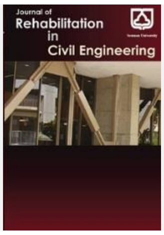

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Effect of Initial Cable Tension on Cable-Stayed Bridge Performance and a New Construction Method

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ABSTRACT

The tension forces of cables in cable-stayed bridges during their construction and operation may differ from initial conditions, which could affect the amount and distribution of internal forces on other parts of the bridge. If some bridge elements are loaded to their yield limits, alternating plasticity and incremental collapse could occur under moving and repeated loading. This study investigated the effect of the initial tension force on the stiffness, strength, shakedown limit load, and alternating plasticity of cable-stayed bridges, which has not been studied so far. Two case study bridge models with cables having different initial forces were tested using nonlinear static analysis under gravitational force and nonlinear dynamic analysis under transient moving loads. The results showed that changes in the initial cable forces did not change the initial stiffness, ultimate strength, shakedown limit, alternating plasticity, or bridge reliability. These results were theoretically validated using plastic analysis theorems. This paper presents a new construction method for cable-stayed bridges based on the finding that adjusting the cable tension to the design value is not necessary during construction and operation. This method eliminates the need for tension adjustment and ensures that the final geometric shape of the bridge matches the expected profile. The proposed method offers a simpler and more efficient approach to constructing cable-stayed bridges without compromising the safety and durability of the structure.

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1. Introduction

Cables are the main components of a cable-stayed bridge. Pre-stressing of the cables is a critical factor in determining the moments and forces of cable-stayed bridge components and can significantly influence their structural behavior. This means that pre-stressing adjustment may be necessary to ensure the optimal design of the bridge, which is crucially important for its serviceability and long-term performance. There are several methods for adjusting the cable tension to produce an optimal design of the cable-stayed bridge, significantly dividing them into three categories: Displacement method [1], Force equilibrium method [2] or Unit load method [3] and Optimization method [4–9].

It is necessary to define a target state before designing a cable-stayed bridge. This stage is known as the Objective Service Stage (OSS) under some given loads known as Target Load [10]. To achieve this, an appropriate set of stay forces must be defined [11].

It is also important to note that since cable-stayed bridges consist of redundant components, tensing a single strand has an adverse effect on the other components of the bridge. For these reasons, a great deal of research has been conducted on the construction process of cable-stayed bridges [12–15].

Traditional methods of simulating cable-stayed bridge construction involve starting at the OSS and dismantling it [16,17]. This backward approach has been proposed by several authors [10,18,19]. However, these methods will not be able to analyze the effects of time-dependent phenomena [20] unless they are combined with a global iterative process or a backward-forward analysis. The forward simulation has been proposed to solve all these problems [18,19,21,22]. In most simulation methods, elements, loads, and boundary conditions from the following or previous construction stage can be deactivated or activated to construct any construction stage. It is assumed that linear superposition of stages can simulate the construction process [10,21]. Superposition is avoided in other methods [23] and time-dependent phenomena are incorporated in the definitions of OSS [24]. Zhang provided a new calibration method based on a kriging surrogate model is proposed for cable-stayed bridges. The method provides an effective interpolation technique for iteration without any further finite element analyses and can be utilized effectively in a staged calibration procedure to provide reasonable estimates of critical cable forces [25]. Moreover, complex structures will be able to be controlled [26] and detected [27] by implementing the Health Monitoring field [28]. It is possible to monitor cable-stayed bridges while they are being constructed and while they are in use. This data can be used to calibrate models related to structural management and maintenance to ensure structural safety and functionality.

Structures like these are extremely redundant, and tensing one cable changes the stresses of the other cables. During the construction process, the contractor should perform careful calculations to ensure that the project has reached its target service state. However, there are deviations between the results obtained on site and the results predicted by a model of the tensioning process. A final restress of the stays is usually required to adjust the final stresses in the cables. To ensure that the cables are reached to their design tension, a comprehensive re-stressing operation needs to be conducted on the whole cable. This cannot be done with strand-by-strand stressing technique as used for the initial stressing operations, and therefore requires a more comprehensive process [29]. The last operation in the traditional method of strand tensioning can be extremely expensive, time-consuming and lacks accuracy when compared to the improved strand-by-strand tensioning techniques [23]. Lozano and Turmo present a novel approach to modeling cable-stayed bridge construction, utilizing an actual sequence of events at site. Construction control of cable-stayed bridges is made easier and there is no need for any additional tensioning stage on site to correct the differences between calculated and measured axial forces [30].

Despite adjusting the tension of the cables, during the construction and subsequent operation of the bridge, the tension force of the cables will inevitably change due to construction errors and other environmental factors such as temperature, creep and shrinkage. Construction errors can include errors in the jacking of the cables or a geometric error when controlling the elevation of the deck. Self-balancing strain effects such as temperature, shrinkage, and creep can also change the initial cable forces [31,32].

Furthermore, the geometry control method offers an alternative approach to constructing cable-stayed bridges [33]. In contrast to the previously mentioned methods, this technique focuses on adjusting the position of the bridge deck during the installation stage rather than adjusting the cable tension. However, it is important to note that in all the geometry control methods discussed in prior research, the tension force of the cables is still adjusted. For this purpose, either the tension forces of the cables are adjusted to reach their design forces, or the structural models are updated again using the tension forces of the cables in the final stage of the bridge construction and the design of the structure is controlled again [34].

Also, change in the initial tension force of the cables can push the internal forces of the bridge elements up to their yield limits. When this occurs, moving loads and repeated bridge loads can create alternating plasticity and incremental collapse of the structure. The present study investigated the effect of changes in the initial tension force of the cables on the shakedown limit and alternating plasticity in cable-stayed bridges.

Shakedown occurs when cyclic and repeated loads in structures fall into the plastic load limit. If repetitive loading such as those from moving vehicles are between the first yield stress and collapse load, the following types of elasto-plastic behavior could occur: elastic shakedown, plastic shakedown, and incremental collapse (ratcheting). Fig. 1 is a schematic of different material and structural behaviors under cyclic and alternating loads [35].

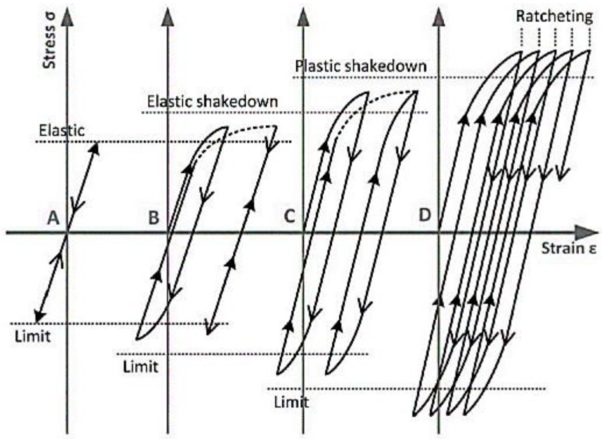

Fig. 1. Different behaviors of materials under alternating loads [35].

Shakedown has been investigated in continuous beams, frames, and arches under cyclic loading, but few studies have examined shakedown under moving loads [36]. Eyre experimentally studied shakedown in a bridge with two spans at fixed distances under concentrated moving loads. This study showed that strain hardening and restraint against buckling is required to prevent shakedown [37].

Chentov calculated shakedown loads using influence lines for plastic rotation. The effect of shear loads was considered using a perfect elasto-plastic model. The experimental results showed that the shakedown occurred after the specified loadings [38]. Lamblin and save examined shakedown in uniform beams [39].

Cichon investigated shakedown in arch beams under moving loads. In their study, the shakedown loads were calculated to protect the structure against ratcheting [40].

Cichon determined the precise shakedown load for an arch bridge. An elasto-plastic hardening material with Bauschinger effect was considered in this research. It was found that the shakedown limit could be predicted from the initial moving loads [41]. Hubel and Vollrath studied alternating plasticity and ratcheting in a continuous beam with three supports under a moving load. The results showed that the direction of the moving load affected the accumulated deformation. Moving loads can either increase or decrease ratcheting [42–46]. This relates to material strain hardening, elastic support, equilibrium conditions on the deformed system (second-order theory), and finite rotation (third-order theory) [47].

The effects of changing in tension forces of cables on the shakedown limit and alternating plasticity of cable-stayed bridges have not been studied in any previous studies. In this research, these effects have been studied analytically and numerically.

In summary, the change in the tensions of cables relative to the computational value, can lead to the following concerns:

• Increase bridge deflections (stiffness)

• Reduction in the ultimate load capacity of the bridge (strength)

• Creation of potential of incremental collapse and low cycle fatigue (shakedown limit)

This study examines all the above issues theoretically and with a case study modeling. In the next section, the problem has been analyzed using fundamental plastic analysis theorems. To ensure the analytical findings, a case study has been modeled by FE method in the next section. Additionally, based on the results of this study, a simple method for building cable-stayed bridges is proposed by eliminating the steps of readjusting cable forces to the assumptions of design.

2. Development of Problem Theory

2.1. Introduction

The study investigated the impact of altering the initial tension of the cables on the structural behavior of a cable-stayed bridge, including stiffness, strength, and shakedown-limit load. Analytical methods based on fundamental theorems for plastic analysis were employed. The hypotheses under study were:

- The cables have a minimum tensile stress of $0.2F_u$ and maximum $0.9F_u$. This assumption means controlling the structure according to the design regulations and establishing the conditions of balance and submission in the structure.
- Structural deformations fall into the range of small displacements.

In the analytical study, the structure was assumed to have sufficient ductility. This made it possible to study brittle failure such as buckling and fatigue, separately.

2.2. Examining stiffness changes

The vertical stiffness of a cable-stayed bridge relates to the stiffness of the cables and the main members of the bridge, deck, and pylons. With the assumption that the tension of the cables had changed, changes in the stiffness of the bridge were assumed to be due to changes in the stiffness of the cables due to the changes in their initial tension. The axial stiffness of the cable was considered by modifying the elastic modulus of the cable considering the effect of tensile force. The effective elastic modulus of the cable can be calculated according to the relationship proposed by Ernst [48] as:

$$E_{eff} = \frac{E_0}{1 + \frac{\gamma^2 l_k^2 E_0}{12\sigma^3}} \quad (1)$$

where E_0 is the modulus of elasticity of a straight cable, γ is the density of the cable, l_k is the horizontal length of the cable, and σ is the tensile stress in the cable. As can be seen from the relationship of Ernst [48], the effects of changes in the tensile stress in the cable on the axial stiffness of the cable were greatly reduced to near zero with an increase in the tensile stress. The minimum tensile stress in the cables was assumed to be $0.2F_u$. Because the yield stress of cables commonly used in bridges is 1700 to 1900 MPa, the minimum tensile stress in cables at $0.2F_u$ should be about 350 MPa. Therefore, according to the Ernst relation, assuming that the horizontal length of the cable was 100 m, and the tensile stress was 350 MPa, the effective hardness of the cable was 200087 MPa, which is about 2% less than the hardness of a straight cable.

Hence, if the variations in cable tension fall within the specified range ($0.2Fu< \sigma <0.9Fu$), the overall stiffness of the bridge remains largely unaffected and resulting in minimal alterations to its vibrational behavior.

2.3. Changes in final resistance of bridge

In accordance with the uniqueness theorem of plasticity analysis, suppose a set of loads on a frame produces bending moments which are statically admissible, safe, and result in a mechanism. The loads will then be equal to the collapse load ($\lambda=\lambda c$) [49].

It was assumed that there were two cable-stayed bridges with the same geometry and section characteristics which differed only in their initial cable tensions. The tension of the cables created a distribution of internal forces in the deck and pylon that was balanced by an external load that was equal to zero. In other words, they were self-balanced.

In the uniqueness theorem of plasticity analysis, for the collapse load of the first bridge, there was a balanced set of moments (M*) with external factored loads ($\lambda c1W$) that did not cross the yield point at any location and did not create a mechanism for the structure. The set of internal forces of the structure also included the effects of cable tension as represented by M*. In this model, ($\lambda c1W$, M*+m*) is an equilibrium set and is a compatible mechanism; thus:

$$(M^* + m^*) \le M_p \quad (2)$$

$$\lambda_{c1} \sum W\delta^* = \sum (M^* + m^*)\theta^* = \sum M_p \left|\theta^*\right| \quad (3)$$

$$0 = \sum m^* \theta^* \quad (4)$$

Due to the self-balancing of the moments and the forces caused by the initial tension of the cables, for any set of cable tensions considered, the sum of the balanced moments, external loads, and self-balancing moments due to cable tension will be in equilibrium with the external loads.

Given the lack of change in the geometry and stiffness of the structure, as well as in the external loads in the second bridge, the balanced moment distribution of the first bridge for the collapse mechanism also was assumed for the second bridge. This distribution of moments was balanced by external loads ($\lambda c1$). Also, because the resistance of the bridge members did not change, the non-yielding condition and establishment of the mechanism was established in the second bridge. Therefore, according to the uniqueness theorem, the collapse factor of the second bridge should be equal to the collapse factor of the first bridge ($\lambda_{c1} = \lambda_{c2}$).

This shows that, despite a change in the initial stresses of the cables in two different models, because these stresses created internal self-balancing forces and had no effect on the external load and strength of the members, their collapse load and the final strength of the structure did not change ($\lambda_{c1}=\lambda_{c2}$).

It is important to note that the obtained result remains valid only if the variations in tensile stress of the cables fall within the originally assumed range (0.2Fu< σ <0.9Fu).

2.4. Shakedown limit load

If the structure is affected by variable loading, the internal forces at different points in the structure will experience different values. This will also occur in bridges due to the passage of moving loads. Thus, the possibility of shakedown or incremental collapse occurring with changes in the initial tension of the cables in the cable-stayed bridges was investigated.

In accordance with the uniqueness shakedown theorem, suppose that, at the load factor λ, it is possible to find a distribution of residual bending moments mi that are statically admissible and satisfy the inequalities listed below [50]. If a mechanism is produced at this load factor, then it should be equal to shakedown limit λs.

$$m_i^* + \lambda M_i^{\max} \le M_p^{k(i)} \tag{5}$$

$$m_i^* + \lambda M_i^{\min} \ge M_p^{k(i)} \tag{6}$$

$$\lambda(M_i^{\max} - M_i^{\min}) \le 2M_y^{k(i)} \tag{7}$$

Where M_{max} and M_{min} are the maximum and minimum elastic moments on the structure are under all alternating loading conditions.

The third condition prevents alternating plasticity. If load coefficient λ exists in at least one point of the structure, the third condition will not be satisfied; however, the other conditions will be met, and alternating plasticity will govern the behavior of the structure. In technical literature, this condition is also called plastic shakedown [35].

If the load assigned in relation to the applied coefficient causes a mechanism, the actual moment remaining after the load passes over the structure ($\bar{m}_j$) at the mechanism must apply as:

$$\bar{m}_j + \lambda M_j^{\max} = M_p^{k(i)} \quad if \quad \theta_j > 0 \tag{8}$$

$$\bar{m}_j + \lambda M_j^{\min} = M_p^{k(i)} \quad if \quad \theta_j < 0 \tag{9}$$

To calculate λs, the following relations can be obtained by examining all possible states of the mechanism and by establishing a virtual work relationship. Given that the moments remaining in the structure are balanced, then the work done by them is zero.

$$\sum_{j=1}^{N_p} \bar{m}_j \theta_j = 0 \tag{10}$$

$$\lambda \sum_{j=1}^{N_p} M_j^{max} \theta_j = \sum_{j=1}^{N_p} M_p^{k(j)} \theta_j \quad if \ \theta_j > 0 \tag{11}$$

$$\lambda \sum_{j=1}^{N_p} M_j^{min} \theta_j = -\sum_{j=1}^{N_p} M_p^{k(j)} \theta_j \quad if \ \theta_j < 0 \tag{12}$$

In literature, Eqs. (10) Through (12) are usually combined into a single expression as:

$$\lambda \sum_{j=1}^{N_p} \begin{pmatrix} M_j^{max} \\ M_j^{min} \end{pmatrix} \theta_j = \sum_{j=1}^{N_p} M_p^{k(j)} \left|\theta_j\right| \tag{13}$$

Therefore, to calculate the shakedown limit load, all the states of the probable collapse mechanism must be considered, and the load coefficient must be calculated for them using Eq. (13). In addition to these collapse mechanisms, alternating the plasticity mechanisms from Eq. (14) can be calculated as all possible point as:

$$\lambda\left(M_i^{max} - M_i^{min}\right) = 2M_y^{k(i)} \tag{14}$$

Where M is the set of all possible collapse mechanisms. Using the upper bound theorem for shakedown, the mechanism with the smallest value of λ is the shakedown limit as:

$$\lambda_s = \min_{m \in M}(\lambda_m) \tag{15}$$

If the alternating plasticity mechanisms are the smallest, the structure will be exposed to alternating plasticity (plastic shakedown) before incremental collapse occurs and low-cycle fatigue will occur in the bridge members.

As can be seen in Eqs. (13) and (14) that shakedown limit load coefficient λs was obtained regardless of the amount of moment distribution remaining in the structure ($\overline{m}_j$). Therefore, the residual moment distribution did not affect the shakedown load factor.

Due to the self-balancing of the initial tensions of the cables, its effect on the cables was like the distribution of the remaining self-balancing moment. Therefore, the amount of tensile force in the cables and the primary moments in the bridge had no effect on the shakedown limit. The strength of the bridge members ($M_p^{k(i)}$) and the elastic stiffness of the bridge affected the values and were the only factors affecting the shakedown limit load.

Due to the self-balancing stresses like creep, shrinkage, fabrication defects, and other factors that lead to changes in cable tension do not impact the external loads. Consequently, these effects will not affect the stiffness, strength, and alternating loads. Therefore, it was not necessary to reset the tension of the cables after execution or after operation to achieve the forces of the initial design of the bridge.

It is important to note that the obtained result remains valid only if the variations in tensile stress of the cables fall within the originally assumed range (0.2Fu< σ <0.9Fu).

3. Case study and analysis method

In the present study, a cable-stayed bridge was analyzed that considered two cases of initial cable tension force. In case 1, the cable force was adjusted to obtain the optimum moment for proper design of the deck and all members. In the case 2, the initial cable force was adjusted to obtain a minimum cable stress of greater than $0.2F_u$. The stiffness, capacity, and shakedown loads of each case were investigated under pushdown analysis (nonlinear static analysis in the gravitational direction) and dynamic moving load analysis.

3.1. Model of structure

The bridge selected for the case study was adapted from a project that is under construction the city of Basrah in Iraq. This bridge has a length of 320m. Its deck consists of two steel boxes with concrete slabs (composite deck). The diamond pylon consists of a steel portion above deck and a concrete portion below the deck. The bridge has three spans, with the main one having a mid-span length of 160m.

The slab of the deck is 250mm with a steel box of 1750mm in width and 2000mm in height. The pylon height is 70 m from the top of the foundation, which has a rectangular shape of 4x6m. The upper part of the steel box is 750x1500mm. The secondary girder is a wide flange section as shown in Fig. 2. The steel and concrete properties are shown in Table 1.

Table 1. Steel and concrete properties of the bridge.

Density (ton.m3)	Poisson coefficient (ν)	Compressive specified strength (f_C') (MPa)	Yield stress (f_y) (MPa)	Elasticity modulus (E) (GPa)	Properties
7.85	0.3	---	360	200	steel
2.5	0.2	30	---	7.25	concrete

The 2D modeling approach was employed in this study. The cable inclination of the cable plane has been ignored. Only the main girder and half of the pylon were modeled. Simple roller support was considered for the end support of the girder (only vertical displacement was restrained). The other supports of the pylon were fixed. The pylon, deck girder, and cable were modeled as frame elements. Plastic bending hinges were used for the ends of the pylon at the cable connection part and the deck girder beams. Strain hardening was assumed to be 3% after yielding and Axial-bending hinge was assumed for the pylon.

To determine the shakedown loads, initial stiffness, and gravitational capacity, two models with two initial cable force conditions were considered. Both models had the same number and types of members with different initial cable tension forces. In case 1, the cable forces were adjusted to obtain the optimum moment in the decks and all member designs. In case 2, the initial cable force was adjusted to obtain a minimum cable stress of greater than $0.2F_u$.

Table 2 shows the specifications of the cables, initial tension of the cables under dead loads in each model, and the difference between the tension values in the two models. As shown in Table 2, the force on the cables ranged from -46% to +70% in different cables.

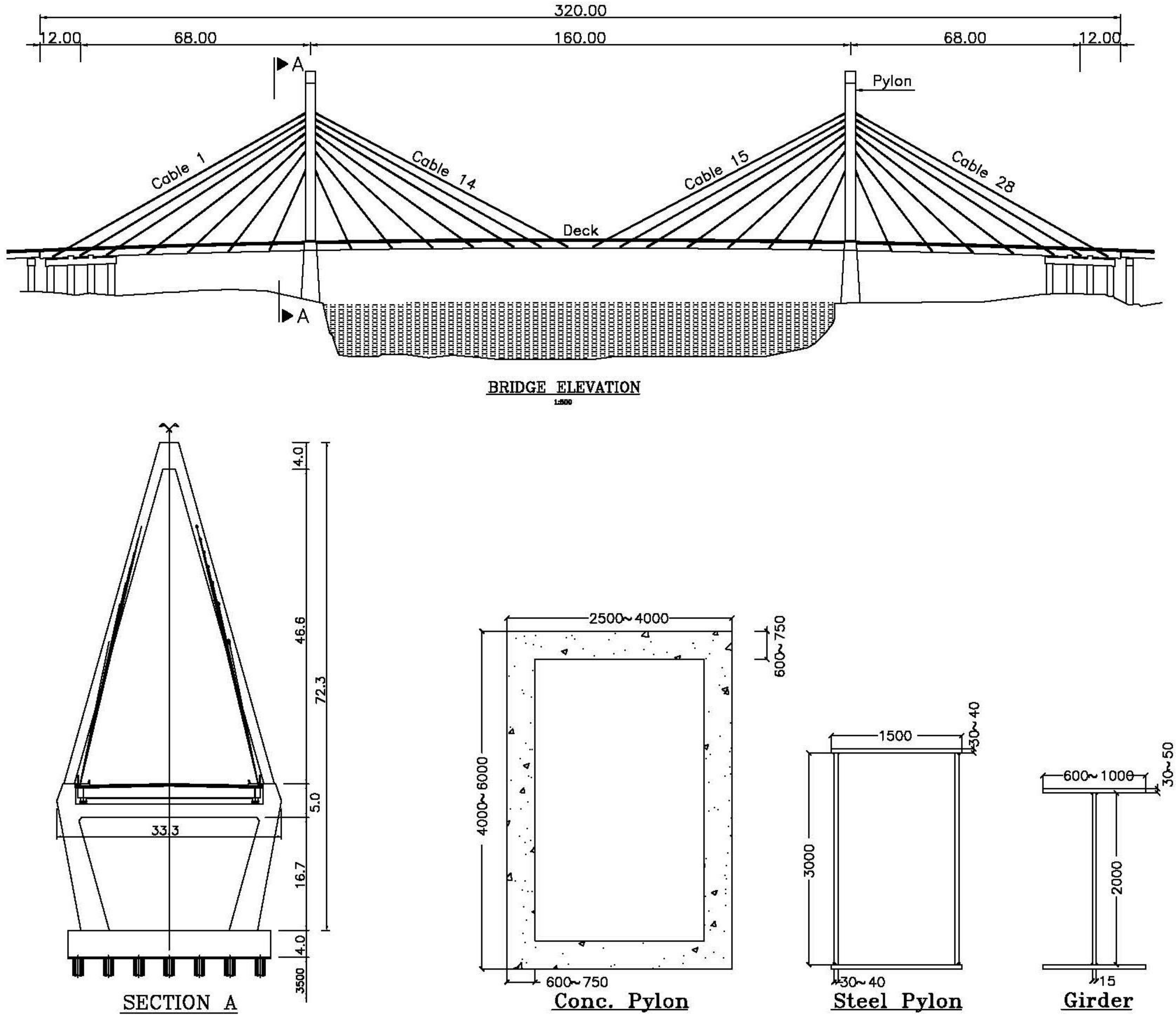

Fig. 2. Elements and sections of bridge.

Table 2. Initial tensile values of cables under dead loads.

Cable No.	Area (cm2)	Tension model 1 (ton)	Tension model 2 (ton)	Differen ce (%)	Minimum tension (0.2AFu)
1	72	382	274	-28	273.6
2	57	301	219	-27	216.6
3	57	301	461	53	216.6
4	57	361	484	34	216.6
5	51	325	406	25	193.8
6	45	282	237	-16	171
7	37.5	263	209	-21	142.5
8	37.5	269	186	-31	142.5
9	45	291	422	45	171
10	51	335	288	-14	193.8
11	57	379	288	-24	216.6
12	57	380	211	-44	216.6
13	51	291	349	20	193.8
14	51	291	485	67	193.8
15	51	291	429	47	194
16	51	291	365	25	194
17	57	380	207	-46	217
18	57	379	256	-32	217
19	51	335	337	1	194
20	45	291	373	28	171
21	37.5	269	254	-6	143
22	37.5	263	223	-15	143
23	45	282	258	-9	171
24	51	325	249	-23	194
25	57	361	612	70	217
26	57	301	342	14	217
27	57	301	223	-26	217
28	72	382	272	-29	274

Figs. 3 and 4 show the bridge deformation under the initial cable tension force plus the permanent dead load. As can be seen, differences in the initial tension changed the initial conditions of the structure in terms of deformation. One crucial factor for regulating the amount of cable tension is the initial deformation of the bridge.

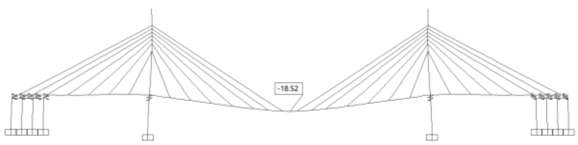

Fig. 3. Vertical deformation of bridge under initial cable tension and dead loads in mode1 (cm).

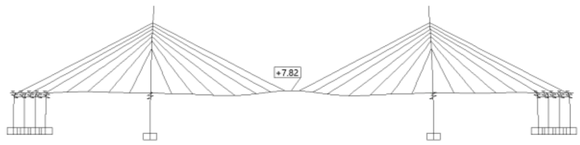

Fig. 4. Vertical deformation of bridge under initial cable tension and dead loads in mode2 (cm)

Figs. 5 and 6 show the moment and Figs. 7 and 8 show the axial force of the bridge for both cases. For cases at the mid-span section, the moment of the deck girder changed 452% and reached 60% plastic capacity of the section under dead loads.

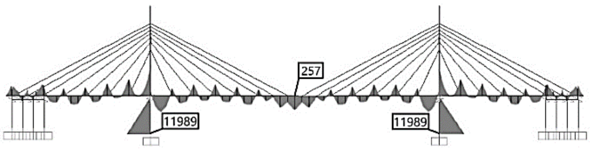

Fig. 5. Bending moment of girders and bridge pylons under initial cable tension and dead loads in model 1 (ton.m).

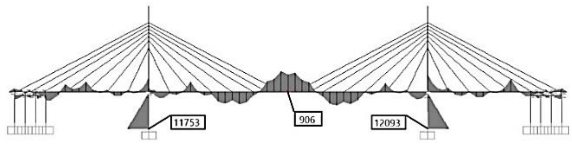

Fig. 6. Bending moment of girders and bridge pylons under initial cable tension and dead loads in model 2 (ton.m).

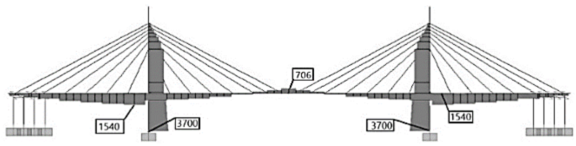

Fig. 7. Axial force of girders and bridge pylons under initial cable tension and dead loads in model 1 (ton.m).

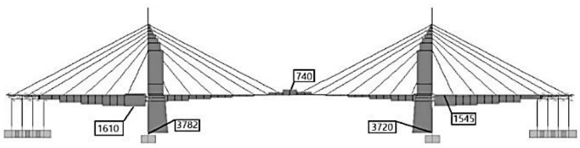

Fig. 8. Axial force of girders and bridge pylons under initial cable tension and dead loads in model 2 (ton.m).

The changes in the initial tension of the cables changed the amount and distribution of the bending moment and the axial force of the bridge deck and pylons. Usually, bridge designers consider cables to be like model 1 in order to achieve an optimal design for tensile distribution to minimize the deck flexural moments. In the next section, the effect of the cable tension adjustment method in both models on the overall behavior of the structure (strength and stiffness) is presented.

3.2. Strength and Stiffness of Bridge

Modal analysis was performed and the periods of the dominant modes of the two models were compared. The initial stiffness of the bridge due to initial tensile changes in the cables was investigated. In this analysis, the stiffness of the bridge was calculated after applying cable tension and dead loads to the structure. Figs. 9 and 10 show the two dominant modes and their periods in both models.

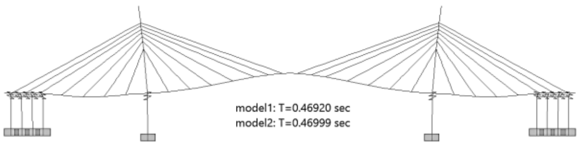

Fig. 9. First modal shape and its period of rotation in models with different initial cable tensions.

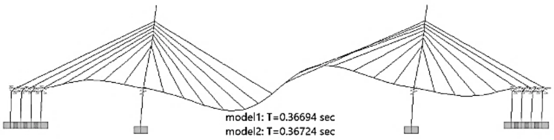

Fig. 10. Second modal shape and its period of rotation in models with different initial cable tensions.

As seen, although the initial cable tension forces of the two models were different, their modal periods and modal shapes were similar. It can be inferred that changing the initial cable force did not affect the overall stiffness.

To compare the stiffness and bridge capacity, nonlinear static analysis under gravitation loading, called pushdown analysis, was done for four loading patterns. The structure was loaded until it reached its collapse mechanism. Fig. 11 shows the pushdown of the capacity curve.

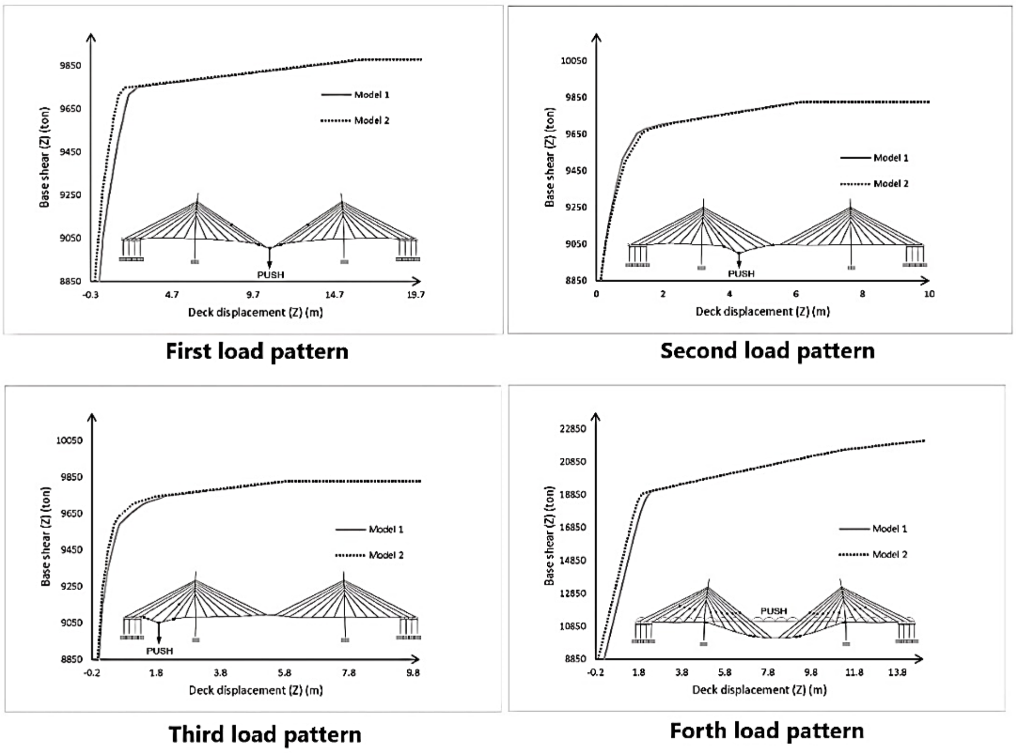

Fig. 11. Second modal shape and its period of rotation in models with different initial cable tensions.

In the first three analyses, they applied concentrated loads at three points. In the fourth analysis, the load was applied uniformly and extensively to the structure. As can be seen, the change in the initial tensile values of the cables in the two models changed the starting point of the diagram and the initial displacement of the study point. However, the initial slopes of the graphs in all four cases for the first and second models were approximately equal. This indicated that the elastic stiffness of the structure was fixed in both design models for cable tension. The hinge sequence mechanisms were different for both models, but their failure models (collapse mechanism) were similar. This means that, although both models had different stiffnesses after the initiation of plastic hinges, they had similar ultimate capacities and failure modes. In other words, the stiffness of the structure after undergoing plastic deformation differed between the two models. However, since there were no alterations in the final mechanism type and member resistance in either model, the final strength of the structure was the same in both models.

3.3. Nonlinear moving load concept

The nonlinear time-history under moving loads was considered when investigating the shakedown limit and alternating plasticity. To simulate the transient moving loads, several sequences of concentrated loads with a triangle time-series function were applied with a shift time phase. For example, consider a transient moving load in time-history analysis. Several point loads were equally spaced on the bridge girder and applied with a triangle time-history. When the load applied, to set it properly, the transient moving load was obtained. Fig. 12 shows the pattern of applying the three loads to the structure.

Because the present study investigated shakedown and alternating plasticity, the size of the point load was selected as required and was larger than normal traffic loads. The effect of the vehicle axle distances, and their number can be examined in another study, although such effects are not expected to change the overall behavior of the structure.

3.3.1. Vehicle passing speed

Vehicle speed can influence structural behavior. Different vehicle speeds can be selected and adjusted as shown in Fig. 12, The vehicle speed is equal to $V = 2\Delta L/\Delta t$. In this study, a constant speed (110 km/h) was considered in both models.

The effect of changes in cable tension on the shakedown limit load caused by the passage of moving loads in a cable-stayed-bridge has been investigated. The shakedown limit loads can be calculated by theoretical methods that consider all possible collapse mechanisms and the minimum loads obtained were considered as the shakedown loads. For this case, there were several complicated failure mechanisms so the theoretical method could not be used. Thus, the shakedown limit load was calculated by increasing the moving loads and assessing the deformation and plasticity mechanism.

To calculate the shakedown load limit for the cable-stayed bridge, a code was developed that automates the successive analysis process. In the first step, the required inputs are collected, including the number of load crossings, load speed, and initial amount of applied load.

The code then automatically generates the load patterns necessary for the nonlinear dynamic analysis under moving load, as described in section 3.4. Nonlinear time history analysis is performed to obtain the behavior of the structure after passing multiple loads. The deformation rate of the control point, located at the mid-span of the bridge, is used to determine whether the amount of applied load exceeded the shakedown load limit or not. To facilitate the calculation process, a flowchart of the code is presented in Fig. 13.

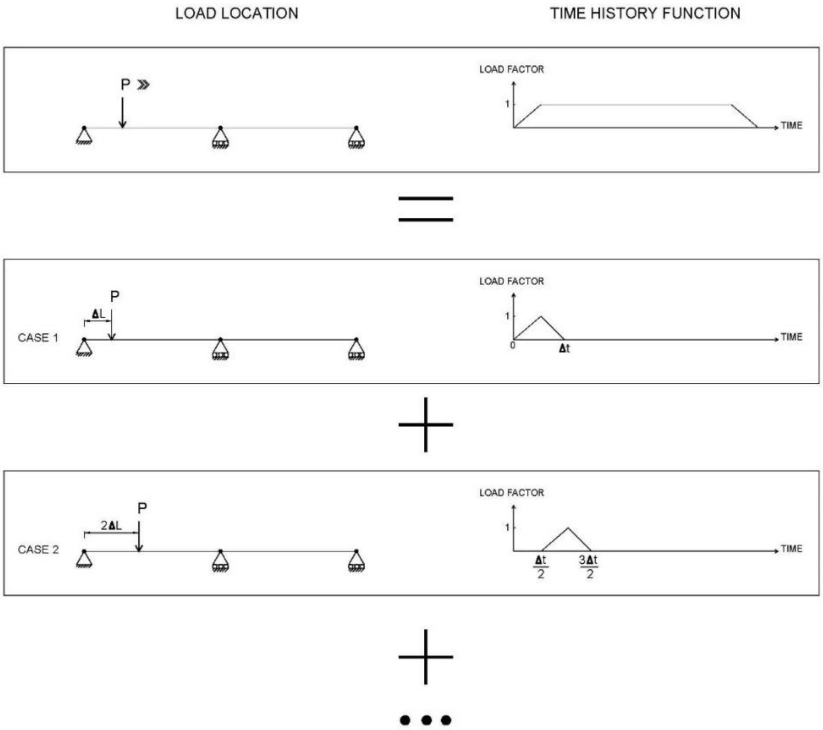

Fig. 12. Decomposition of moving loads into several concentrated loads at specified intervals3.4. Calculating the shakedown limit load.

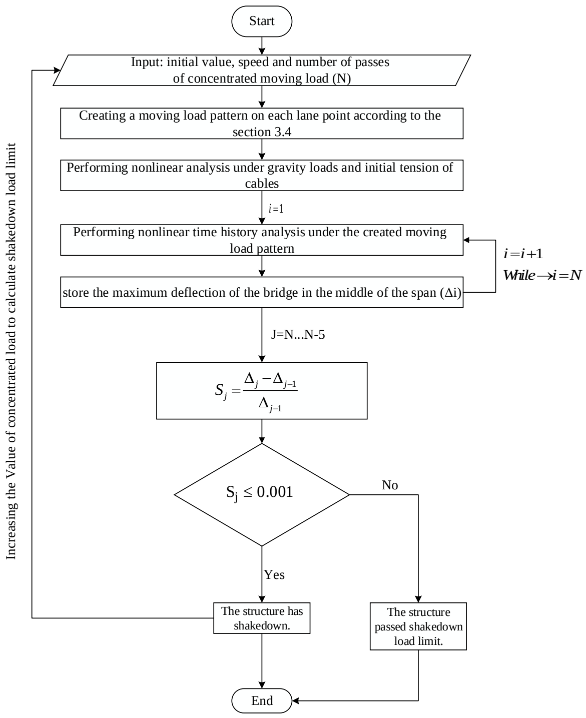

Fig. 13. Flowchart illustrating the automated code developed for calculating the shakedown load limit in cable-stayed bridges.

The results of calculating the Shakedown limit load on two models with different initial tension states of the cables are presented. In both models, despite the change in the initial tension of the cables and the initial moments in the bridge, a two-dimensional analysis shows that the Shakedown load limit was approximately 435 tons during the passage of a concentrated load with a speed of 30 meters per second (Fig. 16, Fig. 17). This indicates that during several cycles of passing a load less than this value on the bridge, the amount of displacements converges to a constant value. However, if a larger load passes over the bridge, the amount of displacements increases with each load cycle compared to the previous cycle, and the structure undergoes progressive deformations. Fig. 14 illustrates the mid-span deflection of the bridge under the passage of 19 times concentrated loads of 420 tons alternately, showing that the amount of displacements has converged to a constant value after several load cycles, and the load passed was less than the Shakedown limit load. Similarly, Fig. 15 displays the amount of bridge deflection under the passage of a concentrated load of 480 tons, indicating that the amount of displacements is increasing, and the structure has exceeded the Shakedown limit load.

Figs. 16 and 17 illustrate the accurate results obtained by the developed code for calculating the Shakedown limit load. The load passing over the bridge was increased by 5 tons, and it passed 100 times during the analysis. The horizontal axis of these figures shows the number of load cycles passing over the bridge, while the vertical axis represents the maximum mid-span deflection. It is observed that the structure was able to resist the assumed error in increasing the deformation under the intermittent passage of the load until it reached the Shakedown limit load of 435 tons. However, the deformations of the structure increased significantly after each load cycle passing beyond this limit.

As can be seen in both models, deformation of the mid-span opening was constant under a load of 435 tons and the bridge experienced shakedown. However, at 440 tons, both models underwent incremental collapse and the deformation of the mid-span opening increased.

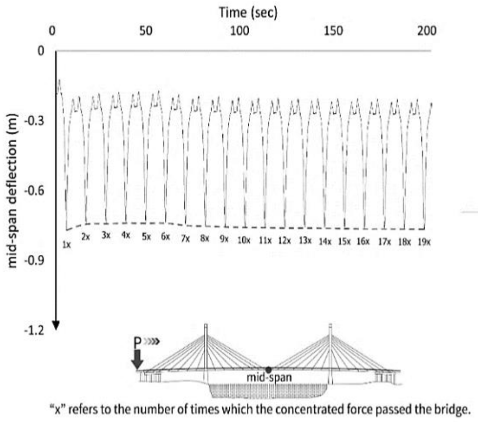

Fig. 14. Mid-span deflection caused by intermittent passage of 420 ton over the bridge in Model1.

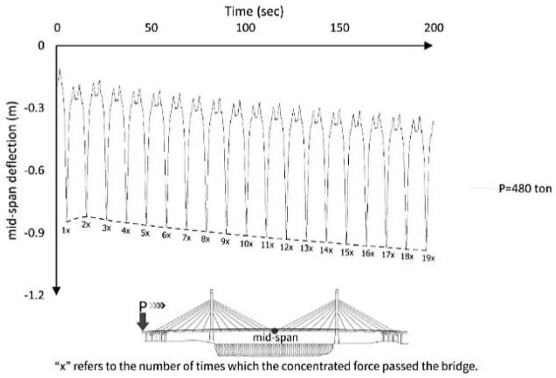

Fig. 15. Mid-span deflection caused by intermittent passage of 480 ton over the bridge in Model1.

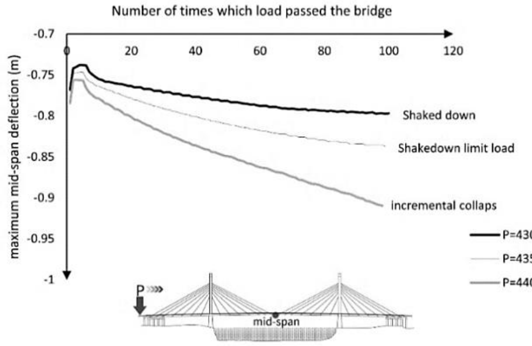

Fig. 16. Maximum mid-span deflection caused by intermittent passage of load over the bridge in Model 1.

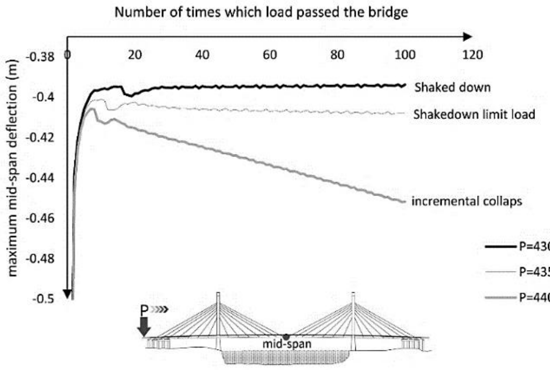

Fig. 17. Maximum mid-span deflection caused by intermittent passage of load over the bridge in Model 2.

Consequently, despite variations in the initial tension of the cables and the initial moments within the two bridge models, the Shakedown load limit remains constant at 435 tons for both structures. It should be noted that in the 2D modeling, only half of the bridge was considered. As a result, the shakedown limit for the bridge was calculated to be 860 tons, which is 2.5 times higher than the maximum live load design. This indicates that the safety factors against the shakedown limit in the design code are appropriate.

3.5. Discussion of Results Obtained for Alternating Plasticity

The minimum moment of M_i^{min}, maximum moment of M_i^{max}, and yield moment of $M_y^{k(i)}$ can be caused by dead and moving loads at any point on the deck. If relation (M_i^{max} - M_i^{min}) > $2M_y^{k(i)}$ is true at the shakedown limit load, it indicates that the structure has alternating plasticity for loads of less than the shakedown load and there will be a probability of failure with low-cycle fatigue in the structure.

The maximum and minimum elastic moment due to the passage of alternating loads at all points were calculated to determine the effect of a change in cable tension in both models for different initial cable tensions. Figs. 18 and 19 show the elastic bending moment diagrams for the bridge deck caused by a moving load, dead load, and initial tension of the cables in both models. The difference between the maximum and minimum elastic moments at four points is specified.

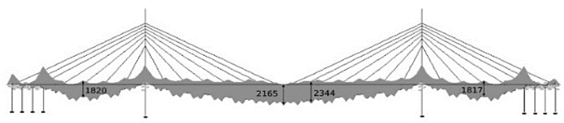

Fig. 18. Elastic bending moment of bridge deck under moving load, dead load, and initial cable tension in model 1.

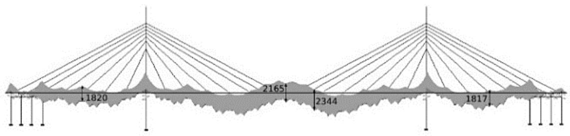

Fig. 19. Elastic bending moment of bridge deck under moving load, dead load, and initial cable tension in model 2.

The observed results demonstrate that while the maximum and minimum moments at various points in the bridge differed between models, the difference between them remained constant. This can be attributed to the fact that the variation between maximum and minimum moments at any point was solely due to the live load, which was identical in both models. This difference was less than $2M_y^{k(i)}$ at any point; therefore, the bridge did not experience change caused by alternating plasticity and failure due to low-cycle fatigue. Because the moments gained with the initial tension of the cables was self-balancing, their initial tension did not change the behavior of the structure under alternating plasticity.

4. A new method for cable-stayed bridge construction phases

As mentioned in the research history, different methods such as backward and forward analyses during construction are used to achieve the assumed initial cable tensions in the design of cable-stayed bridges. However, due to the presence of uncertainty parameters such as construction errors, creep and shrinkage effects, temperature loads, etc., cable tensions and the initial shape of the bridge are different from the initial design assumptions. Moreover, the presence of uncertainty effects leads to the fact that if the initial cable tensions are adjusted with the values obtained from the available during-construction analyses, the deck segments do not match each other in the free cantilever methods, and the last segment, which is the key piece, undergoes deformation and initial stresses due to displacement.

According to the results of this research, it was observed that changes in initial cable forces compared to the initial design assumptions will not affect the ultimate strength, stiffness, and shakedown limit loads of cable-stayed bridges. Therefore, in this section, we present a method that, unlike the available construction methods, does not require readjust cable tensions at the end of the bridge construction. In this method, after constructing the bridge components according to their desired geometric shape, the last cable connected to the deck is tightened to the extent that the installed end point of the deck matches the expected geometric profile of the bridge. In this method, the final geometric shape of the bridge will be uniform with the expected profile to a very desirable extent (with less than the permissible deformation error), and there will be no connection problem for the key piece. Furthermore, there is no need to readjust cable tensions or apply undesirable effects on the structure.

The traditional construction method for cable-stayed bridges involves the following steps:

1. Determining the initial tension force of the cables based on the initial design assumptions.

2. Fabricating the bridge deck components and installing them.

3. Tensioning the cables after each deck segment is installed and readjusting the tension of all installed cables based on on-site analysis.

4. Applying test loads to investigate the behavior of the structure under real conditions.

5. Readjusting the cable tension to achieve the design forces for all cables.

However, a new proposed method for cable-stayed bridge construction involves the following steps:

1. Fabricating the bridge deck components and installing them according to the desired geometric profile.

2. After installing each deck segment, the final cable connected to that segment is tensioned such that the installed end of the deck sits on the expected geometric profile.

3. Continuing this process for all deck segments.

4. After all deck and cable installation is complete, test loads are applied to the structure.

This new method eliminates the problems of key segment connection and cable tension readjustment at the end of bridge construction and ensures that the final geometry of the bridge matches the desired profile to a desirable degree. Moreover, this method does not require on-site analysis and cable tension readjustment. The comprehensive analysis of the proposed method, considering various structural models, is detailed in a separate article [51].

5. Conclusions

The present study examined the effect of changes in the tensile strength of cables on the stiffness, strength, and shakedown limit load in cable-stayed bridges. Two-dimensional analytical models were prepared from a case study of a cable-stayed bridge and analyzed static nonlinear gravitational analysis and dynamic nonlinear analysis under moving loads and the results were adapted to the theoretical solution. Based on the results of the study and considering existing hypotheses and limitations, the most important results obtained from this research are as follows:

- In accordance with the basic theorems of plasticity analysis and the behavior of the cables within the allowable design limit, changes in cable tension due to self-balancing effects did not affect the elastic stiffness and final strength of the structure.
- Theoretical analysis based on the shakedown theorems showed that the changes in cable tension had no effect on the shakedown limit load and did not reduce or increase that, causing incremental collapse. These did not affect the behavior of the structure under alternating plasticity and low cycle fatigue.
- In case study model, when the cable tension changed from -46% to +70%, it was

observed that the elastic stiffness and final strength of the structure did not change. The shakedown limit load and alternating plasticity of the structure also did not change. These findings have implications for engineering design, maintenance, and safety considerations, urging researchers and practitioners to explore the underlying mechanisms that enable such stability.

- In accordance with the theory, the existence of any self-balancing effects in the structure, including manufacturing effects, jack force error, temperature changes, creep, and shrinkage of concrete, etc., had no effect on the elastic stiffness, final strength, shakedown limit load, alternating plasticity, or structural safety.
- Because of the self-balancing force of the cables and changes in the cables, it was not necessary to readjust the force of the cables after execution or after operation to achieve the forces of the original design of the bridge. Based on this result a novel construction method for cable-stayed bridges presented that adjusting cable tension to the design value is not necessary during both construction and operation.
- According to the results obtained from the analytical analysis, it is expected that the obtained results can be cited for all types of cable-stayed bridges.

In summary, this research underscores the stability and resilience of cable-stayed bridges, even when faced with tension variations. These findings have practical implications for bridge design and construction practices.

In future studies, the effect of accurate modeling of truck wheel axes loads and the effect of different moving load speeds over the bridge on the final resistance and shakedown limit load will be considered.

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Conflicts of interest

There are no conflicts of interest to declare.

Authors contribution statement

The primary author, H. Dehghani, performed the modeling and analysis, while the supervisor, E. Dehghani, offered guidance and the initial study idea. M. Sharifi and S.R. Hoseini Vaez reviewed the manuscript.

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