

PLANNING THE UPPER STRUCTURE OF A STEEL ARCH BRIDGE WITH 3D MODEL SIMULATION USING SAP2000 SOFTWARE

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Abstract

Steel arch bridges are one of the structural bridge types widely used due to their ability to transfer loads primarily through compressive forces, making them efficient for medium- to long-span applications. This study aims to design the superstructure of a steel arch bridge using three-dimensional modeling in SAP2000 and to evaluate its structural performance under various loading conditions in accordance with SNI and Load and Resistance Factor Design (LRFD) provisions. The research employs a quantitative approach based on numerical analysis using the Finite Element Method (FEM), consisting of several stages, including a literature review, collection of geometric and material data, structural modeling, load application, and analysis of the results. The structural model represents a bridge with a span of 100 m, a width of 9.4 m, and a height of 18.5 m, with variations in hanger configurations. The analysis results indicate that the steel arch bridge structure demonstrates good performance in resisting traffic, wind, and seismic loads, with a maximum deflection of 0.103, which remains within the allowable deflection limit for bridge loading specified in SNI 1725:2016. Furthermore, the axial forces, shear forces, and bending moments in all primary structural members satisfy the nominal capacity requirements in accordance with SNI 1729:2020. These findings demonstrate that the network-type hanger configuration with a 5° inclination angle provides the optimal performance, achieving the highest structural efficiency ratio compared with the Nielsen-type and vertical hanger configurations.

Keywords:

Finite element method; hanger configuration; numerical analysis; SAP2000; steel arch bridge

Abstrak

Jembatan pelengkung baja merupakan salah satu tipe struktur yang banyak diaplikasikan karena kemampuannya menyalurkan beban terutama melalui gaya tekan, sehingga efisien untuk bentang menengah hingga panjang. Penelitian ini bertujuan merencanakan struktur atas jembatan baja pelengkung dengan pemodelan tiga dimensi menggunakan perangkat lunak SAP2000 serta menganalisis kinerja struktur terhadap beban sesuai ketentuan SNI dan LRFD. Metode penelitian yang digunakan adalah pendekatan kuantitatif berbasis analisis numerik dengan *finite element method* (FEM), melalui tahapan studi pustaka, pengumpulan data geometri dan material, pemodelan struktur, pembebanan, hingga analisis hasil. Pemodelan dilakukan pada jembatan dengan bentang 100 m, lebar 9,4 m, dan tinggi 18,5 m dengan variasi konfigurasi hanger. Hasil analisis menunjukkan bahwa struktur jembatan pelengkung baja memiliki kinerja yang baik dalam menahan beban lalu lintas, angin, dan gempa dengan lendutan maksimum sebesar 0,103 m, masih berada dalam batas izin pembebanan pada jembatan sesuai SNI 1725:2016. Selain itu, gaya aksial, geser, dan momen pada elemen struktur utama semuanya memenuhi persyaratan kapasitas nominal sesuai SNI 1729:2020. Temuan ini menegaskan bahwa konfigurasi hanger tipe network dengan sudut 5° menghasilkan performa optimal dengan rasio efisiensi struktur tertinggi dibandingkan dengan tipe nielsen dan tipe tegak lurus.

Kata Kunci:

Analisis numerik; finite element method; jembatan baja pelengkung; konfigurasi hanger; SAP2000

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1. INTRODUCTION

Bridges are a strategic piece of infrastructure that plays a role in supporting connectivity between regions, community mobility and distribution of goods. The increase in traffic volume and vehicle loads due to developments in construction technology requires bridge planning not only to have aspects of strength and safety, but also structural efficiency, sustainability, and integration with architectural and environmental aspects (Kim & Lee, 2022; Zhang et al., 2021). Therefore, the selection of a bridge structural system is an important factor in ensuring the performance of the bridge structure over a long period of time and the reliability of the transportation infrastructure. One type of bridge that has been widely developed for medium to long spans is the steel arch bridge. Bridges with this system utilize dominant compressive forces in the arch elements which are distributed to the supports and foundations, thereby reducing bending moments and increasing material efficiency compared to conventional girder systems (Batista et al., 2021; Chen et al., 2020). Apart from that, steel arch bridges have advantages in terms of aesthetics and flexibility in geometric design, so they are often applied in urban areas and main river crossings.

The development of structural analysis technology based on the finite element method (FEM) has encouraged an increase in the accuracy of steel arch bridge planning. Structural analysis software such as SAP2000 allows 3D modeling to simplify and evaluate the response of structures to various types of loads, including dead loads, live traffic loads, wind loads, and earthquake loads (Computers and Structures Inc., 2022; Wang et al., 2023). Several recent studies show that the geometric configuration of arches and hanger systems has a significant influence on internal force distribution, deformation and bridge stability. J. Liu et al. (2020) stated that varying the angle and distance of the hanger can optimize the distribution of tensile and compressive forces on the arch bridge system.

Based on a review of previous studies, there is still considerable potential for further development in the design of steel arch bridges, particularly with regard to the hanger configurations and bridge cross-sectional profiles employed. Previous studies have primarily focused on a single type of hanger configuration, while comparative studies examining network, Nielsen, and vertical hanger configurations remain limited. Therefore, this study focuses on a comparative structural analysis of different hanger configurations in steel arch bridges using three-dimensional modeling to evaluate the structural performance of the bridge based on applicable loading standards. The results of this study are expected to provide practical contributions to the development and application of network, Nielsen, and vertical hanger configurations in steel arch bridge design.

2. LITERATURE REVIEW

2.1. Steel arch bridge

A steel arch bridge is a bridge system that works by utilizing the dominant axial compressive force in the arch element. Based on SNI 1725:2016, bridges are classified into several types: type A, B, and C. Type A bridges have a width of ≥ 9 m, type B bridges have a width of 7-9 m, and type C bridges have a width of < 7 m. The load from the vehicle floor is distributed through a hanger system which is then distributed to the arch, then forwarded to the placement and foundation. This system is considered efficient because it is able to reduce bending moment in the main structure and increase the performance of axial compressive forces (He et al., 2020). According to Ghasemi & Nowak (2021) steel arch bridges have a high strength to weight ratio and good bridge structural performance for medium to long spans. Apart from the structural aspect, the arch shape also provides high aesthetic value, so it is often used on bridges in urban areas and as iconic bridges.

2.2. Classification of steel arch bridge systems

Based on the position of the vehicle floor relative to the arch, steel arch bridges can be classified into deck arch, through arch, and half-through arch. Each system has different characteristics of force distribution and structural response. Bridges with a through arc system are generally used for long spans because they provide much better structural efficiency and have a free space area under the bridge (Zhu et al., 2021). Other research by Al-Hussein et al. (2022) shows that the choice of arch system type greatly influences the deviation of the bridge structure and internal forces on the bridge, especially when analyzed using dynamic loading.

2.3. Hanger systems and their characteristics

A hanger is a tensile element that functions to distribute the load from the floor girder to the arch. There are several types of hanger systems, namely vertical hangers, inclined hangers (nielsen), and network hangers. This difference in hanger configuration affects the stiffness of the bridge and the distribution of forces in the bridge structure (Schlaich & Scheef, 2020). Other research by Deng et al. (2022) shows that the use of network hangers can reduce the maximum force on the arch and increase structural redundancy. Meanwhile Huang et al. (2023) stated that closer distances between hangers tend to produce a more even force distribution, but have an impact on increasing construction costs.

2.4. Arch geometry and its influence on structural performance

The geometry of steel arches, including the span to height ratio and the shape of the arch curve, has a direct influence on the compressive axial forces and bending moments that occur. An arch with a geometry close to a funicular shape will produce a more optimal force distribution (Qin et al., 2021). Another study by Torres et al. (2024) shows that optimizing arch geometry using a numerical approach can reduce the weight of the structure by up to 10-15% without reducing the bearing capacity. This proves the importance of evaluating the shape of the arch at the planning stage.

2.5. Loading of steel arch bridges

The loading of a steel arch bridge must consider the dead load, live traffic load, wind load and earthquake load in accordance with the standards applicable in Indonesia. Modern traffic loads tend to increase as vehicle volume and weight grow, so load analysis becomes increasingly complex (Nowak & Collins, 2020). According to Siringoringo & Mulyono (2022) a combination of dynamic loads that are not properly calculated can cause a mismatch between the behavior of the design result of the structure and the actual conditions in the field.

2.6. Analysis of steel arch bridges

Dynamic analysis is needed to evaluate the response of bridge structures to earthquake loads, wind and moving vehicles. Steel arch bridges are sensitive to vibration due to the slender nature of the arch and hanger elements (Casas & Moughty, 2021). Other research by Ren et al. (2023) shows that the dynamic response of steel arch bridges is greatly influenced by hanger stiffness and placement conditions, therefore, dynamic analysis is an important aspect in planning medium to long span bridges.

2.7. Finite element modeling of steel arch bridges

The finite element method is a numerical approach commonly used in bridge structural analysis. 3D modeling allows more realistic calculation of internal forces, deformations and vibration modes (Kwon & Kim, 2020). According to Z. Liu et al. (2024) finite element modeling by considering material and geometric nonlinearity is able to provide analysis results that are closer to actual conditions, especially for steel arch bridges with complex structural configurations.

2.8. SAP2000 software

SAP2000 is a structural analysis software that is widely used in steel bridge planning. This software supports static, dynamic analysis and 3D structural modeling with various loading conditions (Computers and Structures Inc., 2022). Other research by Ramadhan et al. (2025) shows that the use of SAP2000 in the analysis of steel arch bridges provides planning time efficiency as well as analysis results that are consistent with theoretical and experimental approaches.

2.9. Previous research and research positions

Previous research has discussed the performance of steel arch bridges from the aspects of geometry, hanger system and dynamic loading. For example He et al. (2020) and Zhu et al. (2021) investigated the basic behavior of steel arch bridges. Deng et al. (2022) and Huang et al. (2023) studied the influence of hanger configuration. Torres et al. (2024) and Z. Liu et al. (2024) emphasized the importance of advanced numerical modeling. However, there is still limited research that integrates variations in arch geometry and hanger configuration in 3D numerical models with a combination of static and dynamic loads. Therefore, this research focuses on the analysis of steel arch bridges based on 3D modeling using SAP2000 software to obtain a more comprehensive structural performance evaluation.

3. RESEARCH METHODS

This research uses quantitative research methods with a numerical analysis approach based on the finite element method (FEM). The numerical approach was chosen because it is able to model the behavior of steel arch bridge structures in 3D with a high degree of accuracy, especially in evaluating the structural response to a combination of static and dynamic loads (Huang et al., 2023; Z. Liu et al., 2024). The analysis was carried out using SAP2000 software, which is widely used in planning and evaluating modern steel bridge structures because of its capabilities in static, dynamic analysis and complex structural modeling (Computers and Structures Inc., 2023; Ramadhan et al., 2025). The research object is a through arch type steel arch bridge model, with medium to long spans. The selection of the through arch type is based on structural efficiency and its ability to optimally distribute the compressive axial force of the arch and the tensile force of the hanger (Torres et al., 2024; Zhou et al., 2022).

Variations in geometric parameters were determined based on recommendations from recent research regarding optimization of arch shapes and hanger systems (Deng et al., 2022; Ma et al., 2024). Bridge geometry data in the form of bridge length, arch height, vehicle floor width, distance and hanger angle. The materials for the bridge use structural steel with quality SS400 and concrete with F_c 30 MPa, material properties refer to bridge steel specifications commonly used in current research (Zhu et al., 2021). Loading on bridges refers to SNI 1725:2016 in the form of dead load, live traffic load, wind load, earthquake load. Load combinations are arranged for maximum limit conditions and service limit conditions, taking into account the dynamic response of the structure to traffic and earthquake loads (Ren et al., 2023; Siringoringo & Mulyono, 2022). This research was conducted through several interconnected stages. An explanation of the research methodology, presented as a diagram is provided below:

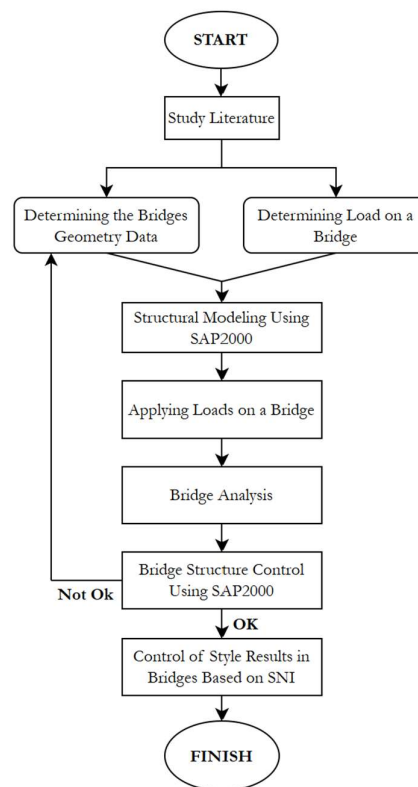


Figure 1. Flowchart

First, a literature study was carried out to obtain a theoretical basis and comparisons from previous research. To understand the development of steel arch bridge designs, internal force calculation methods, and structural parameters that need to be considered. Literature study includes a review of research Fajrin (2016) which focuses on planning steel arch bridges using SAP2000, research Anggraini (2019) which discusses design modification with a steel box through arch system, and research Bathista et al. (2019) which analyzes the optimization of hanger distances on arch bridges. The next stage is data collection. The data

used in this study include the geometric parameters of the bridge, namely a bridge span of 100 m, an arch height of 18.5 m, a traffic lane width of $3.5 \text{ m} \times 2$ lanes, and a sidewalk width of $2 \text{ m} \times 1$ lane. The data also include connection components, such as plates, bolts, nuts, washers, and welded connections; the assumed bridge support conditions consisting of a pin and roller; the mechanical properties of the materials, including SS400 structural steel and concrete with a compressive strength of 30 MPa ($f_c' = 30 \text{ MPa}$); and the design loading data, including dead loads, live loads, traffic loads, wind loads, and seismic loads. All data used in this study refer to the applicable design standards in Indonesia, namely the Indonesian National Standards (SNI).

After the data were collected, the structural model was developed using SAP2000. The 3D model comprised the main components of the bridge superstructure, including the main arch ribs, cross girders, roadway deck, and hanger cables. Three types of hanger configurations were considered: network, nielsen, and vertical configurations. All three hanger configurations employed the same inclination angle of 5° . The connections between the arch ribs, girders, and hangers were modeled using fork/clevis connections, while the support system consisted of pin and roller. Under loading conditions, the hangers were assumed to carry tensile forces only. The next stage involved structural analysis. The analysis was performed using the finite element method available in SAP2000 to determine the distribution of internal forces, support reactions, structural deformations, and stresses within the bridge structure. The analysis results were subsequently verified and validated by comparing the SAP2000 outputs with the applicable design provisions (SNI and LRFD) as well as the results of previous studies. This comparison also served as the basis for a gap analysis, which was conducted by comparing *das sollen* (the ideal conditions according to design standards and theoretical considerations) with *das sein* (the actual results obtained from the numerical simulation). The final stage involved drawing conclusions. The conclusions were derived from the numerical analysis results by comparing the structural performance of the different hanger configurations and determining the configuration that provided the most favorable structural performance.

4. RESULTS AND DISCUSSION

4.1. Structural modeling

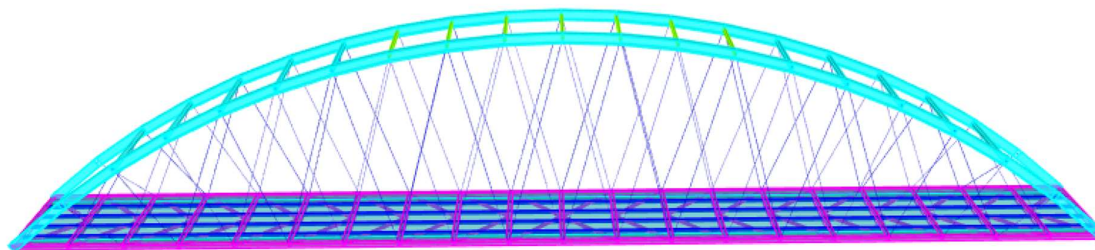


Figure 2. Bridge structure modeling

Modeling of the steel arch bridge structure was carried out with the help of SAP2000 software to provide an overview of the structure against the working loads. In the initial stage, the structure is modeled in 3D by considering the main arch elements, transverse girders, vehicle floor, and hanger cables that connect the girder to the arch. The load included dead load, which includes the own weight of the structure and pavement layers, as well as live loads due to vehicle traffic in accordance with applicable loading standards (Badan Standardisasi Nasional, 2016). The condition is in line with the basic principles of arch bridges, where most of the force received by the structure is resisted in the form of compressive action, not bending action (Anggraini, 2019; Fajrin, 2016). The maximum deflection that occurs is still far below the limit permitted by regulations, so it can be said that the steel arch bridge is able to withstand traffic loads safely (Triyoga, 2017).

The chosen bridge type is a steel arch bridge with cable tendons steel arch bridges demonstrate greater material efficiency compared to girder bridges. This is because more of the load is supported by the compressive force acting on the arch, so that the bending moment that occurs is relatively smaller (Setiawan, 2017) this efficiency ultimately has implications for the long term durability of the structure as well as the potential for saving maintenance costs (Mulyadi, 2016). The following are the bridge design specifications:

Table 1. Bridge design specifications

No	Bridge criteria	Information
1	Bridge type	Arch bridge with tendon cables
2	Bridge class	Type A
3	Bridge width	9.4 m
4	Bridge length	100 m
5	Bridge height	18.5 m
6	Road width	3.5 m x 2 m
7	Sidewalk width	2 m x 1 m
8	Connecting tool	Plates, bolts, nuts, washers and welded connections
9	Steel quality	SS400
10	Floor plate	Fc' 30 MPa
11	Bridge abutment	Hinge and Hinge

The use of profiles in abridge significantly influences the bridges structural performance.

The steel profile used in the bridge can be seen in table 2. Below:

Table 2. Specifications of the steel profile used

No	Function	Profile
1	Arch	Box 800 x 600 x 25 mm
2	Longitudinal girder	Box 400 x 400 x 20 mm
3	Transverse girder	Box 400 x 400 x 20 mm
4	Stringer	Box 200 x 200 x 10 mm
5	Wind bond	T 150 x 350 x 25 mm
6	Lateral strut	Box 300 x 300 x 20 mm Box 250 x 250 x 10 mm

The materials used in this bridge structure are as listed in table 3. Below:

Table 3. Material specifications used in bridge structures

Material	Specification	Value	Unit
Steel SS400	Yield strength (F_y)	245	MPa
	Ultimate strength (F_u)	400	MPa
	Modulus of elasticity (E)	200,000	MPa
	Density	7,850	Kg/m ³
Bolt - nut ASTM 325	Tensile stress (F_{nt})	620	MPa
	Shear stress (F_{nv})	372	MPa
	Weight of each bolt and nut	8.1 – 36	Gram
Sling (<i>Locked coil 1770</i>)	Yield strength (F_y)	1,350	MPa
	Ultimate strength (F_u)	1,770	MPa
	Modulus of elasticity (E)	200,000	MPa
	Density	7,850	Kg/m ³
Floor slab Fc' 30 MPa	Modulus of elasticity (E)	25,000	MPa
	Density	2,400	Kg/m ³
	Fc'	30	MPa

The following are the specifications of the connecting equipment used on the bridge, which can be seen in table 4.

Table 4. Material quality of bridge connecting equipment

Specification	Information
Bolts and nuts	ASTM A325
Ring	ASTM A325
Plates and bolts	Gusset plates 20 mm dan 15 mm

The selection of the hanger configuration was carried out through trial and error on a number of configuration models using SAP2000 program. Arch bridges receive dominant loads in the vertical direction

so they must consider the configuration of the hanging cables. The following is a selection of suspension configurations on the bridge, which can be seen in table 5.

4.2. Bridge loading

The loading of a steel arch bridge must consider the dead load, live traffic load, wind load and earthquake load in accordance with the standards applicable in Indonesia. The following is a table of loads on the bridge:

Table 5. Loading on bridges

No	Load type	Value	Unit
1	Centralized line load	49	kN/m ²
2	The load is distributed evenly on the vehicle floor plat	5.85	kN/m ²
3	Pedestrian load	5	kN/m ²
4	Super dead load on the vehicle floor plate	3.01	kN/m ²
5	Super dead load on the pavement	4.52	kN/m ²
6	Vehicle brake force load	3.43	kN

Table 6. Compressive wind load

Wind press bow				
AS	Z mm	Ews N/mm	P kN	Design requirements P > 4.4 4,4
1	3809	5.30	13.25	SAFE
2	7035	9.79	24.47	SAFE
3	9759	13.58	33.94	SAFE
4	12037	16.75	41.86	SAFE
5	13910	19.35	48.38	SAFE
6	15408	21.43	53.59	SAFE
7	16553	23.03	57.57	SAFE
8	17360	24.15	60.38	SAFE
9	17840	24.82	62.05	SAFE
10	18000	25.04	62.60	SAFE

Table 7. Suction wind load

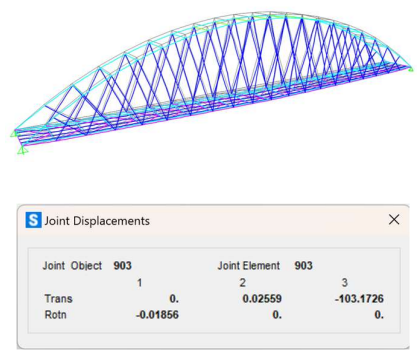
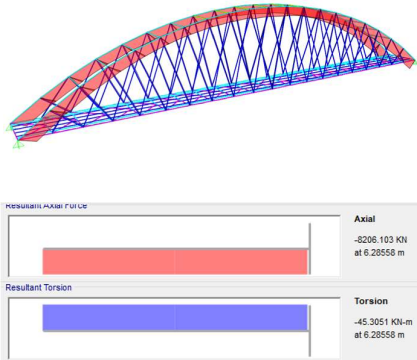
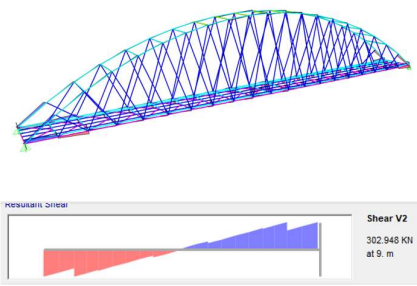
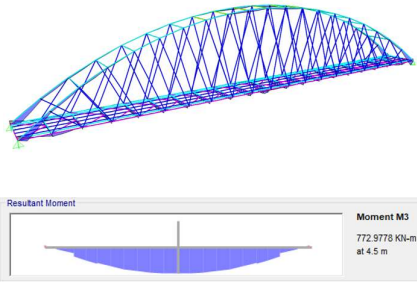
Bow suction wind				
AS	Z mm	Ews N/mm	P kN	Design requirements P > 2.2 2,2
1	3809	2.65	6.62	SAFE
2	7035	4.89	12.23	SAFE
3	9759	6.79	16.97	SAFE
4	12037	8.37	20.93	SAFE
5	13910	9.68	24.19	SAFE
6	15408	10.72	26.79	SAFE
7	16553	11.51	28.78	SAFE
8	17360	12.08	30.19	SAFE
9	17840	12.41	31.02	SAFE
10	18000	12.52	31.30	SAFE

4.3. Results of structural analysis

After all the modeling and loading is complete, the next step is to analyze the bridge using the SAP2000. The selection of the hanger configuration was carried out through trial and error on a number of configuration models using SAP2000 program. Arch bridges receive dominant loads in the vertical direction so they must consider the configuration of the hanging cables. The following is a selection of suspension configurations on the bridge.

4.3.1. Network hanger

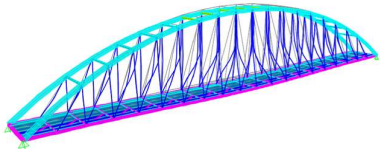
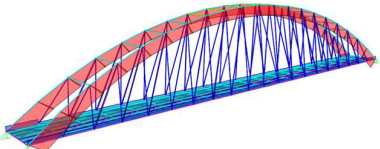
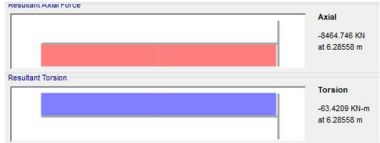
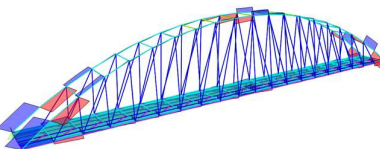
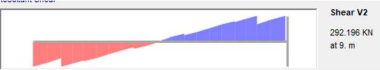
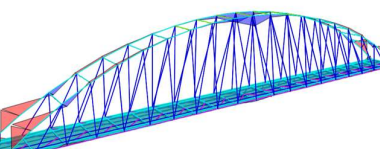
Table 8. Structural analysis results network hanger

No	Force results in SAP2000	Information
1		Node Displacement (Deflection) at half span obtained a maximum deflection of (U3theorycal) = 103.173 mm = 0.103 m this figure still meets $\Delta_{ijin} = \frac{L}{8} = 125\text{ mm}$ according to SNI 1725:2016.
2		Axial Force obtained maximum value = 8,206.10 KN (Compression)
3		Shear Y (Shear force) obtained a maximum value = 302.948 KN
4		Moment Z obtained the maximum value = 772.978 KN-m

Based on the results presented in the table above, it can be concluded that the bridge with the network-type hanger configuration exhibits a maximum deflection of 103.173 mm (0.103 m). The maximum axial force (nominal force) is 8,206.10 kN in compression, while the maximum shear force in the Y-direction is 302.948 kN. The maximum bending moment about the Z-axis is 772.978 kN·m.

4.3.2. Nielsen hanger

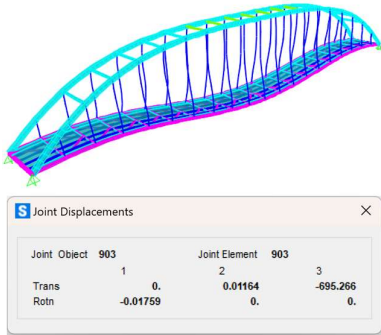
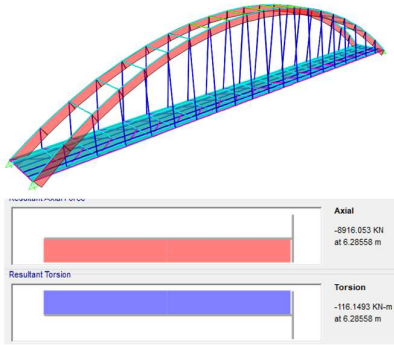
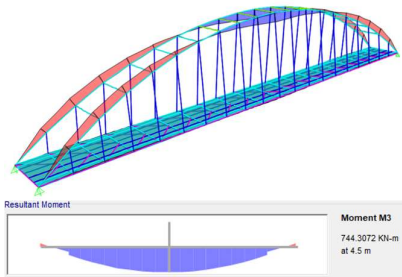
Table 9. Structural analysis results nielsen hanger

No	Force results in SAP2000	Information
1	 	Node Displacement (Deflection) at half span obtained a maximum deflection of (U3theorycal) = 195.570 mm = 0.195 m which exceeds the allowable deflection limit of $\Delta_{ijin} = \frac{L}{8} = 125 \text{ mm}$ according to SNI 1725:2016.
2	 	Axial Force obtained maximum value = 8,464.746 kN (Compression)
3	 	Shear Y (Shear force) obtained a maximum value = 292.196 kN
4	 	Moment Z obtained the maximum value = 756.173 kN-m

Based on the results presented in the table above, it can be concluded that the bridge with the network-type hanger configuration exhibits a maximum deflection of 195.570 mm (0.195 m). The maximum axial force (nominal force) is 8,464.746 kN in compression, while the maximum shear force in the Y-direction is 292.196 kN. The maximum bending moment about the Z-axis is 756.173 kN·m.

4.3.3. Vertical hanger

Table 10. Structural analysis results vertical hanger

No	Force results in SAP2000	Information
1		Node Displacement (Deflection) at half span obtained a maximum deflection of (U3theoritycal) = 695.266 mm = 0.695 m which exceeds the allowable deflection limit of $\Delta_{ijin} = \frac{L}{8} = 125 \text{ mm}$ according to SNI 1725:2016.
2		Axial Force obtained maximum value = 8,916.053 KN (Compression)
3		Shear Y (Shear force) obtained a maximum value = 297.931 KN
4		Moment Z obtained the maximum value = 744.07 KN-m

Based on the results presented in the table above, it can be concluded that the bridge with the network-type hanger configuration exhibits a maximum deflection of 695.266 mm (0.695 m). The maximum axial force (nominal force) is 8,916.053 kN in compression, while the maximum shear force in the Y-direction is 297.931 kN. The maximum bending moment about the Z-axis is 744.07 kN·m.

4.4. Control of bridge structures

Analysis was carried out on the bridge structure which consisted of tensile, compressive, shear and moment strength controls. When the nominal capacity is more than the ultimate force, it is said to meet the requirements. The nominal capacity is obtained from manual calculations referring to (SNI 1729:2020), and with the help of SAP2000 software the ultimate force is obtained as follows:

4.4.1. Network hanger

Table 11. Recapitulation of cross-sectional capacity and ultimate force network hanger

Internal forces (SNI 1729:2020)	Profile elements	Nominal capacity	Ultimate forces	Control
Compression (Pascal E3)	Box 800x600x25	14,557,855.97 N	8,369,130.46 N	(SAFE)
	Box 400x400x20	5,629,643.98 N	345,455.21 N	(SAFE)
	Box 300x300x20	3,888,109.81N	67,679.53 N	(SAFE)
	Box 250x200x10	1,530,033.88 N	17,265.46 N	(SAFE)
	Box 200x200x10	1,350,953.12 N	76,294.94 N	(SAFE)
	TI 150x350x25	601,146.90 N	21,816.62 N	(SAFE)
Tensile (Pascal D2)	Box 400x400x20	7,392,000 N	345,455.21 N	(SAFE)
	Box 200x200x10	1,921,500 N	17,265.46 N	(SAFE)
	Box 250x200x10	2,097,375 N	76,294.94 N	(SAFE)
	TI 150x350x25	2,618,437,5 N	218,816.62 N	(SAFE)
	Sling 70	6,005,468.54 N	606,505.8 N	(SAFE)
Moment (Pascal F7 and F9)	Box 800x600x25	4,113,703,125 N-mm	394,590,388 N-mm	(SAFE)
	Box 400x400x20	956,088,000 N-mm	772,977,806 N-mm	(SAFE)
	Box 300x300x20	519,498,000 N-mm	52,479,511 N-mm	(SAFE)
	Box 250x200x10	164,162,250 N-mm	129,670,959 N-mm	(SAFE)
	Box 200x200x10	119,511,000 N-mm	31,107,747.87 N-mm	(SAFE)
	TI 150x350x25	69,352,618.13N-mm	29,308,645.1 N-mm	(SAFE)
Shear (Pascal G4 and G3)	Box 800x600x25	5,292,000 N	59,040.86 N	(SAFE)
	Box 400x400x20	2,116,800 N	302,948.17 N	(SAFE)
	Box 300x300x20	793,800 N	139,104.95 N	(SAFE)
	Box 250x200x10	264,600 N	44,217.89 N	(SAFE)
	Box 200x200x10	264,600 N	12,880.6 N	(SAFE)
	TI 150x350x25	1,286,250 N	27,674.06 N	(SAFE)

Based on the results presented in the table above, it can be concluded that the maximum compressive force occurs in the Box $800 \times 600 \times 25$ section, with a value of 8,369,130.46 N. The maximum tensile force occurs in the Sling 70, with a value of 606,505.8 N. The maximum bending moment occurs in the Box $400 \times 400 \times 20$ section, with a value of 772,977,806 N-mm. Meanwhile, the maximum shear force occurs in the Box $400 \times 400 \times 20$ section, with a value of 302,948.17 N.

4.4.2. Nielsen hanger

Table 12. Recapitulation of cross-sectional capacity and ultimate force nielsen hanger

Internal forces (SNI 1729:2020)	Profile elements	Nominal capacity	Ultimate forces	Control
Compression (Pascal E3)	Box 800x600x25	14,557,855.97 N	8,464,901.04 N	(SAFE)
	Box 400x400x20	5,629,643.98 N	372,226.44 N	(SAFE)
	Box 300x300x20	3,888,109.81N	78,492.78 N	(SAFE)
	Box 250x200x10	1,530,033.88 N	23,363.22 N	(SAFE)
	Box 200x200x10	1,350,953.12 N	73,992.87 N	(SAFE)
	TI 150x350x25	601,146.90 N	18,281.08 N	(SAFE)
Tensile (Pascal D2)	Box 400x400x20	7,392,000 N	3,372,226.44 N	(SAFE)
	Box 200x200x10	1,921,500 N	23,363.22 N	(SAFE)
	Box 250x200x10	2,097,375 N	73,992.87 N	(SAFE)
	TI 150x350x25	2,618,437,5 N	18,281.08 N	(SAFE)

Internal forces (SNI 1729:2020)	Profile elements	Nominal capacity	Ultimate forces	Control
	Sling 70	6,005,468.54 N	995,632.01 N	(SAFE)
Moment (Pascal F7 and F9)	Box 800x600x25	4,113,703,125 N-mm	1,441,964,206 N-mm	(SAFE)
	Box 400x400x20	956,088,000 N-mm	756,172,904 N-mm	(SAFE)
	Box 300x300x20	519,498,000 N-mm	55,633,232 N-mm	(SAFE)
	Box 250x200x10	164,162,250 N-mm	111,129,817 N-mm	(SAFE)
	Box 200x200x10	119,511,000 N-mm	39,460,824.14 N-mm	(SAFE)
	TI 150x350x25	69,352,618.13N-mm	32,425,521 N-mm	(SAFE)
Shear (Pascal G4 and G3)	Box 800x600x25	5,292,000 N	364,202.15 N	(SAFE)
	Box 400x400x20	2,116,800 N	292,195.86 N	(SAFE)
	Box 300x300x20	793,800 N	147,028.97 N	(SAFE)
	Box 250x200x10	264,600 N	37,557.53 N	(SAFE)
	Box 200x200x10	264,600 N	14,250.71 N	(SAFE)
	TI 150x350x25	1,286,250 N	27,886.01 N	(SAFE)

Based on the results presented in the table above, it can be concluded that the maximum compressive force occurs in the Box $800 \times 600 \times 25$ section, with a value of 8,464,901.04 N. The maximum tensile force occurs in the Box $400 \times 400 \times 20$ section, with a value of 3,372,226.44 N. The maximum bending moment occurs in the Box $800 \times 600 \times 25$ section, with a value of 1,441,964,206 N-mm. Meanwhile, the maximum shear force occurs in the Box $800 \times 600 \times 25$ section, with a value of 364,202.15 N.

4.4.3 Vertical hanger

Table 13. Recapitulation of cross-sectional capacity and ultimate force vertical hanger

Internal forces (SNI 1729:2020)	Profile elements	Nominal capacity	Ultimate forces	Control
Compression (Pascal E3)	Box 800x600x25	14,557,855.97 N	8,917,885.85 N	(SAFE)
	Box 400x400x20	5,629,643.98 N	490,876.06 N	(SAFE)
	Box 300x300x20	3,888,109.81N	83,810.51 N	(SAFE)
	Box 250x200x10	1,530,033.88 N	19,064.92 N	(SAFE)
	Box 200x200x10	1,350,953.12 N	71,309.36 N	(SAFE)
	TI 150x350x25	601,146.90 N	13,147.86 N	(SAFE)
Tensile (Pascal D2)	Box 400x400x20	7,392,000 N	490,876.06 N	(SAFE)
	Box 200x200x10	1,921,500 N	19,064.92 N	(SAFE)
	Box 250x200x10	2,097,375 N	71,309.36 N	(SAFE)
	TI 150x350x25	2,618,437.5 N	13,147.86 N	(SAFE)
	Sling 70	6,005,468.54 N	711,411.69 N	(SAFE)
Moment (Pascal F7 and F9)	Box 800x600x25	4,113,703,125 N-mm	3,790,551,550 N-mm	(SAFE)
	Box 400x400x20	956,088,000 N-mm	744,307,223 N-mm	(SAFE)
	Box 300x300x20	519,498,000 N-mm	66,327,936 N-mm	(SAFE)
	Box 250x200x10	164,162,250 N-mm	79,732,095.12 N-mm	(SAFE)
	Box 200x200x10	119,511,000 N-mm	58,042,521.23 N-mm	(SAFE)
	TI 150x350x25	69,352,618.13N-mm	28,558,348.5 N-mm	(SAFE)
Shear (Pascal G4 and G3)	Box 800x600x25	5,292,000 N	412,085.33 N	(SAFE)
	Box 400x400x20	2,116,800 N	297,930.92 N	(SAFE)
	Box 300x300x20	793,800 N	179,103.04 N	(SAFE)
	Box 250x200x10	264,600 N	26,075.48 N	(SAFE)
	Box 200x200x10	264,600 N	13,632.32 N	(SAFE)
	TI 150x350x25	1,286,250 N	27,818.44 N	(SAFE)

Based on the results presented in the table above, it can be concluded that the maximum compressive force occurs in the Box $800 \times 600 \times 25$ section, with a value of 8,917,885.85 N. The maximum tensile force occurs in the Sling 70, with a value of 711,411.69 N. The maximum bending moment occurs in the Box $800 \times 600 \times 25$ section, with a value of 3,790,551,550 N-mm. Meanwhile, the maximum shear force occurs in the Box $800 \times 600 \times 25$ section, with a value of 412,085.33 N.

4.5. Combination control

Calculations are carried out in terms of style combinations based on SNI 1729:2020 Articles H1 and H3.

4.5.1. Network hanger

Table 14. Recapitulation of load combination check network hanger

Profile	Control (HI-1a)	Control (H3-6)
	$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0$	$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c} \right) \leq 1.0$
Box 800x600x25	$0.660 \leq 1.0$ (SAFE)	$0.682 \leq 1.0$ (SAFE)
Box 400x400x20	$0.383 \leq 1.0$ (SAFE)	$0.566 \leq 1.0$ (SAFE)
Box 300x300x20	$0.107 \leq 1.0$ (SAFE)	$0.294 \leq 1.0$ (SAFE)
Box 250x200x10	$0.713 \leq 1.0$ (SAFE)	$0.968 \leq 1.0$ (SAFE)
Box 200x200x10	$0.288 \leq 1.0$ (SAFE)	$0.365 \leq 1.0$ (SAFE)
T1 150x350x25	$0.384 \leq 1.0$ (SAFE)	$0.452 \leq 1.0$ (SAFE)

Based on the results presented in the table above, it can be concluded that the Box $250 \times 200 \times 10$ section has the highest load combination demand-to-capacity ratio, with a value of 0.968. In contrast, the Box $300 \times 300 \times 20$ section has the lowest load combination demand-to-capacity ratio, with a value of 0.107.

4.5.2. Nielsen hanger

Table 15. Recapitulation of load combination check nielsen hanger

Profile	Control (HI-1a)	Control (H3-6)
	$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0$	$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c} \right) \leq 1.0$
Box 800x600x25	$0.893 \leq 1.0$ (SAFE)	$1.001 \leq 1.0$ (NOT SAFE)
Box 400x400x20	$0.381 \leq 1.0$ (SAFE)	$0.558 \leq 1.0$ (SAFE)
Box 300x300x20	$0.115 \leq 1.0$ (SAFE)	$0.312 \leq 1.0$ (SAFE)
Box 250x200x10	$0.617 \leq 1.0$ (SAFE)	$0.834 \leq 1.0$ (SAFE)
Box 200x200x10	$0.348 \leq 1.0$ (SAFE)	$0.439 \leq 1.0$ (SAFE)
T1 150x350x25	$0.423 \leq 1.0$ (SAFE)	$0.496 \leq 1.0$ (SAFE)

Based on the results presented in the table above, it can be concluded that the Box $800 \times 600 \times 25$ section has the highest load combination demand-to-capacity ratio, with a value of 1.001. The Box $800 \times 600 \times 25$ section fails the safety check under load combination H3-6, with a demand-to-capacity ratio of 1.001. Since this value exceeds the allowable limit of 1.0, the structural member is considered unsafe under the specified load combination. In contrast, the Box $300 \times 300 \times 20$ section has the lowest load combination demand-to-capacity ratio, with a value of 0.115.

4.5.3. Vertical hanger

Table 16. Recapitulation of load combination check vertical hanger

Profile	Control (HI-1a)	Control (H3-6)
	$\frac{P_r}{P_c} + \frac{8}{9} \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}} \right) \leq 1.0$	$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c} \right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c} \right) \leq 1.0$
Box 800x600x25	$1.432 \leq 1.0$ (NOT SAFE)	$1.612 \leq 1.0$ (NOT SAFE)
Box 400x400x20	$0.397 \leq 1.0$ (SAFE)	$0.576 \leq 1.0$ (SAFE)
Box 300x300x20	$0.135 \leq 1.0$ (SAFE)	$0.375 \leq 1.0$ (SAFE)
Box 250x200x10	$0.444 \leq 1.0$ (SAFE)	$0.597 \leq 1.0$ (SAFE)
Box 200x200x10	$0.484 \leq 1.0$ (SAFE)	$0.590 \leq 1.0$ (SAFE)
T1 150x350x25	$0.371 \leq 1.0$ (SAFE)	$0.438 \leq 1.0$ (SAFE)

Based on the results presented in the table above, it can be concluded that the Box $800 \times 600 \times 25$ section has the highest load combination demand-to-capacity ratio, with a value of 1.612. The Box $800 \times 600 \times 25$ section fails the safety check under load combination H3-6 and H1-1a, with a demand-to-capacity

ratio of 1.612 and 1.432. Since this value exceeds the allowable limit of 1.0, the structural member is considered unsafe under the specified load combination. In contrast, the Box $300 \times 300 \times 20$ section has the lowest load combination demand-to-capacity ratio, with a value of 0.135.

5. CONCLUSION

5.1. Network hanger

The steel arch bridge with the network-type hanger configuration exhibits a maximum deflection of 103.173 mm (0.103 m), which satisfies the allowable deflection limit of $\Delta_{ijin} = \frac{L}{8} = 125 \text{ mm}$. The maximum axial force is 8,206.10 kN in compression, the maximum shear force is 302.948 kN, and the maximum bending moment is 772.978 kN·m. The maximum compressive force occurs in the Box $800 \times 600 \times 25$ section, with a value of 8,369,130.46 N. The maximum tensile force occurs in the Sling 70 section, with a value of 606,505.8 N. The maximum bending moment occurs in the Box $400 \times 400 \times 20$ section, with a value of 772,977,806 N·mm. Meanwhile, the maximum shear force occurs in the Box $400 \times 400 \times 20$ section, with a value of 302,948.17 N. Following the load combination checks, the highest demand-to-capacity ratio is found in the Box $250 \times 200 \times 10$ section under load combination H3-6, with a value of 0.968. In contrast, the lowest ratio is found in the Box $300 \times 300 \times 20$ section under load combination H1-1a, with a value of 0.107.

5.2. Neilsen hanger

The steel arch bridge with the Nielsen-type hanger configuration exhibits a maximum deflection of 195.570 mm (0.195 m), which exceeds the allowable deflection limit of $\Delta_{ijin} = \frac{L}{8} = 125 \text{ mm}$. The maximum axial force is 8,464.746 kN in compression, the maximum shear force is 292.196 kN, and the maximum bending moment is 756.173 kN·m. The maximum compressive force occurs in the Box $800 \times 600 \times 25$ section, with a value of 8,464,901.04 N. The maximum tensile force occurs in the Box $400 \times 400 \times 20$ section, with a value of 3,372,226.44 N. The maximum bending moment occurs in the Box $800 \times 600 \times 25$ section, with a value of 1,441,964,206 N·mm. Meanwhile, the maximum shear force occurs in the Box $800 \times 600 \times 25$ section, with a value of 364,202.15 N. Following the load combination checks, the highest demand-to-capacity ratio is found in the Box $800 \times 600 \times 25$ section under load combination H3-6, with a value of 1.001. Since this value exceeds the allowable limit of 1.0, the structural member does not satisfy the design requirements under the specified load combination. In contrast, the lowest demand-to-capacity ratio is found in the Box $300 \times 300 \times 20$ section under load combination H1-1a, with a value of 0.115.

5.3. Vertical hanger

The steel arch bridge with the Nielsen-type hanger configuration exhibits a maximum deflection of 695.266 mm (0.695 m), which exceeds the allowable deflection limit of $\Delta_{ijin} = \frac{L}{8} = 125 \text{ mm}$. The maximum axial force is 8,916.053 kN in compression, the maximum shear force is 297.931 kN, and the maximum bending moment is 297.931 kN·m. The maximum compressive force occurs in the Box $800 \times 600 \times 25$ section, with a value of 8,917,885.85 N. The maximum tensile force occurs in the Sling 70 section, with a value of 711,411.69 N. The maximum bending moment occurs in the Box $800 \times 600 \times 25$ section, with a value of 3,790,551,550 N·mm. Meanwhile, the maximum shear force occurs in the Box $800 \times 600 \times 25$ section, with a value of 412,085.33 N. Following the load combination checks, the highest demand-to-capacity ratios are found in the Box $800 \times 600 \times 25$ section, with values of 1.612 under load combination H3-6 and 1.432 under load combination H1-1a. Since both values exceed the allowable limit of 1.0, the structural member does not satisfy the design requirements under these load combinations. In contrast, the lowest demand-to-capacity ratio is found in the Box $300 \times 300 \times 20$ section under load combination H1-1a, with a value of 0.135.

Based on the findings presented above, among the three hanger configurations network, Nielsen, and vertical the network-type hanger configuration demonstrates the best overall structural performance. This configuration produces a deflection that remains within the allowable limit, whereas the deflections obtained from the bridges using the Nielsen-type and vertical hanger configurations exceed the allowable deflection limit. Furthermore, the internal forces, including axial forces, tensile forces, shear forces, and bending moments, are lower in the bridge with the network-type hanger configuration than in those with the Nielsen-type and vertical hanger configurations. The load combination checks also indicate that the network-type

hanger configuration satisfies the design criteria, with demand-to-capacity ratios remaining below 1.0. In contrast, the ratios obtained for the bridges with the Nielsen-type and vertical hanger configurations exceed 1.0, indicating that the corresponding structural members do not satisfy the required capacity criteria.

BIBLIOGRAPHY

- Al-Hussein, M., Ali, M., & Hassan, R. (2022). Dynamic response of steel arch bridges under seismic loads. *Advances in Structural Engineering*, 25(14), 3021–3034.
- Anggraini, D. A. (2019). *Modifikasi perencanaan jembatan kali kendeng dengan menggunakan sistem steel box through arch*.
- Badan Standardisasi Nasional. (2016). *SNI 1725:2016 - Beban minimum untuk perancangan bangunan gedung dan struktur lain*. BSN.
- Bathista, W., Sulasmono, B., & Risdanareni, P. (2019). Optimization of hanger spacing of steel arch bridges using dynamic loads. *INERSLA: Informasi Dan Ekspose Hasil Riset Teknik Sipil Dan Arsitektur*, 15(2), 67–75.
- Batista, R., Silva, L., & Mendes, P. (2021). Efficiency of steel arch bridges for medium and long spans. *Journal of Constructional Steel Research*, 182, 106659.
- Casas, J. R., & Moughty, J. (2021). Dynamic behaviour of steel bridges under traffic loads. *Structure and Infrastructure Engineering*, 17(9), 1205–1220.
- Chen, Y., Li, H., & Xu, Z. (2020). Structural behavior of steel arch bridges under combined loading. *Journal of Bridge Engineering*, 25(6), 4020045.
- Computers and Structures Inc. (2022). *SAP2000 integrated structural analysis and design software*. CSI.
- Computers and Structures Inc. (2023). *SAP2000 integrated software for structural analysis & design: User manual v24*.
- Deng, Y., Liu, H., & Zhao, M. (2022). Influence of hanger arrangement on steel arch bridge performance. *Engineering Structures*, 255, 113930.
- Fajrin, A. (2016). Analisa perencanaan struktur atas jembatan pelengkung baja. *Rekayasa Teknik Sipil*, 2(1), 45–53.
- Ghasemi, H., & Nowak, A. (2021). Structural efficiency of long-span Steel arch bridges. *Engineering Structures*, 234, 111979.
- He, X., Li, Q., & Sun, L. (2020). Structural performance of steel arch bridges under service loads. *Journal of Bridge Engineering*, 25(4), 4020012.
- Huang, T., Zhou, K., & Lin, J. (2023). Effect of hanger spacing on structural performance of arch bridges. *Journal of Bridge Engineering*, 28(5), 4023022.
- Kim, H., & Lee, J. (2022). Sustainable bridge design considering structural efficiency. *Sustainability*, 14(5), 2689.
- Kwon, S., & Kim, Y. (2020). Finite element modeling techniques for steel bridge structures. *Steel and Composite Structures*, 35(2), 215–228.
- Liu, J., Wang, P., & Zhao, X. (2020). Effect of hanger arrangement on steel arch bridge performance. *Engineering Structures*, 213, 110551.
- Liu, Z., Wang, Y., & Chen, H. (2024). Advanced finite element modeling of steel arch bridges. *Engineering Structures*, 287, 116137.
- Ma, X., Zhou, Y., & Liu, K. (2024). Integrated static and dynamic analysis of steel arch bridges. *Structures*, 54, 1298–1310.
- Mulyadi, A. (2016). Kelebihan material baja dalam struktur jembatan. *Media Teknik Sipil*, 18(1), 55–62.
- Nowak, A. S., & Collins, K. R. (2020). *Reliability of structures* (3rd ed.). CRC Press.
- Qin, X., Zhang, Y., & Li, J. (2021). Optimization of arch geometry for steel arch bridges. *Structures*, 34, 3251–3263.
- Ramadhan, R., Prabowo, A., & Setiawan, D. (2025). Structural analysis of steel arch bridges using SAP2000. *Jurnal Rekayasa Infrastruktur*, 11(1), 1–12.
- Ren, W., Xu, F., & Gao, Y. (2023). Dynamic analysis of steel arch bridges considering support conditions. *Structures*, 46, 1523–1536.
- Schlaich, M., & Scheef, H. (2020). Network arch bridges: Recent developments and design concepts. *Structural Engineering International*, 30(3), 394–401.
- Setiawan, B. (2017). Analisis struktur jembatan pelengkung baja. *Jurnal Teknik Sipil*, 24(2), 113–121.
- Siringoringo, D., & Mulyono, A. (2022). Evaluasi kombinasi beban dinamis pada jembatan baja di Indonesia.

- Jurnal Teknik Sipil*, 29(3), 189–200.
- Torres, J., Martinez, P., & Gomez, R. (2024). Numerical optimization of steel arch bridge geometry. *Journal of Constructional Steel Research*, 206, 107973.
- Triyoga, B. (2017). *Modifikasi struktur jembatan Musi IV dengan menggunakan sistem box half through arch*.
- Wang, T., Huang, Y., & Lin, J. (2023). Dynamic analysis of steel arch bridges under traffic loading. *Journal of Bridge Engineering*, 28(3), 4022124.
- Zhang, L., Sun, Y., & Zhou, M. (2021). Three-dimensional finite element modeling of long-span arch bridges. *Structures*, 33, 2145–2156.
- Zhou, Q., Li, R., & Chen, S. (2022). Comparative study of hanger systems in steel arch bridges. *Engineering Structures*, 252, 113643.
- Zhu, B., Chen, S., & Wang, Z. (2021). Structural characteristics of through arch bridges. *Journal of Constructional Steel Research*, 183, 106734.