

Article

Numerical and Experimental Analysis of Innovative Cable-Stayed-String Steel Bridge with Intersecting Cable Stays

Povilas Dabrila * and Algirdas Juozapaitis

Civil Engineering, Vilnius Gediminas Technical University, 10223 Vilnius, Lithuania;
algirdas.juozapaitis@vilniustech.lt

* Correspondence: povilas.dabrila@vilniustech.lt

Abstract

Cable-stayed bridges are sensitive to deformation under asymmetrical and localized loads, usually requiring heavy stiffening girders and massive pylons to ensure sufficient stiffness and deformation control. This paper proposes an innovative cable-stayed steel bridge system that incorporates intersecting stay cables, additional intermediate pylons, and a prestressed stringer, rather than a conventional stiffening girder. A calculation approach based on interactions among the string, stay cables, and pylons is developed to evaluate the structural behavior of the proposed system. Experimental studies using a scaled physical model were conducted to validate the numerical and analytical results. This research provides a basis for the practical application and further development of the novel cable-stayed-string bridge system.

Keywords: steel bridge; cable-stayed-string structure; intermediate pylons; intersecting cable stays; nonlinear behavior

1. Introduction

In recent years, pedestrian bridges have become an essential component of urban infrastructure, providing safe pedestrian movement, particularly in densely populated areas [1,2]. Depending on structural configuration and functional requirements, various types of pedestrian bridges have been constructed worldwide, including beam, truss, arch, suspension, and cable-stayed bridges [3,4]. Among them, the cable-stayed pedestrian bridge (CSPB) is particularly popular in modern cities due to its aesthetic appearance, cost-effectiveness, and relatively simple erection and construction process [1,5]. The main structural components of a CSPB are the deck or stiffening girder, pylons, stay cables, and foundations. Stay cables, acting as elastically flexible supports, are essential for distributing permanent and service loads acting on the bridge's stiffening girder. They also play an important role in controlling the vertical and horizontal displacements of the stiffening girder and pylons, respectively [6–9].

However, advances in design methods and the use of high-strength materials have led to increasingly slender pedestrian bridge structures [9,10]. Consequently, these bridges are more deformable and more susceptible to dynamic effects and pedestrian-induced vibrations, which may negatively affect user comfort and serviceability [11,12]. Therefore, human-induced vibrations, dynamic response, and structural control of cable-stayed pedestrian bridges have become important topics in recent research [8,13,14]. In

Academic Editor: Sang-Hyo Kim

Received: 10 June 2026

Revised: 9 July 2026

Accepted: 13 July 2026

Published: 14 July 2026

Copyright: © 2026 by the authors. Licensee MDPI, Basel, Switzerland. This article is an open access article distributed under the terms and conditions of the [Creative Commons Attribution \(CC BY\) license](https://creativecommons.org/licenses/by/4.0/).

addition, numerical modeling and model updating are increasingly used for both analysis and validation of the static and dynamic behavior of cable-stayed bridges [15,16].

Innovative solutions have been proposed to address these challenges in steel pedestrian bridges, including replacing conventional stiffening girders with prestressed string elements. Recent theoretical and experimental studies have shown that the use of such string systems instead of conventional strutted structures can significantly reduce bridge mass and increase structural efficiency [17,18]. In these systems, the string acts as a component that stabilizes kinematic displacements, thus enabling the effective use of a lightweight deck.

In order to reduce the deformability of cable-stayed pedestrian bridges, intermediate pylons and intersecting stay cables connecting them have been introduced [19]. The use of intersecting stay-cable configurations provides considerable structural advantages for steel cable-stayed bridges, especially under asymmetric loading [20–22]. The use of intersecting stays can reduce the required stiffness and mass of the stiffening girder and, consequently, the bridge's total mass. At present, eight pedestrian bridges with such intersecting stay-cable systems have been constructed. However, it should be noted that the calculation and analysis methods for such bridges with intersecting stay cables are still insufficiently developed.

To further increase the efficiency of these bridges with intersecting stays, an innovative pedestrian bridge solution has been proposed in which the stiffening girder is replaced by a prestressed flexible steel string [23]. It should be emphasized that this new structural solution has not yet been implemented in practice.

Despite the established design experience accumulated in conventional cable-stayed bridge structural systems, the calculation methods used remain limited. Cable-stayed bridges, as highly statically indeterminate systems, should be designed using rational structural parameters. However, their application is difficult without appropriate calculation methods and experimental validation [11,24,25]. In particular, determining the geometry and prestressing forces in cable-supported systems requires consideration of the nonlinear behavior of stay cables, cable sag effects, and construction sequence. It is also important to evaluate the complex interaction between stay cables and horizontal load-bearing elements [26–30]. Recent studies on the optimization and adjustment of stay-cable prestressing forces further underscore the need to improve calculation methods for cable-stayed bridges to accurately predict internal forces, displacements, and the final structural configuration [31–35]. Therefore, the design of an innovative bridge system with intersecting stays, intermediate pylons, and a prestressed string requires additional analytical, numerical, and experimental studies to justify the practical application of prestressed steel cable-stayed bridges with intersecting stay supports [36,37].

This paper discusses an innovative structural system of a steel pedestrian bridge with intersecting stay cables and prestressed strings. The main focus is placed on the nonlinear behavior analysis of this bridge system and the development of its calculation method. A calculation method for such a cable-stayed system is developed based on the interaction between the string, intersecting stay cables, and intermediate pylons. Numerical and experimental methods are also used to analyze the behavior of the bridge under investigation. Experimental tests of the bridge model were conducted to validate the developed analytical solutions and numerical results. This study provides a basis for the practical application and further development of the new cable-stayed-string bridge system.

2. Calculation of the Innovative Cable-Stayed-String Bridge

The analyzed bridge system uses an intersecting cable-stay arrangement, where the stays cross each other between the pylons and the string. In this system, the load is transferred not through a conventional stiffening girder, but through the interaction of the stay

cables, intermediate pylons, and the pre-stressed string. This allows the bridge to operate primarily under axial forces and reduces the need for a massive bending-stiff deck element.

A conventional cable-stayed structural system has a typical limitation associated with the use of a massive girder. This girder must resist bending moments and considerable axial compressive forces, which may lead to stability-related issues. One possible alternative is to replace rigid bending elements with flexible suspended elements, such as cables or strings. These elements are usually prestressed and anchored to the foundation; therefore, when external loads are applied, the structural response is governed mainly by tensile stresses. In the investigated bridge system, the load-bearing element of the bridge deck may therefore be represented by a string. Its behavior can first be described by analyzing a single string internode.

The analyzed innovative bridge system consists of intersecting cable stays, four intermediate pylons, and a string, as shown in Figure 1. The anchorage stays of the bridge are assumed to be fixed at the end supports. A planar structural model is considered in order to determine the variables governed by the interaction between the cable system and the string.

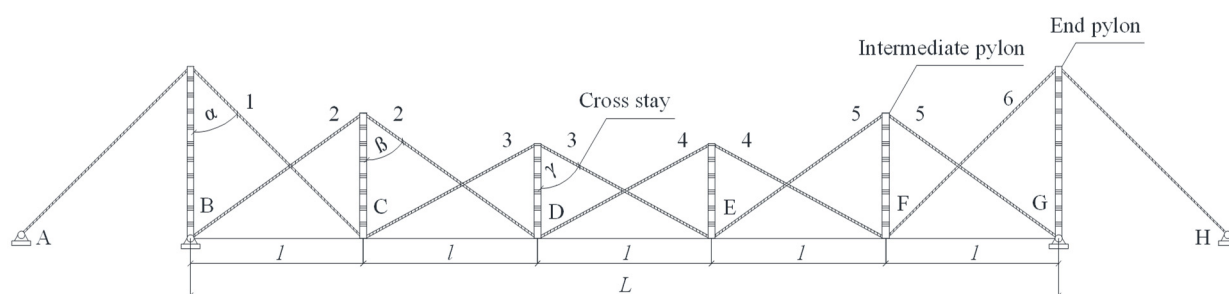


Figure 1. Scheme of the innovative bridge with intersecting cable stays and intermediate pylons.

Figure 2 shows the deformed configuration of the cable-stayed-string bridge under symmetrical loading. The applied load is assumed to be uniformly distributed along the string. In this case, the main unknowns are the displacements at the connection points between the intersecting stays and the string, Δ_c , Δ_d , Δ_e , and Δ_f , and the axial forces in the stay cables, N_i . The supporting forces transferred from the stay cables to the string, F_1 , F_2 , F_3 , and F_4 , are also considered as additional unknowns.

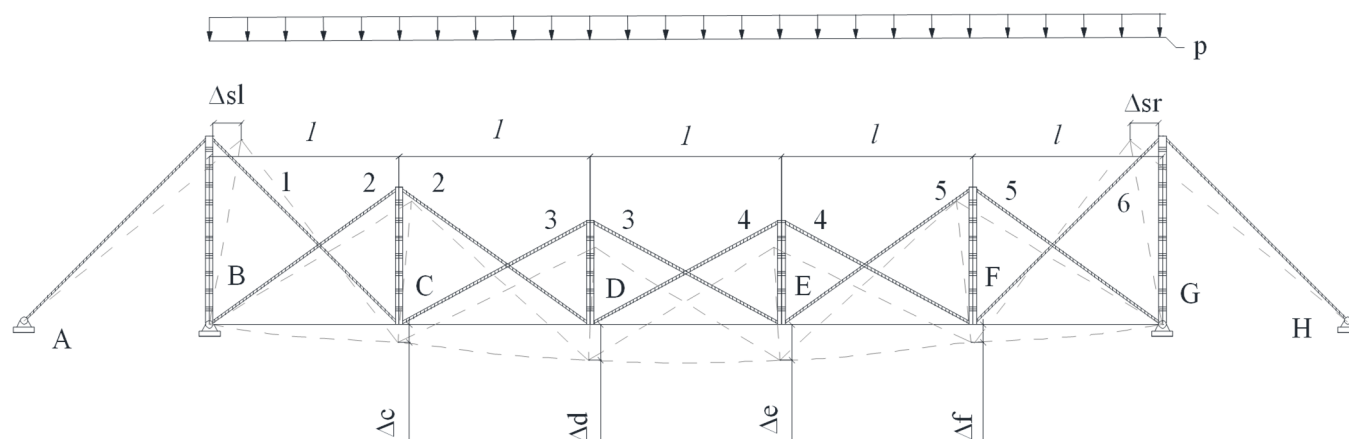


Figure 2. Deformed scheme of the bridge.

2.1. Cross-Stay Calculation

The first step is to determine the axial forces in the stay cables. For this purpose, the stay-cable system is divided into triangular subsystems formed by the intermediate pylons and the cables connected at their upper nodes. In each subsystem, cable elongation is governed by the displacements of the cable ends and by the vertical displacement of the corresponding intermediate pylon.

This relationship makes it possible to analyze the behavior of the connected members separately. Stay cables connected to the same pylon at the same angle are assumed to have equal tensile forces under equilibrium conditions. This assumption simplifies the calculation by reducing the number of unknowns and by relating the system displacements to the internal forces in the stay cables. The corresponding expressions are given in Equations (1)–(6), from [36], where the initial prestressing forces are denoted by N_{pi} , as shown in Figure 3.

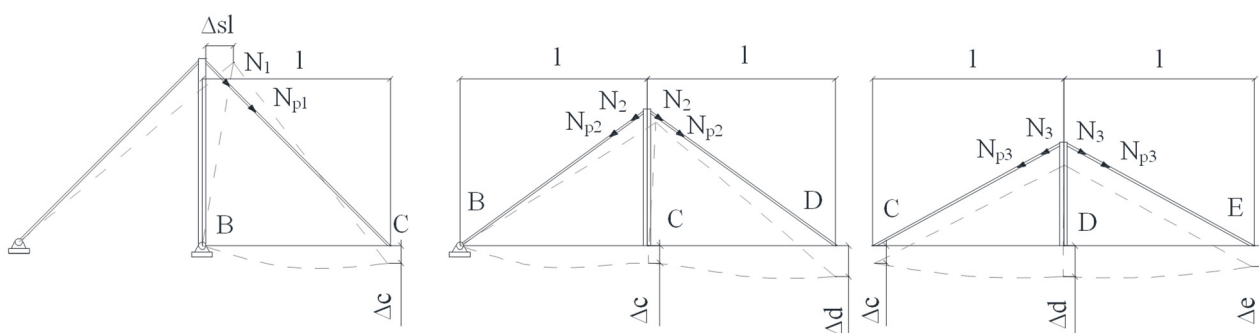


Figure 3. Cable stays and pylons, divided into triangular systems (left side of the system).

$$N_1 = \frac{E_{v1}A_v(\cos\alpha \cdot (\Delta_c) - \sin\alpha \cdot (\Delta_{sl}))}{l_1} + N_{p1} \quad (1)$$

$$N_2 = \frac{E_{v2}A_v\cos\beta \cdot (\Delta_d - 2\Delta_c)}{2l_2} + N_{p2} \quad (2)$$

$$N_3 = \frac{E_{v3}A_v\cos\gamma \cdot (\Delta_c - 2\Delta_d + \Delta_e)}{2l_3} + N_{p3} \quad (3)$$

$$N_4 = \frac{E_{v4}A_v\cos\gamma \cdot (\Delta_f - 2\Delta_e + \Delta_d)}{2l_4} + N_{p4} \quad (4)$$

$$N_5 = \frac{E_{v5}A_v\cos\beta \cdot (\Delta_e - 2\Delta_f)}{2l_5} + N_{p5} \quad (5)$$

$$N_6 = \frac{E_{v6}A_v(\cos\alpha \cdot (\Delta_f) - \sin\alpha \cdot (\Delta_{sr}))}{l_6} + N_{p6} \quad (6)$$

E_{vi} —cable stay elastic modulus, A_v —cable stay cross-sectional area, N_{pi} —prestressing force in stays, Δ_{sl} —horizontal left pylon top deflection (symmetrically Δ_{sr}).

The vertical components of the cable forces N_i are used to determine the resulting support forces F_i that the stay cables transfer to the string. It should be noted that pylon compression is not considered in this formulation. Therefore, the forces F_i can be expressed only in terms of the cable tensile forces and their corresponding angles in the initial structural configuration, as shown in Figure 4.

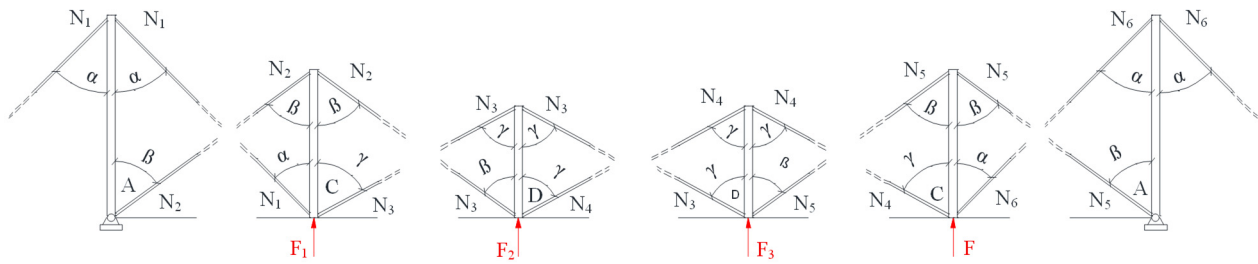


Figure 4. Cable stay forces generating uplifting force F .

To determine the uplift forces F_i generated by the stay cables and transferred to the string, the force state at the cable attachment points, shown in Figure 2, is considered [36]:

$$F_1 = N_1 \cos \alpha + N_3 \cos \gamma - 2N_2 \cos \beta \quad (7)$$

$$F_2 = N_2 \cos \beta + N_4 \cos \gamma - 2N_3 \cos \gamma \quad (8)$$

$$F_3 = N_5 \cos \beta + N_3 \cos \gamma - 2N_4 \cos \gamma \quad (9)$$

$$F_4 = N_6 \cos \alpha + N_4 \cos \gamma - 2N_5 \cos \beta \quad (10)$$

The bridge's stay cables can be modeled as inclined cables with limited sag. Since the stay-cable response is nonlinear, the effect of cable sag should be accounted for in the analysis. In the initial stage of the calculation, the cable is assumed to be unstressed. The main unknowns are the displacements of the stay-cable joints C, D, E, and F, the change in cable length Δl , and the axial force $S(x)$. The geometric elongation of the cable, Δs_g , is also considered as an additional unknown, as shown in Figure 5.

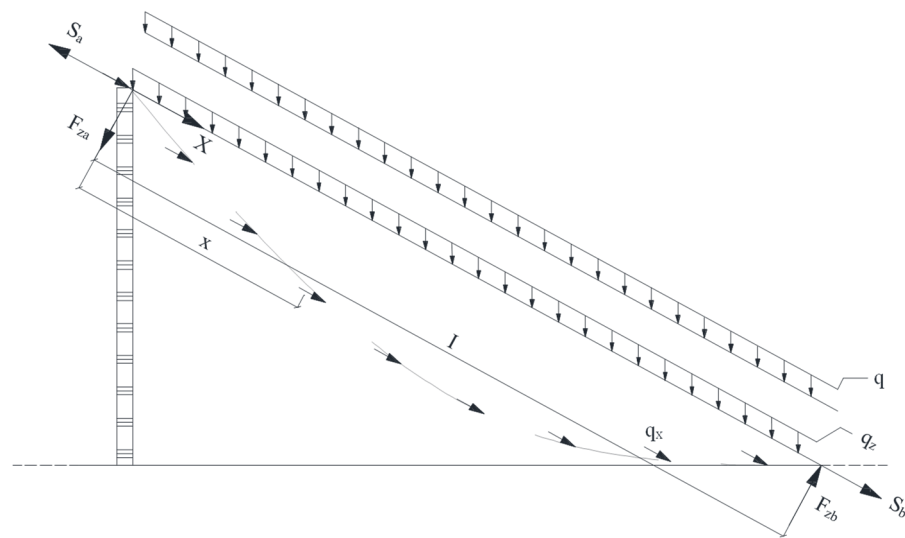


Figure 5. Cable-stay calculation scheme.

First, the supportive reactions at the cable's ends are determined [36]:

$$F_{za} \cong \frac{q_z \cdot l}{2} + \frac{2}{3} q_x f \quad (11)$$

$$F_{zb} \cong \frac{q_z \cdot l}{2} - \frac{2}{3} q_x f \quad (12)$$

q_z and q_x —cables weight load in z and x -axis, f —cable sag.

The axial (tension) force on the cable can be determined throughout its length; thus,

$$S(x) \cong \frac{q_z \cdot l^2}{8f} + \frac{q_x \cdot l}{2} + \left(1 - \frac{2x}{l}\right) \quad (13)$$

The cable's curve shape $z(x)$ is determined by the ratio of the bending moments caused by the applied loads and the axial tension force in the cable:

$$z(x) \cong \frac{M_z(x) + M_x(x)}{S(x)} \quad (14)$$

$$M_z(x) \cong \frac{q_z \cdot l}{8} \left(\frac{4x}{l} - \frac{4x^2}{l^2} \right) \quad (15)$$

$$M_x(x) \cong \frac{2}{3} q_x l f \left(\frac{2x^3}{l^3} - \frac{3x^2}{l^2} + \frac{x}{l} \right) \quad (16)$$

The cable length difference before deformation can be described as follows:

$$S \cong l + \frac{8f^2}{3l} \quad (17)$$

With this expression, the cable's geometric elongation after deformation can be obtained.

$$\Delta S_g = S - S_0 \cong \frac{8(f^2 - f_0^2)}{3l} \quad (18)$$

These expressions can be used to evaluate the nonlinear behavior of cable-stays. Axial cable forces can define the start through the change in cable-stay deflection:

$$N_{in} = \frac{p_l \cdot l_i^2}{8(f_i - f_{vi})} \quad (19)$$

p_l —cable weight; f_{vi} —sag loss due to vertical displacement of nodes.

The discussed deflection change can be obtained by subtracting the geometrical cable length change from the elastic elongation [36]:

$$s_{0i} - s_{1i} = \frac{N_i \cdot l_i}{E_v A_v} \quad (20)$$

where

$$s_{1i} = \left(l_i + \Delta l_i + \frac{8(\Delta f_{0i} - \Delta f_{v1})^2}{l_i + \Delta l_i} \right) \quad (21)$$

$$s_{0i} = \left(l_i + \frac{8\Delta f_{0i}^2}{l_i} \right) \quad (22)$$

$$\Delta l_1 = \cos \alpha \cdot \Delta_C \quad (23)$$

$$\Delta l_2 = \frac{\cos \beta \cdot (\Delta_D - 2\Delta_C)}{2} \quad (24)$$

$$\Delta l_3 = \frac{\cos \gamma \cdot (\Delta_C - 2\Delta_D + \Delta_E)}{2} \quad (25)$$

$$\Delta l_4 = \frac{\cos \gamma \cdot (\Delta_F - 2\Delta_E + \Delta_D)}{2} \quad (26)$$

$$\Delta l_5 = \frac{\cos \beta \cdot (\Delta_E - 2\Delta_F)}{2} \quad (27)$$

$$\Delta l_6 = \cos \alpha \cdot \Delta_F \quad (28)$$

2.2. Calculation of String Members

The bridge string is designed as a continuous pre-stressed element with a length equal to the bridge span length (see Figure 1). However, due to the different axial tensile forces developing in the string internodes B–C and C–D (see Figure 2), separate string members are analyzed.

The string member B–C, subjected to the distributed load p and the vertical components of the forces transferred by the intersecting stays, experiences nodal displacements Δv and Δh , and deforms within the internode according to $w_1(x)$. The equilibrium condition of the string member B–C can therefore be written as follows (see Figure 6).

$$H_{bc} = \frac{p l^2}{8 \Delta f_1} \quad (29)$$

H_{bc} —the tensile force in the string internode B–C; a —the length of the string internode B–C; Δf_1 —the displacement at the midspan of the string internode B–C.

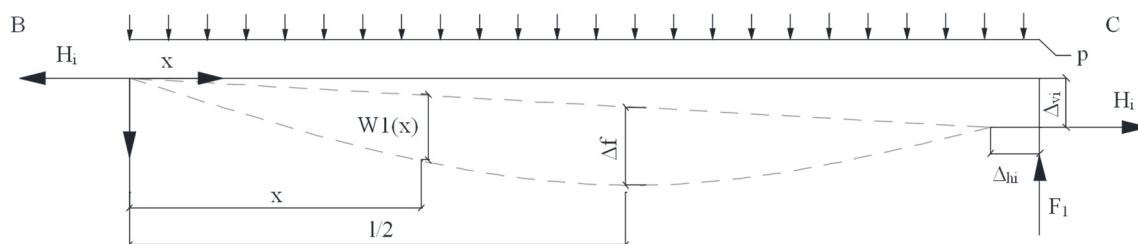


Figure 6. String calculation scheme B–C.

The vertical support reaction at node C is equal to

$$F_c = \frac{p l}{2} - H_{bc} \frac{\Delta v}{a} = \frac{p l}{2} \left(1 - \frac{\Delta v}{4 \Delta f_1} \right) \quad (30)$$

An additional condition for determining the unknowns is obtained from the deformation compatibility equation of the string member:

$$s_0 + \Delta s_{el} = s_1 \quad (31)$$

$$\Delta s_{el} = \frac{H_{bc} s_0}{EA} \quad (32)$$

$$s_1 = s_0 - \Delta h + \frac{8 \Delta f_1^2}{3 (l - \Delta h)} + \frac{\Delta v^2}{2 (l - \Delta h)} \quad (33)$$

$s_0 = a$ —initial length of the string internode; Δs_{el} —the elastic elongation of the string internode; s_1 —the length of the string internode after deformation; EA —the axial stiffness of the string.

After rearranging the expression, the midspan displacement Δf_1 of the string member can be determined when one end has a vertical displacement Δv and a horizontal displacement Δh , as shown in Figure 6:

$$\Delta f_1 = \sqrt[3]{\frac{3 p l^4}{64 EA d}} \quad (34)$$

$$d = 1 + \frac{3 \Delta v^2}{16 \Delta f_1^2} \pm \frac{3 \Delta h \cdot l}{8 \Delta f_1^2} \quad (35)$$

Here, d is a coefficient that accounts for the influence of node C displacements on the string displacement Δf_1 .

Then, string tension force can be described:

$$H_{bc} = \sqrt[3]{\frac{p^2 l^2 EA d}{24}} \quad (36)$$

Using Equations (34) and (35), the displacement of the string internode, Δf_1 , can be directly determined when the displacement of node C is known. The tensile force H_{bc} can then be calculated using the corresponding expression (Figure 7). These formulas are analogous to the previously presented equations for calculating a string member [12].

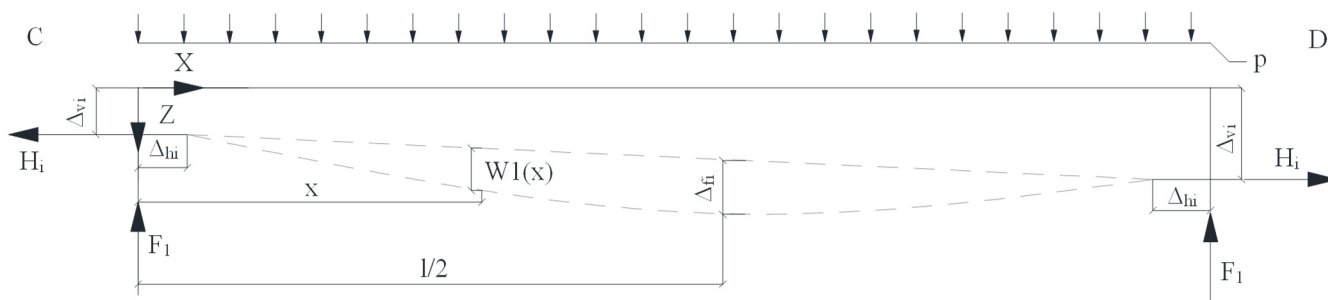


Figure 7. String calculation scheme C–D.

To reduce bridge displacements, the string may be pre-stressed in certain cases. If the ratio between the initial pre-stressing force of the string, N_0 , and the tensile force H_{pr1} is denoted by

$$n = \frac{N_0}{H_{pr1}} \quad (37)$$

we obtain the formula for calculating the midspan displacement of the string internode B–C

$$\Delta f_{pr,1} = \sqrt[3]{\frac{3 p l^4 (1 - n)}{64 EA d}} \quad (38)$$

Then, the expression for calculating the tensile force in the pre-stressed string member B–C can be written as follows:

$$H_{pr,bc} = \sqrt[3]{\frac{p^2 l^2 EA d}{24 (1 - n)}} \quad (39)$$

It can be seen from Equations (38) and (39) that pre-stressing reduces the string displacement $\Delta f_{pr,1}$ and increases the tensile force $H_{pr,bc}$.

To evaluate the multi-span string system, an auxiliary moment diagram is used only for calculation purposes. It does not represent actual bending of the string but helps to determine the tensile forces in different string internodes. The auxiliary moment function is obtained from the distributed load and the nodal forces transferred from the cable-stay system:

$$M(x) = M(x)_{dist} - M(x)_{nodal} \quad (40)$$

$$M_{Cnodal} = \left(\frac{4F_1 + 3F_2 + 2F_3 + F_4}{5} \right) l \quad (41)$$

$$M_{Dnodal} = \left(\frac{3F_1 + 6F_2 + 4F_3 + 2F_4}{5} \right) l \quad (42)$$

$$M_{Enodal} = \left(\frac{2F_1 + 4F_2 + 6F_3 + 3F_4}{5} \right) l \quad (43)$$

$$M_{Fnodal} = \left(\frac{F_1 + 2F_2 + 3F_3 + 4F_4}{5} \right) l \quad (44)$$

$$M_{Cdist} = 2 p \cdot l^2 \quad (45)$$

$$M_{Ddist} = 3 p \cdot l^2 \quad (46)$$

After defining the nonlinear behavior of the string and stay cables, a calculation sequence for the entire bridge system can be established. As noted previously, an iterative solution is required because the unknown parameters are interdependent. For symmetrical loading, the iterative procedure may be carried out as follows. First, initial assumed values of the displacements Δc and Δd at the string–cable connection points are selected, while Δe and Δf are taken as equal due to symmetry. Based on these assumed displacements, the axial forces in the stay cables N_i and their horizontal components N_{i-hor} can be determined [36]:

$$N_{1-hor} = \sin\alpha \cdot N_1 \quad (47)$$

$$N_{2-hor} = \sin\beta \cdot N_2 \quad (48)$$

$$N_{3-hor} = \sin\gamma \cdot N_3 \quad (49)$$

The support forces F_i and the fictitious bending moments at points C and D can then be calculated. Based on these values, the fictitious horizontal forces H_c and H_d are determined:

$$H_i = \frac{M(x)}{F_i} \quad (50)$$

The axial force H_{cd} is then determined as the average value of v . This step is used to estimate the axial force acting in the string internode between points C and D based on the corresponding fictitious force values.

$$H_{cd} = \frac{H_c + H_d}{2} \quad (51)$$

Having H_{cd} , H_{bc} and H_{de} can be defined:

$$H_{bc} = H_{cd} - H_{\Delta c} \quad (52)$$

$$H_{de} = H_{cd} + H_{\Delta d} \quad (53)$$

Here,

$$H_{\Delta c} = N_{1-hor} - N_{3-hor} \quad (54)$$

$$H_{\Delta d} = N_{2-hor} - N_{4-hor} \quad (55)$$

After defining the calculation procedure, the obtained results can be verified using Equations (34)–(36). The iterative cycle is repeated until the required accuracy is reached. The scheme of the proposed calculation approach is shown in Figure 8.

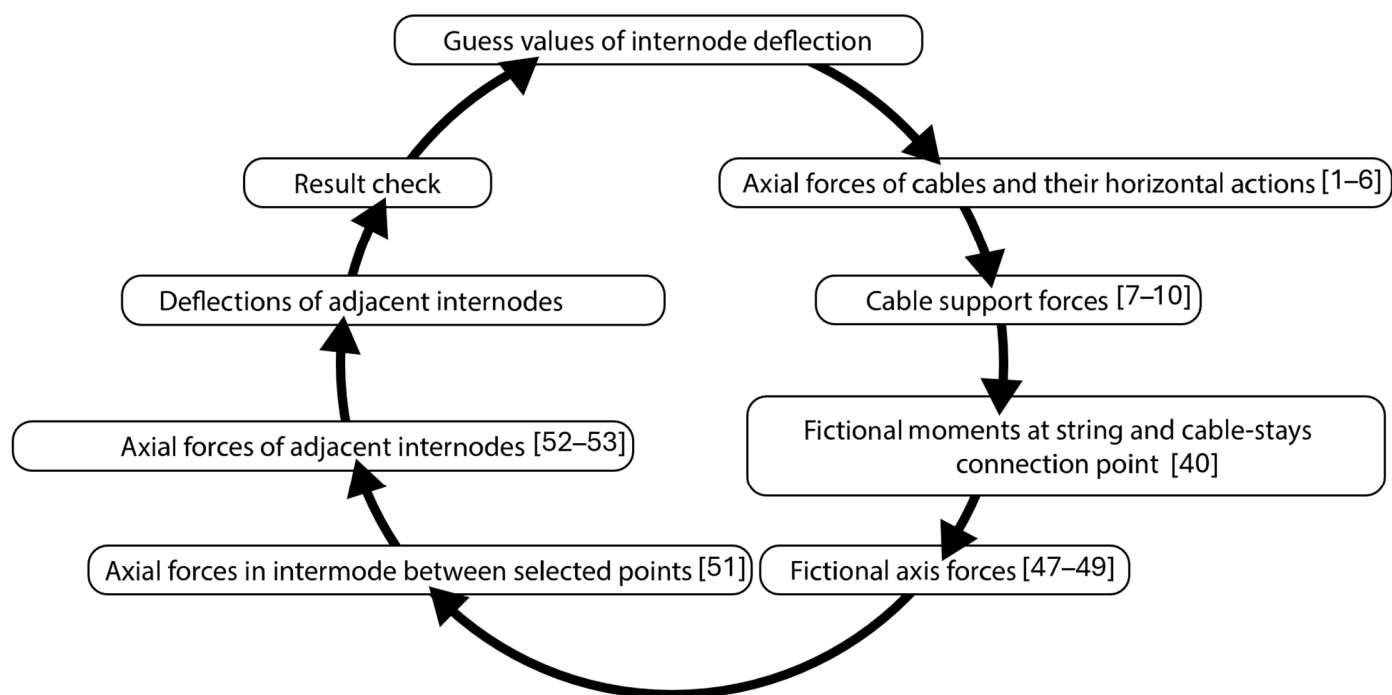


Figure 8. Calculation iteration sequence for bridge system.

These calculations were performed to verify that the proposed iterative method accurately predicts the displacements and internal forces of the string-cable system before applying it to the experimental model. For this numerical/analytical verification model, a string-cable system with the cable and pylon profiles listed in Table 1 was calculated using the proposed method and compared with the FE numerical modeling results. Therefore, the parameters in Table 1 refer to the verification model and not to the reduced-scale physical specimen described in Section 3. Although the main focus of this paper is the static behavior of the proposed system, a limited modal analysis was also conducted to indicate its natural vibration characteristics and possible serviceability limitations.

Table 1. Cross-sections of structural elements.

Element	Steel Type	Parameters	Finite Element Type
Cables	S960	$d = 0.05$ m	Cable
Pylons	S355	$d = 0.508$ m, $t = 0.03$ m	Beam
String	S960	$d = 0.10$ m	Cable

The results of this comparison are provided in Table 2.

Table 2. Systems forces and displacements under symmetrical loading.

	String			Cable Stays		
	Maximum Axial Force [kN]	Deflection [m]				Maximum Axial Force [kN]
		Δ_c	Δ_d	Δ_e	Δ_f	
FEM Model	6683	0.423	1.134	1.134	0.423	3761
Analytical solution	6472	0.417	1.122	1.118	0.413	3745

Deflection differences are under 2%, and axial force differences are under 4%, highlighting the accuracy of this calculation methodology.

The dynamic behavior of the cable-stayed-string bridge differs substantially from that typically observed in conventional cable-stayed bridges. Due to the relatively low

rotational stiffness of the system, which is governed by the cross-sections of the stay cables and their initial prestressing level, the structure exhibits very low natural frequencies. The results of the modal analysis showed that the first vibration mode has a frequency of only 0.037 Hz. This indicates that, when designing this type of bridge, additional structural stabilization measures must be considered, and a detailed dynamic analysis should be performed.

3. Physical Experiment

This chapter presents the experimental results obtained from the physical model with a geometrical scale of global dimensions to the full structure of 1:20 (Figure 9). The model consisted of four intermediate pylons and separate end pylons that supported a pre-tensioned steel string (Figure 10). Instead of conventional main-side pylons, in-place supports were used in this model. These supports were intended to limit possible horizontal displacements of the intermediate pylons during the experiment.

The ends of the pre-tensioned string were also anchored to the same point supports.



Figure 9. Cable-stayed bridge model.

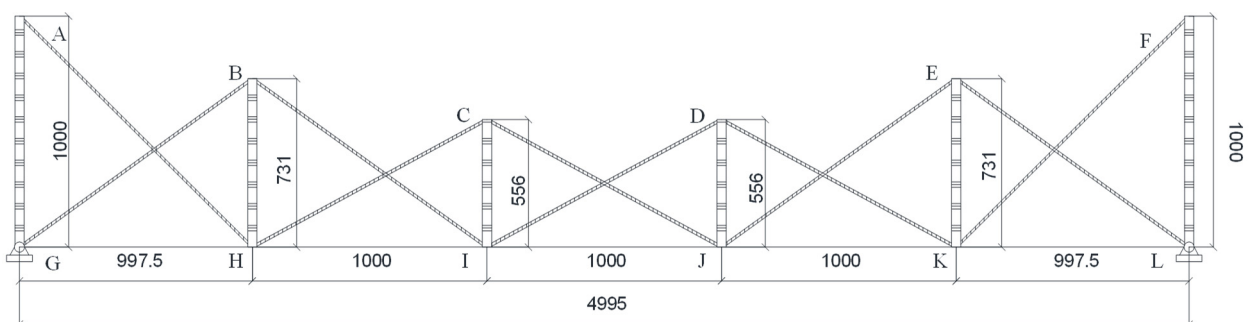


Figure 10. Geometrical parameters of the cable-stayed-string bridge model.

The total length of the bridge model was 4995 mm. The middle spans between the pylons were 1000 mm, while the outer spans were 997.5 mm. The outer pylons were 731 mm high, and the inner pylons were 556 mm high. The intermediate pylons were connected according to a parabolic curve. The vertical distance between the upper supports

and the lower anchorage was 1000 mm. The distance between the strings was 320 mm. The string and the outer anchorage stays were connected to support bearings that allowed for rotation around the horizontal axis (Figure 11).

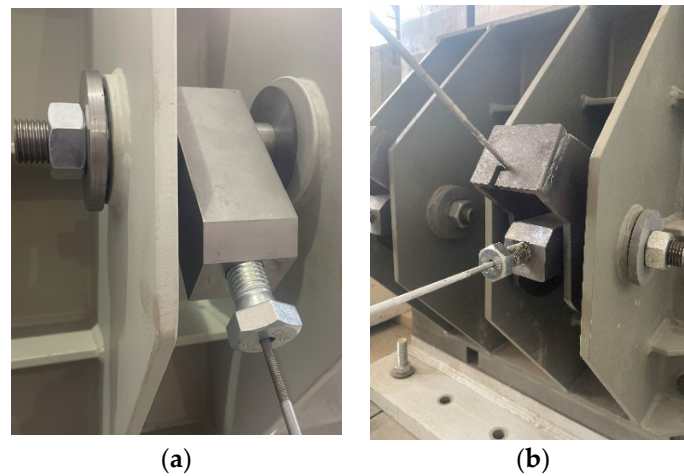


Figure 11. Anchorage stay connection at point “A” (a); support at point “L” (b).

To ensure uniform tensioning, the string element was designed as a continuous steel rod. All stay and string ends had threads intended for assembly and tensioning. The string was tensioned using anchorage stays at the support bearings and screw couplers. The cross-stays were connected to the pylons via extended screw couplers fitted to the stays’ threaded ends. The pylons were supported on the string and transferred forces to it, while the string was connected to the outer U-shaped supports by bolts. This ensured that the pylons would not move along the string’s longitudinal direction.

The string and diagonal stays were made of 6 mm diameter round steel rods from high-strength steel S700. The string was designed as a continuous steel rod to ensure uniform prestressing along the entire element. The pylon elements were made from 40 × 40 × 3 mm rectangular hollow sections from structural steel S355. These cross-sections were selected to limit pylon deformations and minimize their influence on the global deformation of the bridge model.

The supports of the anchorage stays were designed so that both the anchorage stay and the string could be tensioned without introducing additional stresses, particularly bending stresses. The pre-tensioning of the string and the stays is particularly important in this model because it maintains continuous tension in the diagonal stays. In addition, at certain loading levels, the outer spans of the string may become compressed. The axial forces of the model elements were measured using tensiometers (Figure 12).



Figure 12. Loaded internode C-D.

The pretensioning of the model elements was carried out gradually; its results are presented in Table 3. First, the string was prestressed before the pylons and cable-stays were installed. After that, pylons were fixed to the string once the prestress reached 2.38 kN. After that, the cable-stays were prestressed by tightening the fastening nuts at the ends of the threaded rods.

Table 3. Elements axial forces after prestressing steps [kN].

Prestressing Element	Outer String Internode (G–H and K–L)	Middle String Internode (I–J)	Anchor Cable-Stay (A–H and F–K)	Middle Cable-Stays (C–J and I–D)
	Strain Gauge 1	Strain Gauge 3	Strain Gauge 7	Strain Gauge 11&12
1. String pre-stressing	2.38	2.38	0	0
2. Cable stays prestressing	2.81	1.51	0.38	1.13

The displacement data for the string were obtained experimentally and numerically using the finite element method. These results are presented in Table 4, and sensor locations in Figure 13. The experimental and numerical string displacements differed by approximately 9–10%, regardless of whether symmetrical or asymmetrical loading was applied.

Table 4. Comparison of displacements.

Loading	Load Step	Deflection Data	Outer Span (G–H)	Inner Span (H–I)	Middle Span (I–J)
			Disp. Meter 3	Disp. Meter 5	Disp. Meter 7
Symmetrical	1	Test [mm]	16.05	14.96	15.95
		FEM [mm]	17.70	14.40	14.40
		Analytical	18.90	15.60	15.60
	2	Test [mm]	20.97	15.30	17.50
		FEM [mm]	20.20	16.00	16.00
		Analytical	22.50	17.60	17.70
Asymmetrical	1	Test [mm]	16.00	16.20	10.04

2	FEM [mm]	17.30	15.30	10.40
	Test [mm]	20.40	18.00	12.26
	FEM [mm]	19.60	17.00	11.80

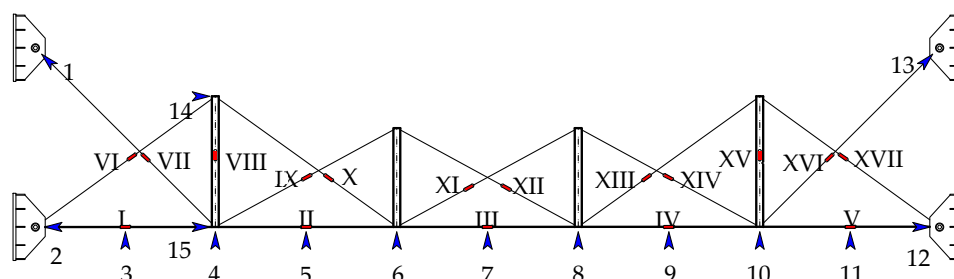


Figure 13. Strain gauges and displacement meters arrangement (red—strain gauges; blue—displacement meters).

The experimental displacements in the middle span of the string were smaller than those obtained numerically. In all other outer spans, the numerically calculated displacement values were higher.

4. Analytical, Experimental, and Numerical Results Analysis

This section summarizes, discusses, and verifies the results obtained from the calculation methodology, experimental investigation, and numerical analysis of the innovative cable-stayed-string bridge. The displacement values under symmetrical and asymmetrical loading are presented in Table 4. The presented data show that the string displacements calculated using the developed calculation methodology are approximately 6–8% higher than those obtained numerically and, on average, approximately 5–15% higher than those obtained experimentally.

The axial forces in the string and diagonal stays, obtained experimentally and numerically, are presented in Table 5. The results show that, in both loading stages, the axial forces calculated numerically in the string spans were higher than those obtained experimentally. The largest differences were observed in the outer string span, with differences not exceeding 7%. Within the middle string span, the maximum displacement difference in the first loading stage reached 19%; however, after increasing the load in the second stage, it decreased to 7%.

Table 5. Comparison of axial forces.

Loading	Load Step	Axial Force Data	Outer String Internode (G–H)	Inner Internode (H–I)	Middle String Internode (I–J)	Anchor Cable Stay (A–H)	Middle Stays (C–J, I–D)	
			Strain Gauge 1	Strain Gauge 2	Strain Gauge 3	Strain Gauge 7	Strain Gauge 11 and 12	
After prestress		Test [kN]	2.93	2.35	1.68	0.49	1.02	
		FEM [kN]	2.81	2.13	1.51	0.38	1.13	
Symmetrical	1	Test [kN]	4.013	4.76	5.03	2.26	0.68	
		FEM [kN]	4.26	5.49	6.02	2.13	0.03	
		Analytical	4.83	5.93	6.27	2.35	0.01	
	2	Test [kN]	4.53	5.35	6.67	2.63	0.51	
		FEM [kN]	4.69	6.26	6.94	2.50	0.02	
		Analytical	5.2	6.4	6.54	2.73	0.01	
Asymmetrical	1	Test [kN]	3.88	4.06	3.67	1.24	1.01	0.72
		FEM [kN]	4.29	4.81	4.27	1.85	0.84	0.08
	2	Test [kN]	4.32	4.61	4.53	1.66	0.93	0.52

FEM [kN]	4.76	5.44	5.78	2.15	0.87	0.01
----------	------	------	------	------	------	------

When analyzing the stress values in the stays, it was found that only the middle stays showed numerical model predictions lower than the measured forces. The largest differences between these values occurred after applying asymmetrical loading. This mismatch in the results was most likely due to inaccuracies between the models.

It should also be noted that, except for the middle diagonal stays, the axial forces measured in the string and anchorage stays during the experiment did not reach the values predicted by the numerical model.

5. Conclusions

1. The investigated innovative pre-stressed cable-stayed-string steel bridge with intersecting cable stays showed non-standard structural behavior. Since the stiffening girder is replaced by a pre-stressed string, loads are mainly transferred through axial forces in the string, stays, and pylons. A calculation method for this innovative bridge is presented, dividing this bridge into separate fragments consisting of an intermediate pylons, cable-stayed string, and string intermediate nodes.
2. This cable-stayed bridge calculation method presents an approximate calculation of the cable-stayed bridge as an inclined cable, assessing the load acting along the cable-stayed span and the cable-stayed displacements. It also discusses the nonlinear calculation of the load-bearing string supported by the cables, taking into account the displacements of the intermediate supports and the pre-tensioning of the string.
3. The analytical results agreed well with the finite element model under symmetrical loading. The difference in the maximum axial force of the string was approximately 3%, vertical displacement differences were below 2%, and the difference in the maximum axial force of the stays was approximately 0.5%.
4. The results of numerical and experimental analysis of the innovative bridge model showed that different tensile forces develop in separate string internodes due to the action of the intersecting stays and nodal displacements. Therefore, the evaluation of string displacements, stay forces, and force redistribution is essential for this bridge system.
5. The physical bridge model confirmed the general deformation pattern predicted by the numerical model. Experimental and numerical string displacements differed by approximately 9–10% for both symmetrical and asymmetrical loading. The largest discrepancies were observed in the diagonal stays, especially under asymmetrical loading.
6. It was found that the tensile (axial) force of the string increases slightly as it approaches the middle of the bridge span and depends on the vertical and horizontal displacements of its nodes and the applied prestressing. The axial forces of the stay cables, on the contrary, decrease as it approaches the middle of the bridge. Under the action of an asymmetric load, this bridge, unlike conventional cable-stayed bridges, deforms evenly and the maximum string displacements do not exceed the displacements under symmetrical loading.

Author Contributions: Conceptualization, P.D.; Methodology, P.D. and A.J.; Software, P.D.; Validation, P.D. and A.J.; Investigation, P.D.; Data curation, P.D.; Writing—original draft, P.D.; Writing—review & editing, A.J.; Visualization, P.D.; Supervision, A.J. All authors have read and agreed to the published version of the manuscript.

Funding: This research received no external funding.

Institutional Review Board Statement: Not applicable.

Informed Consent Statement: Not applicable.

Data Availability Statement: The original contributions presented in this study are included in the article. Further inquiries can be directed to the corresponding author.

Conflicts of Interest: The authors declare no conflicts of interest.

References

1. Svensson, H. *Cable-Stayed Bridges: 40 Years of Experience Worldwide*; Wilhelm Ernst & Sohn: Berlin, Germany, 2015. <https://doi.org/10.1002/9783433601044>.
2. Idelberger, K. *The World of Footbridges: From the Utilitarian to the Spectacular*; Wilhelm Ernst & Sohn: Berlin, Germany, 2011. <https://doi.org/10.1002/9783433600849>.
3. Strasky, J. *Stress Ribbon and Cable-Supported Pedestrian Bridges*, 2nd ed.; ICE Publishing: London, UK, 2011. <https://doi.org/10.1680/srcspb.41462>.
4. FIB. *Guidelines for the Design of Footbridges*; International Federation for Structural Concrete: Lausanne, Switzerland, 2005. Available online: <https://shop.fib-international.org/publications/fib-bulletins/424-guidelines-for-the-design-of-footbridges-pdf> (accessed on 1 March 2026).
5. Pedro, J.J.O.; Reis, A.J. Composite cable-stayed bridges: State of the art. *Proc. Inst. Civ. Eng. Bridge Eng.* **2015**, *169*, 13–38. <https://doi.org/10.1680/bren.14.00005>.
6. Abdelaziz, M.M.; El-Ghazaly, H.A.; Gomaa, M.S. Numerical modeling for collapse analysis of cable-stayed bridges using the improved applied element method. *Iran. J. Sci. Technol. Trans. Civ. Eng.* **2024**, *7*, 2891–2904. <https://doi.org/10.1007/s40996-024-01343-7>.
7. Atmaca, B. Size and post-tensioning cable force optimization of cable-stayed footbridge. *Structures* **2021**, *33*, 2036–2049. <https://doi.org/10.1016/j.istruc.2021.05.050>.
8. Ferreira, F.; Simões, L. Optimum design of a cable-stayed steel footbridge with three-dimensional modelling and control devices. *Eng. Struct.* **2019**, *180*, 510–523. <https://doi.org/10.1016/j.engstruct.2018.11.038>.
9. Gimsing, N.J.; Georgakis, C.T. *Cable Supported Bridges: Concept and Design*; John Wiley & Sons: Chichester, UK, 2011. <https://doi.org/10.1002/9781119978237>.
10. Cho, S.; Yim, J.; Shin, S.W.; Jung, H.-J.; Yun, C.-B.; Wang, M.L. Comparative field study of cable tension measurement for a cable-stayed bridge. *J. Bridge Eng.* **2013**, *18*, 748–757. [https://doi.org/10.1061/\(ASCE\)BE.1943-5592.0000421](https://doi.org/10.1061/(ASCE)BE.1943-5592.0000421).
11. Martins, A.M.B.; Simões, L.M.C.; Negrão, J.H.J.O. Optimization of cable-stayed bridges: A literature survey. *Adv. Eng. Softw.* **2020**, *149*, 102829. <https://doi.org/10.1016/j.advengsoft.2020.102829>.
12. Tubino, F.; Pagnini, L.; Piccardo, G. Uncertainty propagation in the serviceability assessment of footbridges. *Struct. Infrastruct. Eng.* **2020**, *16*, 123–137. <https://doi.org/10.1080/15732479.2019.1618879>.
13. Bassoli, E.; Gambarelli, P.; Vincenzi, L. Human-induced vibrations of a curved cable-stayed footbridge. *J. Constr. Steel Res.* **2018**, *146*, 84–96. <https://doi.org/10.1016/j.jcsr.2018.02.001>.
14. Qiao, H.; Guan, H.; Zhu, Y. Footbridge structural health monitoring—A review of current research and future directions. *Struct. Infrastruct. Eng.* **2025**, 1–24. <https://doi.org/10.1080/15732479.2025.2547349>.
15. Sharry, T.; Guan, H.; Nguyen, A.; Oh, E.; Hoang, N. Latest advances in finite element modelling and model updating of cable-stayed bridges. *Infrastructures* **2022**, *7*, 8. <https://doi.org/10.3390/infrastructures7010008>.
16. Innocenzi, R.D.; Nicoletti, V.; Arezzo, D.; Carbonari, S.; Gara, F.; Dezi, L. A good practice for the proof testing of cable-stayed bridges. *Appl. Sci.* **2022**, *12*, 3547. <https://doi.org/10.3390/app12073547>.
17. Beivydas, E.; Juozapaitis, A. Experimental and Numerical Analysis of an Innovative Combined String–Cable Bridge. *Appl. Sci.* **2024**, *14*, 7542. <https://doi.org/10.3390/app14177542>.
18. Beivydas, E.; Juozapaitis, A.; Paeglite, I. Experimental and analytical studies of string steel structure for bridges. *Balt. J. Road Bridge Eng.* **2023**, *18*, 145–165. <https://doi.org/10.7250/bjrbe.2023-18.622>.
19. Robin, C.; Datry, J.-B.; de Kosmi, B. La passerelle du Grand Large à Dunkerque. *Travaux* **2013**, *896*, 82–91.
20. Cid, C.; Baldomir, A.; Hernández, S. Optimum crossing cable system in multi-span cable-stayed bridges. *Eng. Struct.* **2018**, *160*, 342–355. <https://doi.org/10.1016/j.engstruct.2018.01.019>.
21. Arellano, H.; Gómez, R.; Tolentino, D. Parametric analysis of multi-span cable-stayed bridges under alternate loads. *Balt. J. Road Bridge Eng.* **2019**, *14*, 543–567. <https://doi.org/10.7250/bjrbe.2019-14.457>.

22. Arellano, H.; Tolentino, D.; Gómez, R. Optimum criss-crossing cables in multi-span cable-stayed bridges using genetic algorithms. *KSCE J. Civ. Eng.* **2019**, *23*, 719–728. <https://doi.org/10.1007/s12205-018-5736-2>.
23. Dabrila, P. Behavior of pre-stressed intersecting cable steel bridge. *Eng. Struct. Technol.* **2021**, *13*, 19–25. <https://doi.org/10.3846/est.2021.18402>.
24. Straupe, V.; Paeglitis, A. Analysis of interaction between the elements in cable stayed bridge. *Balt. J. Road Bridge Eng.* **2012**, *7*, 84–91.
25. Gai, T.; Yu, D.; Zeng, S.; Lin, J.C.-W. An optimization neural network model for bridge cable force identification. *Eng. Struct.* **2023**, *286*, 116056. <https://doi.org/10.1016/j.engstruct.2023.116056>.
26. Hassan, M.M.; Nassef, A.O.; El Damatty, A.A. Determination of optimum post-tensioning cable forces of cable-stayed bridges. *Eng. Struct.* **2012**, *44*, 248–259. <https://doi.org/10.1016/j.engstruct.2012.06.009>.
27. Janjic, D.; Pircher, M.; Pircher, H. Optimization of cable tensioning in cable-stayed bridges. *J. Bridge Eng.* **2003**, *8*, 131–137. [https://doi.org/10.1061/\(ASCE\)1084-0702\(2003\)8:3\(131\)](https://doi.org/10.1061/(ASCE)1084-0702(2003)8:3(131)).
28. Sernizon Costa, R.; Cesar Campos Lavall, A.; Gomes Lanna da Silva, R.; Porcino dos Santos, A.; Francisco Viana, H. Cable structures: An exact geometric analysis using catenary curve and considering the material nonlinearity and temperature effect. *Eng. Struct.* **2022**, *253*, 113738. <https://doi.org/10.1016/j.engstruct.2021.113738>.
29. Thai, H.T.; Kim, S.E. Nonlinear static and dynamic analysis of cable structures. *Finite Elem. Anal. Des.* **2011**, *47*, 237–246. <https://doi.org/10.1016/j.finel.2010.10.005>.
30. Wen, Y.; Zhou, Z. Qualification of the Ernst formula for modeling the sag effect of super-long stay cables in long-span railway cable-stayed bridges. *Structures* **2022**, *45*, 99–109. <https://doi.org/10.1016/j.istruc.2022.09.002>.
31. Kaufmann, W.; Karagiannis, D.; Widmer, K. Load factors for permanent actions and cable preload in cable-stayed bridges. *Struct. Eng. Int.* **2021**, *31*, 10–17. <https://doi.org/10.1080/10168664.2019.1667738>.
32. Farré-Checa, J.; Komarizadehasl, S.; Ma, H.; Lozano-Galant, J.A.; Turmo, J. Direct simulation of the tensioning process of cable-stayed bridge cantilever construction. *Autom. Constr.* **2022**, *137*, 104197. <https://doi.org/10.1016/j.autcon.2022.104197>.
33. Guo, J.; Guan, Z. Optimization of the cable forces of completed cable-stayed bridges with differential evolution method. *Structures* **2023**, *47*, 1416–1427. <https://doi.org/10.1016/j.istruc.2022.12.004>.
34. Song, C.; Xiao, R.; Sun, B.; Wang, Z.; Zhang, C. Cable force optimization of cable-stayed bridges: A surrogate model-assisted differential evolution method combined with B-spline interpolation curves. *Eng. Struct.* **2023**, *283*, 115856. <https://doi.org/10.1016/j.engstruct.2023.115856>.
35. Ma, Y.; Song, C.; Wang, Z.; Jiang, Z.; Sun, B.; Xiao, R. Efficient design optimization of cable-stayed bridges: A two-layer framework with surrogate-model-assisted prediction of optimum cable forces. *Appl. Sci.* **2024**, *14*, 2007. <https://doi.org/10.3390/app14052007>.
36. Dabrila, P.; Juozapaitis, A. Calculation of pre-stressed cable-stayed steel bridge with intersecting cable stays. *Bauingenieur* **2024**, *99*, 78–89. <https://doi.org/10.37544/0005-6650-2024-03-38>.
37. Dabrila, P.; Juozapaitis, A. Numerical and experimental investigation of a new cable-stayed string steel bridge. *Bauingenieur* **2024**, *99*, 396–405. <https://doi.org/10.37544/0005-6650-2024-12-46>.

Disclaimer/Publisher’s Note: The statements, opinions and data contained in all publications are solely those of the individual author(s) and contributor(s) and not of MDPI and/or the editor(s). MDPI and/or the editor(s) disclaim responsibility for any injury to people or property resulting from any ideas, methods, instructions or products referred to in the content.