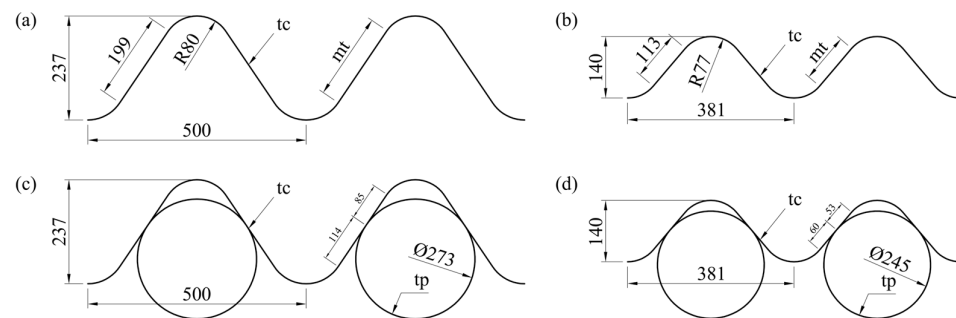


Figure 2. Cross-sections of corrugation profiles investigated in this study: (a) regular corrugation of 500×237 mm; (b) regular corrugation of 381×140 mm; (c) innovative corrugation of 500×237 mm reinforced by $\varnothing 273$ mm steel pipes; (d) innovative corrugation of 381×140 mm reinforced by $\varnothing 245$ mm steel pipes.

3. Numerical Models

A nonlinear plane strain two-dimensional finite element model (FEM), as presented in Figure 3, was developed using Plaxis 2D Geotechnical Engineering Software [34]. It was necessary to identify the internal forces acting on the CSS structure because of soil–steel interactions. Moreover, according to the CHBDC [30] and AASHTO [31] regulations, a rigorous finite element (FE) analysis must be conducted when designing the deep and deeper cross-sections of corrugation profiles. It was believed that a three-dimensional analysis [7,35–37] would not significantly contribute to the study’s objective of comparing the steel material strength utilization results of the SDM [29], CHBDC [30], and AASHTO [31] design philosophies. However, assumptions were needed for the corrugated plate’s equivalent stiffness and the live load distribution in the transverse direction of the road due to the 2D plane strain idealization. Nevertheless, it was decided that the accuracy of internal forces was sufficient [14]; additionally, 2D models require less computational power. A three-dimensional analysis is planned for future research to verify the results [25].

The model boundary conditions extended horizontally to five times the CSS span and vertically to 1.5 times the pile length below the foundations. The corrugated steel arch and concrete foundation were modeled as plate elements. Individual soil regions of the model were set according to the data presented in Figure 3 and summarized in Table 1. Soil regions were modeled with 15-node triangular finite elements. The engineering backfill envelope (the soil region around the CSS structure) was assumed according to the requirements presented in the AASHTO [31] guidelines. The interface elements were set in contact with the soil and plate elements, with a resistance factor of 0.8 for non-cohesive soils [38]. Because of the 2D plane strain idealization, equivalent stiffness and unit weight values were assigned to all plate elements. Table 1 summarizes the material properties used in the FE model. However, it should be noted that, regardless of the importance of the soil geotechnical properties, this study did not analyze the influence of different soil characteristics on the behavior of soil–structure interactions. This study assumed constant soil properties according to national standard requirements and focused only on the strength of the steel material utilized. Despite the sensitivity of soil characteristics

to buckling loads, it is believed that such a detailed investigation would go beyond the research objectives of this article.

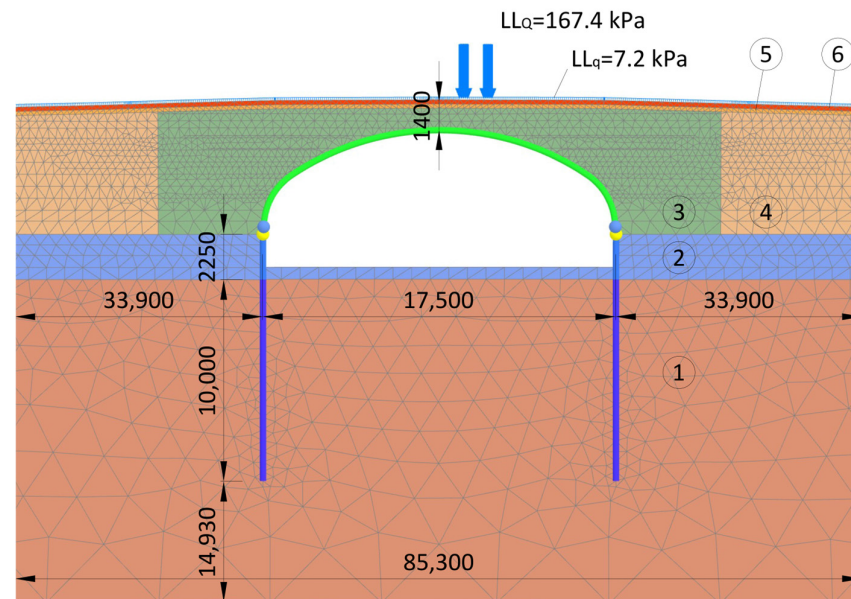


Figure 3. Numerical 2D finite element model of a CSS structure with a span of 17.5 m (soil layer numbering corresponds to Table 1).

Table 1. Summary of soil, concrete, and steel material properties assumed in the FE model.

No.	Identification	Material Model Type	Unit Weight kN/m ³	Elastic Modulus MPa	Poisson's Ratio –	Cohesion kPa	Friction Angle Degrees
1	Subsoil	Mohr–Coulomb	21.5	120	0.20	1	30
2	Foundation backfill	Mohr–Coulomb	21.2	80	0.23	1	29
3	Engineering backfill	Mohr–Coulomb	21.5	35	0.20	0	33
4	Embankment	Mohr–Coulomb	21.2	30	0.23	1	29
5	Road subbase	Mohr–Coulomb	23.0	400	0.23	1	43
6	Asphalt	Linear elastic	23.0	400	0.40	–	–
7	Foundation concrete	Elastic	21.6	30,000	0.20	–	–
8	Corrugated steel shell	Elastic	78.5	210,000	0.30	–	–

The staged backfilling installation method was analyzed. The thickness of each backfill layer was set to 60 cm because thinner layers had no significant effect on the results. However, the influence of unsymmetrical loading was investigated by placing non-symmetrical soil cover at the sides of the CSS structure (only one layer difference).

The tandem live load system LM1 (per Eurocode 1 Part 2 [33]) was positioned at various locations over the CSS structure. The aim of this approach was to simulate moving vehicles in the direction perpendicular to the axis of the structure and to induce the most unfavorable impact. The most critical position of the tandem system is when the interaction of the axial force and bending moment most reduces the structure's load-bearing capacity. In each subsequent phase, the live load (LL) was shifted 1.2 m from the left boundary of

the FE model toward the right. Figure 4 shows the LL located directly above the crown, while Figure 2 shows the tandem system's most critical position.

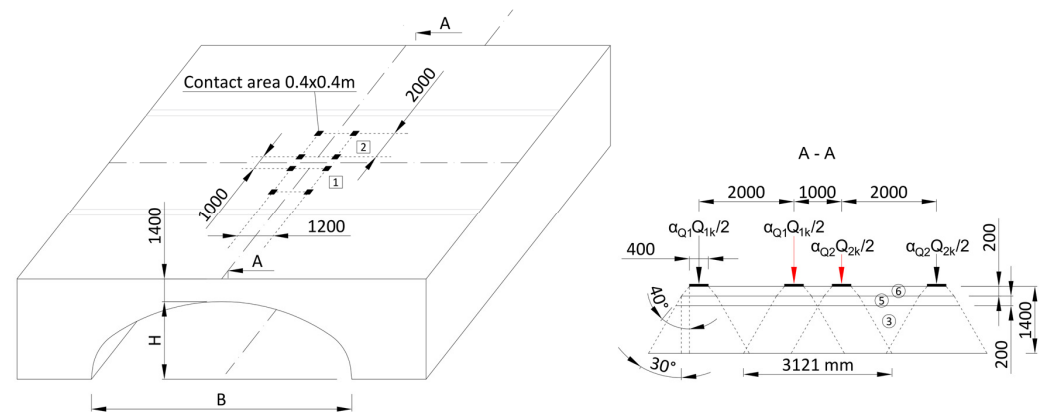


Figure 4. Layout of LM1 tandem system above the crown of the low-profile, two-radius arch structures (soil layer numbering corresponds to that in Table 1).

Because a 2D plane strain idealization model is used, only two wheels were represented in the FE model (see Figure 3). The transverse live load distribution across the road was determined as per AASHTO [31], as the LL footprint was limited to a unit slice in that direction. The reduced surface load (RSL) method was applied to the 2D model, reducing the surface pressure by a reduction factor [39]. The reduction factor can be calculated using Equation (1) and in relation to Figure 4 following the ad hoc method described in the AASHTO [31] guidelines:

$$LL_Q = r(H)P_0 = \frac{2W_0}{S + W_0 + 2\tan\theta \cdot H}P_0 = \frac{\alpha_{Q1} \cdot Q_{1k}/2 + \alpha_{Q2} \cdot Q_{2k}/2}{(S + W_0 + 2\tan\theta \cdot H) \cdot W_0} = \frac{0.8 \cdot 150 + 0.8 \cdot 100}{3.121 \cdot 0.4} = 160.2 \text{ kPa} \quad (1)$$

where LL_Q is the effect of LM1 from the interaction of two wheels on a single axle (a minimal distance of 1.0 m between the first and second line wheels generates the highest pressure on the structure, so the remaining LL axles can be neglected; see red arrows in Figure 4 section A-A); $r(H)$ is the surface pressure reduction factor according to the AASHTO [31] ad hoc method; P_0 is the actual service pressure on the footprint; W_0 is the width of the footprint, which is equal to 0.4 m; S is the distance between the wheels, which is 1.0 m; θ is the angle of the load distribution, which is assumed to be 40° for the asphalt layer and 30° for all other soil layers; H is the height of the soil cover, which is 1.40 m; α_{Q1} and α_{Q2} are the live load reduction factors, which are assumed to be 0.8; and Q_{1k} and Q_{2k} are the first and second lane axle loads, which are equal to 300 kN and 200 kN, respectively.

A total pressure of 167.4 kPa (including a uniformly distributed load of 7.2 kPa) under each wheel was simulated in the FE model (see Figure 2). Uniform pressure was also applied to the asphalt surface as per Eurocode 1 Part 2 [33]. Note that Plaxis 2D (Version 2023.1.0.136) Geotechnical Engineering Software [34] correctly modeled the increase in the footprint length along the road axis as the soil cover increased. The structural response to DL + LL was further examined in this study.

4. Results and Discussion

4.1. Corrugated Steel Plate Utilization

To investigate the results of the corrugated steel plate utilization ratio for regular and innovative corrugation profiles, the Swedish design method [29] was used, which is also adapted to Eurocode 3 Part 2 [32] standard requirements. Only the nature of buckling failure was investigated in this study; therefore, plate utilization ratios were calculated

according to Equation (2). It is important to note that the local buckling provision addressed in the SDM [29] was not applied in this stage of calculations.

$$\frac{N_{Ed}}{\frac{\chi_y N_{Rk}}{\gamma_{M1}}} + \frac{k_{yy} M_{Ed}}{\frac{M_{Rk}}{\gamma_{M1}}} \leq 1.0 \quad (2)$$

where N_{Ed} and M_{Ed} are, respectively, the axial force and bending moment calculated numerically using the FEM; χ_y is the reduction factor for flexural buckling, with $\chi_y = \frac{N_{cr}}{N_u}$; N_{cr} is the buckling load of a buried structure (see Equation (3)); N_u is the axial capacity of a fully plasticized cross-section, with $N_u = f_y A$; k_{yy} is the coefficient of the interaction; N_{Rk} and M_{Rk} are the resistance to the axial force and bending moment, with $N_{Rk} = f_y A$ and $M_{Rk} = f_y W$; f_y is the yield stress of the steel; A is the cross-section area; W is the plastic section modulus; and γ_{M1} is the partial coefficient for the steel material.

According to Klöppel and Glock [40] and Abdel-Sayed [41], the general buckling load for a buried structure can be calculated according to Equation (3):

$$N_{cr} = \frac{3\zeta}{\mu} \sqrt{\frac{E_{s,d} EI}{R_t}} \quad (3)$$

where ζ and μ are the calculation parameters, $E_{s,d}$ is the design soil modulus, EI is the steel structure stiffness, and R_t is the top radius of the soil–steel structure.

The cross-section parameters of the regular and innovative corrugation profiles were determined using SolidWorks (Version 34.1.1.0011) Corporation software [42]. The potential slippage between corrugated plates and circular pipes was evaluated by introducing reduction factors of the cross-section parameters accordingly: 0.5 for the moment of inertia and 0.75 for the plastic section modulus [43]. It is important to note that this overestimated modeling simplification was assumed to indicate the worst negative impact of slippage if such behavior existed. It was believed that the adopted reduction factors of the partially composite system may already be considered conservative; however, justification through parametric sensitivity analysis or experimental validation is planned for future research.

The results regarding the utilization ratio and cross-section area relationship are presented in Figures 5–8. The results of the slippage between the corrugated plates and circular pipes are presented on the right sides of the figures. The regular and innovative corrugation profiles and different steel strengths are also included. The first number in the rectangles represents the corrugated plate thickness, and the second number represents the pipe wall thickness (for example, 7 + 4 mm). The corrugated plate and pipe wall thickness combinations were chosen by taking into account the nearby cross-section area values of the regular and innovative corrugation profiles.

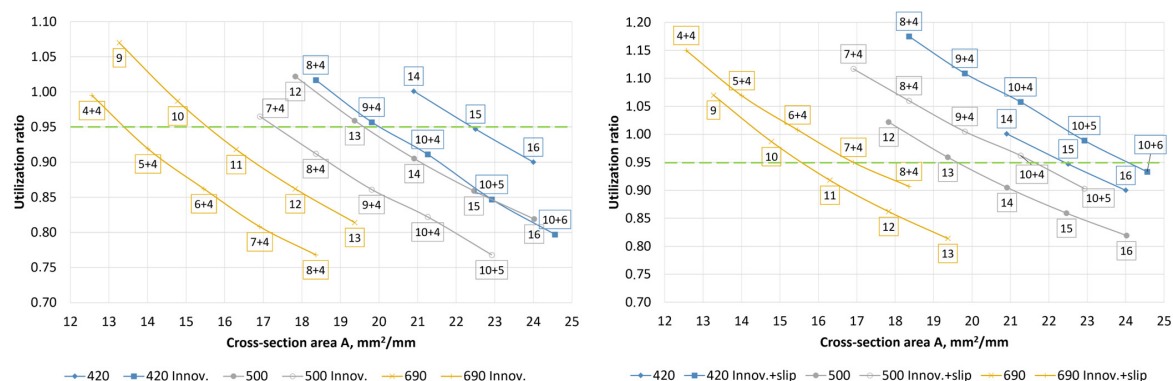


Figure 5. The relationship between the utilization ratio and cross-section area of a CSS structure with a 26 m span and 500 × 237 mm corrugation.

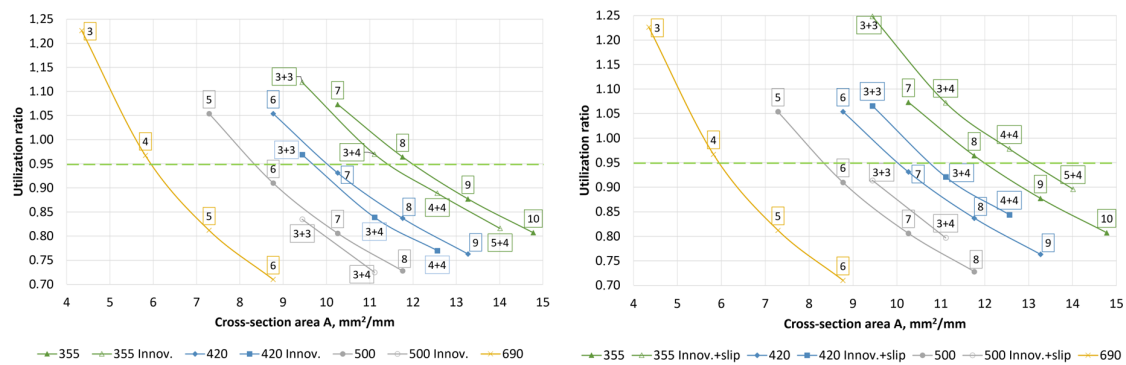


Figure 6. The relationship between the utilization ratio and cross-section area for a CSS structure with a 17.5 m span and 500 × 237 mm corrugation.

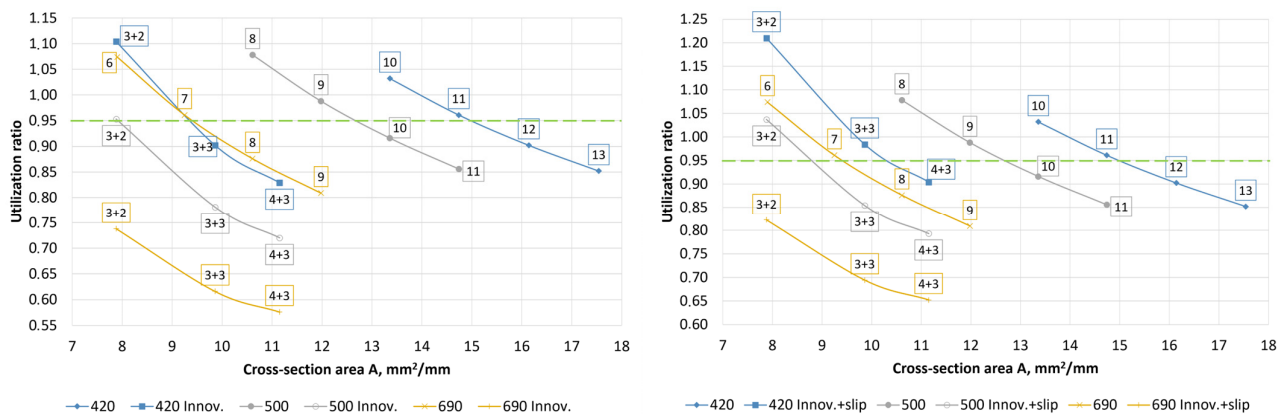


Figure 7. The relationship between utilization ratio and cross-section area of a CSS structure with a 17.5 m span and 381 × 140 mm corrugation.

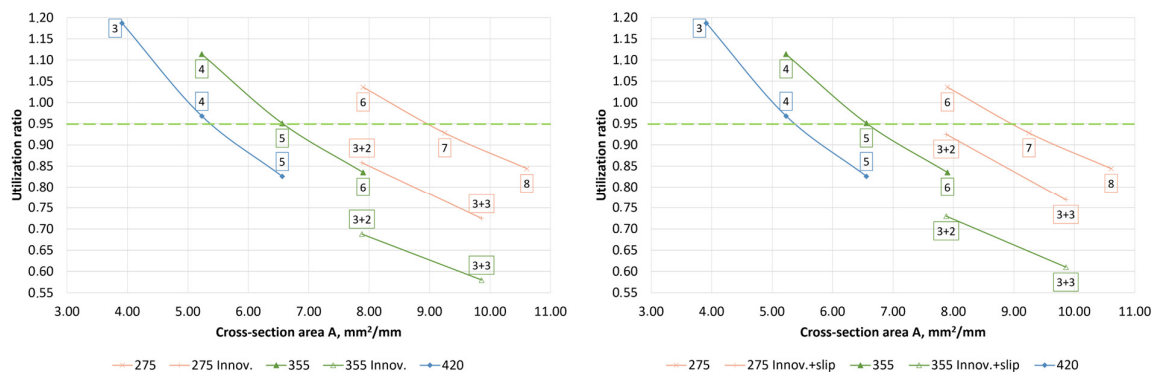


Figure 8. The relationship between the utilization ratio and cross-section area of a CSS structure with a 12 m span and 381 × 140 mm corrugation.

The results in Figures 5–8 indicate that the steel strength has a direct impact on the plate thickness utilization ratio—a higher yield stress allows for the use of thinner steel while maintaining the same strength, which can reduce the overall material usage and weight of the structure. The use of higher-strength steels can help to optimize resource efficiency. For example, Figure 5 shows that switching from 420 MPa steel to 690 MPa high-strength steel can reduce the cross-sectional area of both the regular and innovative corrugations by 31% at a utilization ratio of 0.95. Moreover, the results in Figure 5 show that the innovative corrugated profile strengthened with steel pipes also has a positive effect when potential slippage is not considered. As shown in Figure 5, the cross-section area of the innovative profile is about 12.9% lower than that of the regular profile with

690 MPa steel and a 0.95 utilization ratio. The same relationship can be observed for lower steel grades. However, if the cross-section parameters are reduced to account for the impact of potential slippage, the innovative profile loses its advantage. In such a case, the cross-section area of the innovative profile is about 6.7–10.7% larger at a utilization ratio of 0.95.

A similar relationship is presented in Figure 6 for a CSS structure with a smaller span of 17.5 m but the same corrugation profile of 500×237 mm. For example, by selecting 500 MPa steel instead of 355 MPa steel, the cross-section area can potentially be reduced by 30%. The application of the innovative profile is less beneficial for lower-span CSS structures because of the already high stiffness of the regular deeper corrugation profile. Nevertheless, as shown in Figure 6, the cross-section area of the innovative profile is about 5% smaller than that of the regular profile with 355 MPa steel. However, the cross-section area of the innovative profile is about 7–8.3% larger because of the effect of slippage.

The relationship between the utilization ratio and cross-section area of a steel structure with a 17.5 m span and deep corrugation of 381×140 mm is shown in Figure 7. In this case, when comparing conventional steel of 420 MPa and an HSS of 690 MPa, it was found that the use of the higher-strength steel can reduce the regular corrugation profile cross-section area by 37%. Meanwhile, the innovative profile has much greater potential than the larger-span and deeper corrugation structures shown in Figures 5 and 6. The cross-section area of the innovative 381×140 mm profile is about 36% lower than that of the regular profile with 420 MPa steel. However, the benefits of using higher steel grades for innovative profiles are lost because the high-strength steel plate of the regular profile is already thin enough and already ensures the load-bearing capacity of the CSS structure. Additionally, slippage has a minor impact on the relationship, the results of which are presented on the right side of Figure 7.

According to the results presented in Figure 8, the relationship between the utilization ratio and cross-section area of the CSS structure with a 12 m span and corrugation dimensions of 381×140 mm is still affected when using higher grades of steel. The cross-section area decreases by 40% when using 420 MPa steel instead of 272 MPa steel. However, it must be noted that the use of HSS in a structure with such relatively small dimensions could lead to a much higher risk of local buckling in the straight region of the corrugation because of the use of thin corrugated steel plates. In addition, the impact of the innovative profile on reducing the cross-section area is minor, regardless of whether potential slippage between the corrugated plates and circular pipes is considered.

4.2. Local Buckling Impact

According to the Swedish design method [29] of corrugated soil–steel composite bridges, commonly used systems employ relatively thick corrugated steel plates, where local buckling does not represent a problem. However, it still requires the corrugation used to be examined with regard to local buckling by reducing the plastic moment capacity. The reduction coefficient can be calculated using Equation (4), and this calculation must be performed regardless of the cross-section parameters of the corrugation profile.

$$M_{ucr} = \left(1.429 - 0.156 \cdot \ln \left((m_t/t) \cdot (f_{yk}/227)^{1/2} \right) \right) \cdot M_u \quad (4)$$

where m_t is the corrugated profile's tangent length, t is the corrugated plate thickness, and M_u is the bending moment capacity.

It must be noted that the CHBDC [30] and AASHTO [31] methods do not include any rules regarding local buckling prevention during the design of corrugated soil–steel composite systems, thus giving rise to the following question: is the SDM [29] approach

regarding the evaluation of local buckling too conservative? Considering this, an analysis was conducted to examine how the results are impacted by the local buckling relationship presented in the SDM [29]. In addition, the SDM [29] results were compared with flexural and axial strength designs, as well as global buckling failure checks, according to the methods presented in the AASHTO [31] and CHBDC [30] regulations. When following AASHTO [31] or CHBDC [30], the flexural and axial strength designs should be carried out according to Equation (5):

$$\left(\frac{T_f}{R_t}\right)^2 + \left|\frac{M_u}{M_n}\right| \leq 1.0 \quad (5)$$

where T_f and M_u are, respectively, the factored thrust and moment calculated numerically using the FEM; R_t is the factored thrust resistance, with $R_t = \phi_h F_y A$; M_n is the factored moment resistance, with $M_n = \phi_h M_p$; ϕ_h is the resistance factor; and M_p is the plastic moment capacity of the section.

The global buckling failure check according to AASHTO [31] should be carried out as presented in Equation (6):

$$\frac{T_f}{R_b} \leq 1.0 \quad (6)$$

where T_f is the factored thrust calculated numerically using the FEM, and R_b is the nominal axial force in the wall of the structure that causes general buckling, as shown in Equation (7):

$$R_b = 1.2 \phi_b C_n (E_p I_p)^{1/3} (\phi_s M_s K_b)^{2/3} R_h \quad (7)$$

where ϕ_b is the general buckling resistance factor; C_n is the scalar calibration factor accounting for some nonlinear effects, equal to 0.55; E_p is the steel material elasticity modulus; I_p is the corrugated steel plate moment of inertia per unit length; ϕ_s is the soil resistance factor; M_s is the constrained modulus of embedment, with $K_b = (1 - 2\nu) / (1 - \nu^2)$; ν is the soil Poisson's ratio; R_h is the correction factor for backfill geometry, with $R_h = 11.4 / (11 + S/H)$; S is the span of the structure; and H is the height of the soil cover.

The results of the utilization ratio and cross-section area relationship obtained using the SDM [29], AASHTO method [31], and CHBDC [30] method are presented in Figures 9–12, allowing for their comparison. Findings regarding the innovative corrugation profile are not presented, as investigations were only conducted for the regular corrugation profile and same plate thicknesses, which were closest to the utilization ratio of 1.0 according to the SDM [29] approach.

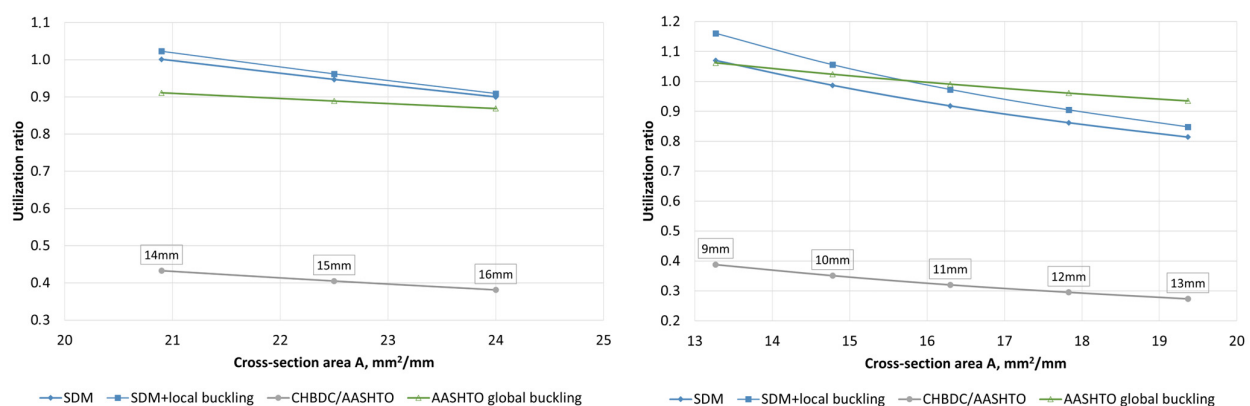


Figure 9. The relationship between the utilization ratio and cross-section area of a CSS structure with a 26 m span and 500 × 237 mm corrugation profile when using 420 MPa and 690 MPa steel yield strength.

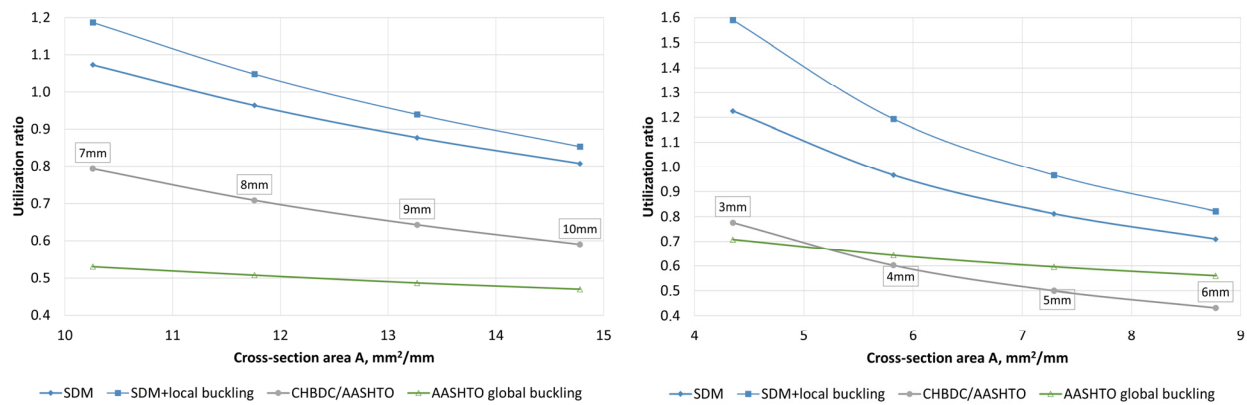


Figure 10. The relationship between the utilization ratio and cross-section area for a CSS structure with a 17.5 m span and 500×237 mm corrugation profile with respect to 355 MPa and 690 MPa steel yield strength.

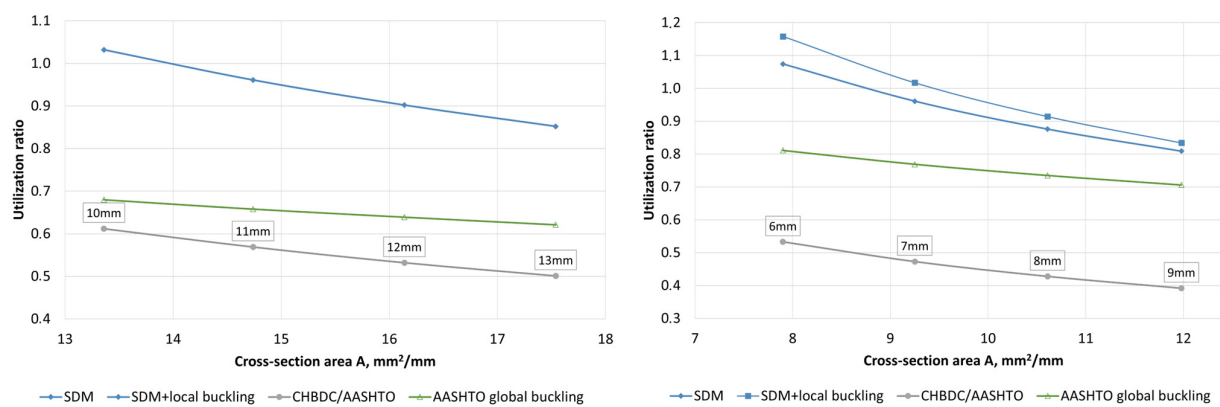


Figure 11. The relationship between the utilization ratio and cross-section area of a CSS structure with a 17.5 m span and 381×140 mm corrugation profile with respect to 420 MPa and 690 MPa steel yield strength.

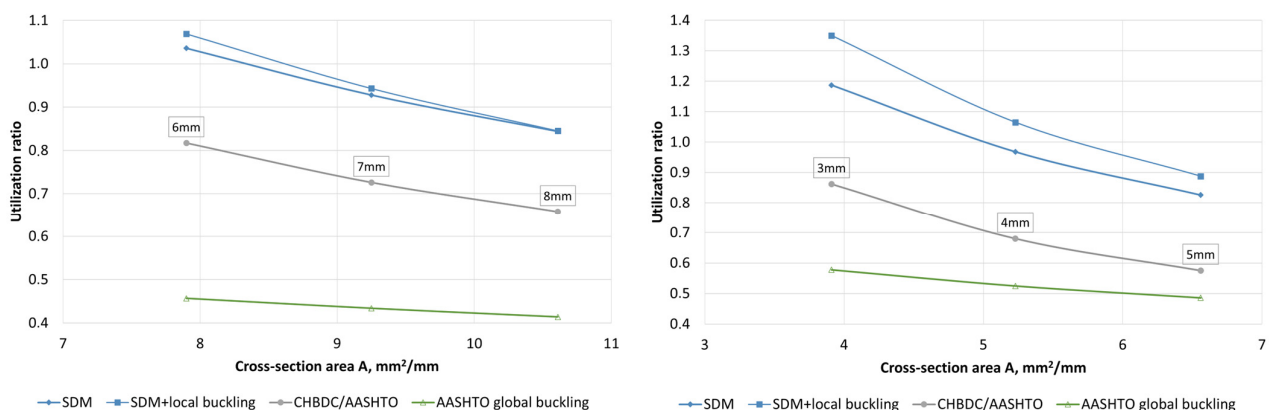


Figure 12. The relationship between the utilization ratio and cross-section area of a CSS structure with a 12 m span and 381×140 mm corrugation profile with respect to 275 MPa and 420 MPa steel yield strength.

The relationship results for a CSS structure with a 26 m span and 500×237 mm corrugation profile when using steel with a strength of 420 MPa and 690 MPa are presented in Figure 9. It can be observed that the flexural and axial strength design according to AASHTO [31] or CHBDC [30] is less conservative than that according to the SDM [29] or AASHTO [31] global buckling approach. In this case, the utilization ratio is reduced by 67% for the plate with a 11 mm thickness. A reasonably higher impact of the local buckling

reduction coefficient as per the SDM [29] is achieved when an HSS of 690 MPa is used. The utilization ratio for the 11 mm plate is 6% higher. Nevertheless, the results obtained in this case are in line with those obtained using the AASHTO [31] global buckling approach.

The results obtained for a CSS structure with a 17.5 m span and 500×237 mm corrugation profile are shown in Figure 10. The SDM [29] approach appears to be more demanding, and the need for a thicker corrugated plate is greater when the plastic moment capacity is reduced. In this case, the CHBDC [30] and AASHTO [31] results are less conservative and more uniform, especially for higher steel grades. The utilization ratio obtained when using the SDM [29] and considering local buckling is 18.5% higher than that obtained when the reduction factor because local buckling is not applied for 5 mm 690 MPa steel plates. However, the utilization ratio is 60% higher than that obtained using the AASHTO [31] global buckling failure check. Furthermore, it can be observed that the global buckling check is more sensitive to thinner plates and higher steel grades than the flexural and axial strength design.

Figure 11 shows the results of a CSS structure with a 17.5 m span and 381×140 mm corrugation profile with 420 MPa and 690 MPa steel strength, respectively. The flexural and axial strength designs according to CHBDC [30] or AASHTO [31] still clearly demonstrate much less conservative results. In this case, the corrugated plate utilization ratio is 46% higher than that observed in the SDM [29] and AASHTO [31] global buckling results for 420 MPa and 11 mm steel. However, because a relatively thick corrugated plate is needed, local buckling under the SDM [29] does not influence the results obtained with the 420 MPa steel. Nevertheless, the use of thinner plates and stronger steel increases the already high estimation obtained using the SDM [29], making the method even less preferable. However, the flexural and axial strength design obtained according to CHBDC [30] or AASHTO [31] is much less conservative than that obtained according to the other methods, especially when higher steel grades are used.

A comparison of the relationships between the utilization ratio and cross-section area of a CSS structure with a 12 m span and 381×140 mm corrugation profile with respect to 275 MPa and 420 MPa steel yield strength is shown in Figure 12. The design according to the SDM [29] remains the most conservative, while the AASHTO [31] global buckling failure check is less decisive for CSS structures with a smaller span. For example, 7 mm and a S275 steel plate is sufficient to ensure load-bearing capacity of the structure under a 94% plate utilization ratio according to the SDM [29], under 73% according to the CHBDC [30] flexural and axial strength design, and only under 43% according to the AASHTO [31] global buckling failure check.

Considering the presented results, it can be concluded that the SDM [29] is the most conservative method for the design of corrugated soil–steel structures presented in this study. Local buckling evaluation by reducing the plastic moment capacity seems to be even more conservative, making the method less preferable. However, it should be noted that the SDM [29] was designed for CSS structures with a relatively small span, and the impact of HSS was not considered in its most recent update, which was carried out in 2014. Considering this, additional local buckling sensitivity analyses were carried out according to the width-to-thickness ratio limit proposed by Sun et al. [27] and Eurocode 3 Part 2 [32] for sections in class 3 under axial compression.

Sun et al. [27] proposed a width-to-thickness ratio β for CSS structures to prevent local buckling. According to their investigation, $\beta = s/2t \leq 63\epsilon_k$, where s is the expansion length of a single repeating corrugation section, t is the corrugated steel plate thickness, and $\epsilon_k = \sqrt{235/f_y}$. In contrast, Eurocode 3 Part 2 [32] for sections in class 3 proposes $c/t \leq 42\epsilon_k$, where c is the web length of the sections applicable for regular steel girder structures. Following these ratios, the results presented in this article were compared

considering the impact of the minimal plate thickness and HSS on local buckling. A comparison of the results is presented in Table 2.

Table 2. Results of local buckling sensitivity analysis in relation to utilization ratio.

Structure Description	t, mm	SDM	SDM + Local Buckling	s/2t	c/t
B = 26 m, 500 × 237 mm (s = 722.4 mm, c = 199 mm), $f_y = 690$ MPa	10	0.987	1.056	36.1 < $63\epsilon_k = 36.7$	19.9 < $42\epsilon_k = 24.5$
B = 17.5 m, 500 × 237 mm (s = 722.4 mm, c = 199 mm), $f_y = 690$ MPa	5	0.812	0.967	72.2 > $63\epsilon_k = 36.7$	39.8 > $42\epsilon_k = 24.5$
B = 17.5 m, 381 × 140 mm (s = 492.2 mm, c = 113 mm), $f_y = 690$ MPa	7	0.961	1.017	35.1 < $63\epsilon_k = 36.7$	16.1 < $42\epsilon_k = 24.5$
B = 12 m, 381 × 140 mm (s = 492.2 mm, c = 113 mm), $f_y = 420$ MPa	5	0.826	0.888	49.2 > $63\epsilon_k = 47.1$	22.6 < $42\epsilon_k = 31.4$

Despite the conservative results of the SDM method [29], in some cases, local buckling prevention assessment according to Sun et al. [27] or Eurocode 3 Part 2 [32] requires even thicker plates, as shown in Table 2 (the results marked in red color). However, the results of this study allow for the conclusion that the SDM [29] of preventing local buckling by reducing the plastic moment capacity should be revised, especially in cases using thicker and lower steel grades, when local buckling in principle can be neglected. The opposite situation is found for the CHBDC [30] and AASHTO [31] methods, which, in most cases, showed much less conservative results. This outcome indicates that local buckling prevention is important and cannot be ignored during the design of CSS structures. Moreover, the results indicate the need for more detailed analyses considering not only the strength utilized (safety limit state) but also serviceability aspects.

5. Summary and Conclusions

This study performed 2D numerical analyses of three low-profile, two-radius arches with spans of 26 m, 17.5 m, and 12 m embedded in soil to identify internal forces from permanent and temporary loads. An analysis of the steel plate utilization ratio was conducted according to the Swedish design method [29] with the aim of identifying the impacts and benefits of high-strength steel and two types of innovative deep and deeper corrugated cross-sections strengthened with circular steel pipes. Results considering slippage between the corrugated plates and circular pipes were also obtained. Afterward, an analysis of the steel plate utilization ratio was conducted according to the AASHTO [31] and CHBDC [30] design methods, and the results were compared to those obtained using the SDM [29] and its requirements for preventing local buckling by reducing the plastic moment capacity. Finally, the corrugated plate utilization results were compared with the width-to-thickness ratio proposed by Sun et al. [27] and with the c/t ratio presented in Eurocode 3 Part 2 [32]. According to the numerical analysis, the following conclusions can be drawn:

- Higher-strength steel allows for thinner plates to be used in CSS structures while maintaining the same strength. HSS can help to optimize and reduce the overall material usage and weight of the structure. For example, as shown in Figures 5–8, the transition from conventional steel to higher-strength steel enables the regular and innovative corrugation cross-section areas to be reduced by 30–40% with a utilization ratio of 95%.
- The cross-section area of the innovative profile can be reduced by 5% to 36% in comparison to that of the regular corrugation profile. The innovative profile with

500 × 237 mm corrugation loses its advantage when the proposed slippage effect is considered, while the 381 × 140 mm corrugation profile reinforced by circular pipes achieves the same load-bearing capacity in less heavy CSS structures. However, the innovative cross-section has a minor impact on CSS structures with shorter spans.

- Flexural and axial strength design according to the CHBDC [30] or AASHTO [31] regulations seems to be less conservative than that of other methods, especially when higher steel grades are used. However, it should be emphasized that differences arise from distinct design philosophies rather than methodological shortcomings. According to the results presented in Figures 9–12, the utilization ratio can be up to 67% lower in contrast to that of the SDM [29]. However, the use of thinner plates and stronger steel increases the already high estimations obtained using the SDM [29], making the method less preferable. The AASHTO [31] global buckling failure check is less decisive for CSS structures with shorter spans.
- In accordance with the results shown in Table 2, in specific cases, the corrugated plate is too thin even when considering the conservative SDM [29] and its local buckling approach. It is worth noting that conservatism is observed within a comparative code framework only. The results are even more surprising knowing that CHBDC [30] and AASHTO [31] propose less conservative design approaches, the results of which are presented in Figures 9–12. Nevertheless, the results presented in Table 2 indicate that more detailed analyses are required and that local buckling should be prevented during the design of CSS structures. Such analyses of local buckling in the straight region of the corrugation, including laboratory tests, are already planned for future research.

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