

Superstructure Design of PCI Girder Bridge (Case Study: Bridge Project in Padang Pariaman, West Sumatera)

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Article Information	ABSTRACT
Article History Received : June 01, 2025 Revised : June 16, 2025 Published : July 04, 2025	This study was conducted in response to the damage of the Padang Pariaman Bridge, which had exceeded its service life, further worsened by a flood disaster in 2023 in Padang Pariaman, West Sumatra. The bridge redesign in this study adopts a PCI Girder type with a span of 35 meters. Load calculations refer to SNI 1725:2016, and structural analysis was carried out using MIDAS Civil software. The bridge design follows the AASHTO LRFD 2007 standards, covering the determination of the number and layout of tendons, loss of prestress analysis, concrete stress evaluation, deflection checks, and shear capacity design. The design results in this study serve as a redesign of the damaged bridge in Padang Pariaman, with adjustments made to the load combinations and response spectrum parameters specific to the region. The analysis results indicate the use of PCI Girder type H-170 with 64 strands divided into 4 tendons. A total prestress loss of 17.94% or 333.77 MPa occurred at midspan. The concrete section stresses remain below the limits specified in the standards, ensuring the section is safe from cracking. Additionally, deflection caused by the combination of vehicle and service loads remains within permissible limits. The bridge design also requires shear reinforcement with D13-200 specifications.
Keywords: <i>Bridge;</i> <i>PCI Girder;</i> <i>AASHTO LRFD;</i> <i>Stress;</i> <i>Deflection.</i>	

INTRODUCTION

Bridges are critical infrastructure that support interregional connectivity (Aleksander, 2020). Bridge serve to connect two points separated by obstacles such as rivers, roads, seas, valleys, railway tracks, or underlying traffic (Mauliana & Wahyu Saputra, 2024). Given the critical role of bridges in linking transportation routes, it is essential that they possess sufficient structural strength to ensure the safety and comfort of their users. In infrastructure planning, service life is one of the key factors that influences the structural performance and durability of a facility (Li et al., 2019). As the service life of a structure increases, its strength and performance may gradually decline due to various factors such as environmental degradation and increasing service loads (Cho et al., 2025). Similarly, bridges are also subject to structural degradation over time as their service life progresses.

The Bridge in Padang Pariaman is one of the bridges that has reached the end of its service life. Furthermore, in 2023, according to reports from the National Disaster Management Agency (BNPB) and several national media outlets, Padang Pariaman in West Sumatra was hit by a flash flood that caused extensive damage to infrastructure, including bridges. Therefore, a redesign is necessary, particularly for the superstructure of the bridge in Padang Pariaman, West Sumatra,

taking into account various considerations such as girder detailing, prestress losses, sectional stresses, deflection, and the structural capacity of the bridge.

Various materials can be used as structural components of bridges, including conventional reinforced concrete, steel trusses, and prestressed concrete (Mauliana & Wahyu Saputra, 2024). The use of steel as tendons in prestressed concrete has proven to be an efficient solution, primarily due to the lighter structural elements and more economical material properties of prestressed concrete compared to structural steel (Frapanti & Zulkarnain, 2021). Therefore, in this study, the superstructure of the bridge is designed using prestressed concrete as the material for the bridge girders. The design specifications are based on the Regulation of the Minister of Public Works and Housing (PUPR) No. 07/SE/M/2015 concerning the “General Guidelines for Bridge Design Requirements.” The bridge to be designed features a single-span PCI girder with a total length of 35 meters and a width of 11 meters.

Prestressed concrete is a composite material consisting of two main components: concrete and reinforcing steel (tendons) (Putra et al., 2017). Each material possesses distinct advantages in resisting different types of forces-concrete exhibits high compressive strength, while steel tendons are strong in tension (Rafieizonooz et al., 2024). Prestressed concrete utilizes initial tensioning of the tendons prior to the application of external loads on the structure (Zebua et al., 2022). The utilization of steel as tendons in prestressed concrete offers a structurally efficient approach. Unlike conventional reinforcement that acts passively after loading, prestressing tendons actively counteract deflection, thereby improving structural performance, particularly in bridge applications (Zou et al., 2019).

Prestressed concrete is susceptible to the effects of shrinkage and creep (Reybrouck et al., 2020). Shrinkage occurs as the concrete undergoes volumetric contraction, while creep refers to the time-dependent deformation of concrete under sustained loading (Brice & State, 2023). Both shrinkage and creep can lead to a loss of the initial prestressing force (Podolny Jr., 1969). If not properly considered in the design process, these factors may reduce the effectiveness of the prestressing system (Kim, 2023). Stressing introduces internal forces in concrete due to the application of prestressing tendons, while additional stresses may result from service loads acting on the bridge (Liu et al., 2021). It is crucial to evaluate these stresses to ensure the girder can resist them without structural failure, particularly in segmental bridge systems (Puspitasari et al., 2023).

Accurate identification of structural loads is a fundamental aspect of bridge design, with load characteristics influenced by bridge type and environmental conditions (Sumargo et al., 2012). In Indonesia, bridge loading is regulated by the national standard SNI 1725:2016. Compliance with this standard is essential to ensure that bridge designs meet the required safety and performance criteria over their intended service life (Sabara & Imran, 2024). Consistent application of these standards plays a key role in ensuring structural reliability and preventing failure risks that may endanger bridge users.

Like other types of infrastructure, bridges are also vulnerable to natural disasters such as earthquakes (Mardhiyah et al., 2022). Geologically, Indonesia is located along the Pacific Ring of Fire, a region of high volcanic activity that frequently triggers tectonic plate movements (Adri et al., 2020). According to the Ministry of Energy and Mineral Resources (ESDM), Indonesia has 127 active volcanoes, 16 of which are currently under alert status. Among the most seismically active regions is West Sumatra, which is highly prone to earthquakes (Baihaqi & Pujiastuti, 2023). This vulnerability stems from the presence of the subduction zone between the Indo-Australian and Eurasian plates, as well as several active fault lines such as the Mentawai Fault and the Sumatran Fault (Hasan & Santosa, 2014). Therefore, it is essential to design structures such as bridges to withstand seismic loads, particularly in regions that are highly vulnerable to such hazards.

In addition to long-term losses due to shrinkage and creep, prestress force may also decrease instantaneously due to factors such as friction and slip during anchorage, elastic shortening of concrete, and tendon relaxation (Han et al., 2023). In bridge structural design, the effects of shrinkage and creep are often neglected, despite their significant influence on internal forces. Furthermore, precast concrete materials exhibit varying compressive strengths at different stages-transfer, construction, and service life. Therefore, this study takes into account all relevant prestress losses that may occur in the prestressing tendons.

RESEARCH METHODS

The case study in this research is based on the Padang Pariaman Bridge project located in West Sumatra. The bridge is designed to accommodate a 2/2-TT traffic configuration, with a span length of 35 meters and a total width of 11 meters. **Figure 1** presents the bridge profile that serves as the object of study in this research.

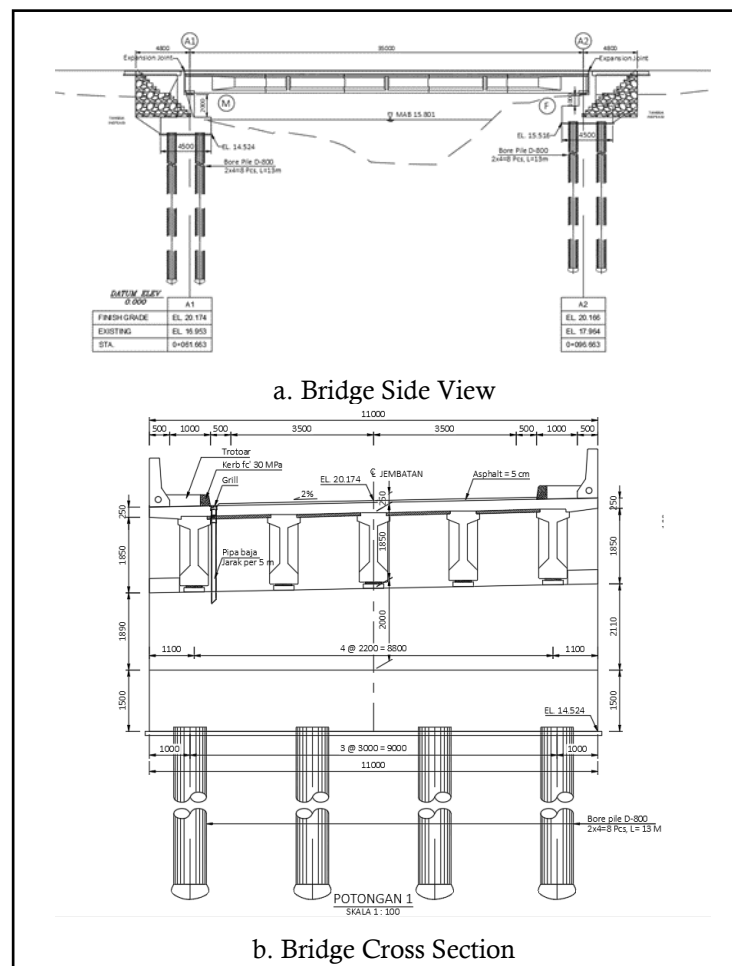


Figure 1. General Overview of The Bridge

Based on the **Figure 1**, the bridge consists of a single span supported by two abutments at its ends. The structural system in this design incorporates hinge or roller supports at each bearing location. The compressive strength specified for the girders is 45 MPa, while for the bridge slab it is 30 MPa.

The research location is based on the bridge project in Padang Pariaman, West Sumatra, with the site appearance shown in **Figure 2**. The research was conducted from December 2024 to May 2025.



Figure 2. Bridge Project Location

In modeling and performing structural analysis of the bridge using MIDAS Civil V22 software, the following steps can be carried out:

1. Defining the material properties used in the MIDAS Civil software.
2. Defining the section properties to be modeled as bridge components, including PCI-H 170 type girders, the bridge slab, and diaphragms.
3. Defining Time Dependent Material to incorporate the effects of concrete shrinkage and creep, as well as specifying the concrete compressive strength up to 28 days.
4. Modeling the bridge based on the predetermined bridge geometry as shown in **Figure 3**, and assigning material links according to the structural behavior of the bridge.
5. Defining loads and load combinations used for analysis based on the provisions of SNI 1725:2016.
6. Summarizing the moments and internal forces based on the structural analysis results using MIDAS Civil software.

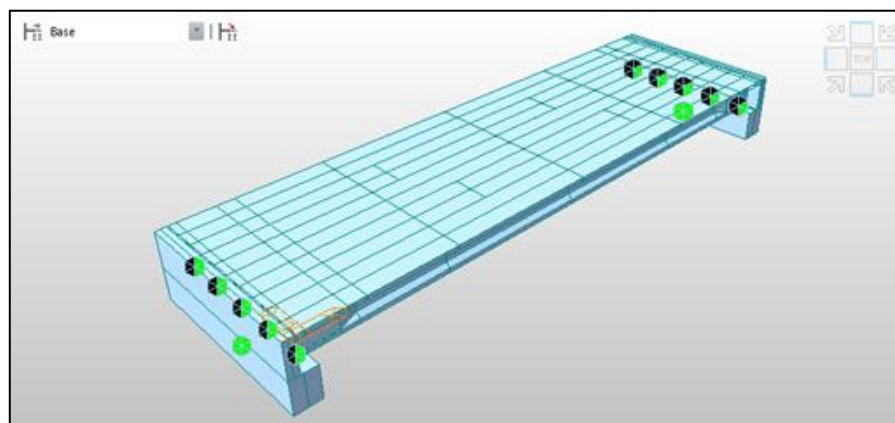


Figure 3. 3D Model of The Bridge in MIDAS Civil

After obtaining the internal moments and forces, the next step is to analyze the quantity of strands and tendons used, as well as to determine the prestress losses occurring in the tendons. Subsequently, checks are performed on the concrete section stresses, the deflections experienced, and the shear capacity analysis of the girder section. **Figure 4** illustrates the flow diagram related to the procedure of this research.

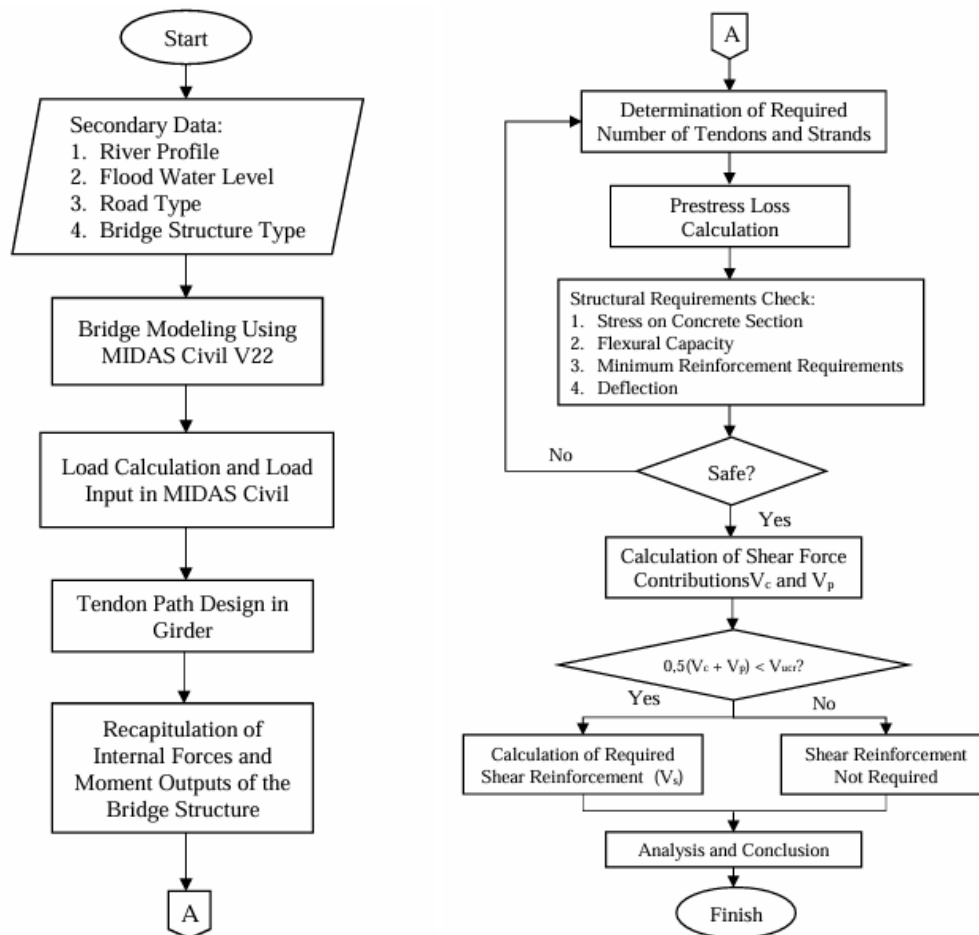


Figure 4. Research Flowchart

RESULTS AND DISCUSSION

In this study, the design of the bridge superstructure was carried out using MIDAS Civil software to obtain internal forces, moments, and loads on the substructure based on the Load Resistance Factor Design (LRFD) method, referring to SNI 1725:2016.

Tabel 1. Summary of Internal Forces and Moments

No	Combination	W (kN/m)	Moment, M (kN.m)	Shear, V (kN)
1	Strength I	86,99	13320,61	1522,36
2	Strength II	79,36	12152,00	1388,80
3	Strength III	53,06	8125,21	928,60
4	Strength IV	52,88	8097,66	925,45
5	Strength V	52,94	8106,68	926,48
6	Extreme I	74,58	11419,57	1305,09
7	Extreme II	58,03	8885,98	1015,54
8	Service I	66,74	10218,99	1167,88
9	Service II	72,08	11036,53	1261,32
10	Service III	62,89	9630,76	1100,66
11	Service IV	47,41	7259,83	829,69
12	Fatigue	13,71	2099,56	239,95

Based on the modeling and internal force analysis conducted using MIDAS Civil software, it was found that the load combination in Strength I produced the highest values of ultimate moment and shear force, which were 13,320.61 kNm and 1,522.36 kN, respectively. Meanwhile,

for serviceability purposes, the highest values were generated by Service II, with a moment of 11,036.53 kNm and a shear force of 1,261.32 kN. Similar findings were also reported in the study by Mauliana & Wahyu Saputra (2024) which referred to SNI 1725:2016, where the governing load combination was found to be Strength I and Service II. However, this does not represent a definitive configuration or conclusion, as larger results may arise from other combinations due to the dominance of different loading conditions.

Strand and Tendon Configuration

In this study, tendons with a bounded type 7 Wire Super Strand ASTM A-416 were used based on the Vorspann System Losinger (VSL) standard. The calculation of the minimum strand requirement was carried out based on the following equation.

$$n_{strand_{req}} = \frac{P_e}{P_{pe_strand}} \quad (1)$$

Where P_e is the effective prestressing force based on the estimated loss of prestress, and P_{pe_strand} is the effective prestressing force per strand. Therefore, a minimum of 52 strands is required.

Tabel 2. Recapitulation of The Number of Strands Used

Tendon	Unit	Number of Strand	Anchor Width (mm)		Anchor Diameter (mm)
			Stressing	Dead End	
1	5 – 7	7	165	175	54
2	5 – 19	19	265	375	87
3	5 – 19	19	265	375	87
4	5 – 19	19	265	375	87

In this design, a total of 64 strands were used, determined through a trial-and-error process after accounting for loss of prestress and performing other necessary verifications. Based on this number of strands, a configuration of 4 tendons was adopted, consisting of 7 and 19 strands per tendon. The following is the formula for the tendon profile.

$$y = 4e_s \frac{X}{L^2} (L - X) \quad (2)$$

Where e_s denotes the tendon eccentricity and L is the span length of the bridge. Below is the tendon trajectory used in the study.

Tabel 3. Tendon Traces

X (m)	Tendon 1	Tendon 2	Tendon 3	Tendon 4
	Z1 (m)	Z2 (m)	Z3 (m)	Z4 (m)
0,000	1,200	0,900	0,600	0,300
1,750	1,029	0,758	0,515	0,272
3,500	0,876	0,630	0,438	0,246
5,250	0,741	0,518	0,371	0,224
7,000	0,624	0,420	0,312	0,204
8,750	0,525	0,338	0,263	0,188
10,500	0,444	0,270	0,222	0,174
12,250	0,381	0,218	0,191	0,164
14,000	0,336	0,180	0,168	0,156
15,750	0,309	0,158	0,155	0,152
17,500	0,300	0,150	0,150	0,150

These values represent the tendon profiles measured from the bottom of the girder. Based on this, the average tendon eccentricity was determined to be 499.22 mm.

Analysis of Prestress Losses

Prestress losses in tendons can occur from the transfer stage to the service life of the bridge. However, throughout this process, the structural behavior differs between the transfer stage, the casting of the bridge slab, and the post-composite girder condition. Therefore, the calculation of prestress losses is adjusted according to the cross-sectional properties during the period from transfer to slab casting, and from slab casting to the end of the service life. The equations used to determine the prestress losses are based on the provisions of AASHTO LRFD 2007. The following presents the results of the prestress losses that occurred.

Tabel 4. Summary of Total Prestress Losses

Condition	Loss of Prestress		% UTS
	(MPa)	(%)	Effective Prestress
Short Term, ΔP_{ST}	141,38	7,60	67,40
Long Term, ΔP_{LT}	192,39	10,34	57,06
Total	333,77	17,94	-

Short-term prestress losses occur due to several factors during the transfer process, such as friction along the tendon, anchorage slip, and elastic shortening of the concrete. In contrast, long-term losses result from concrete shrinkage and creep, as well as strand relaxation. The following presents the detailed remaining prestress in the bridge from the transfer stage to the end of its service life.

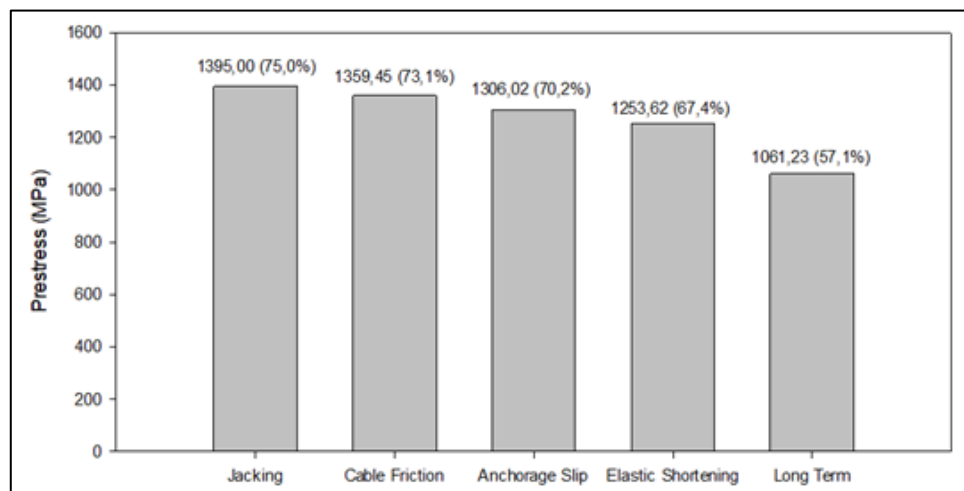


Figure 3. Detailed Remaining Prestress in The Bridge

Based on the calculations that have been conducted in this planning, the immediate loss of prestressing during transfer until the casting of the bridge deck slab is 7.60%. Subsequently, the loss of prestressing that occurs in the mid-span after the casting is completed until the final stage is 10.34%. Therefore, the total prestressing loss from transfer until the end of service (final) is 17.94%, resulting in the remaining prestressing (effective prestressing) of 57.06% of the tensile strength of the prestressing steel, denoted as f_{pu} . These results are consistent with the findings of Han et al. (2023), who compared prestress loss analysis using experimental methods and the AASHTO LRFD 2007 provisions, indicating that prestress losses can occur from the transfer stage through to the end of the bridge's service life.

Stress Analysis on Concrete

The stress examination on the girder cross-section is conducted under the conditions of prestressing transfer, during construction, and at the service stage. The stress limits on the concrete section are regulated in AASHTO LRFD 2007. Below are the equations used to calculate the stress on the concrete section.

$$f_t = \frac{-P}{A_g} + \frac{Pe_{mid}}{S_t} + \frac{-M}{S_t} \quad (3)$$

$$f_b = \frac{-P}{A_g} + \frac{-Pe_{mid}}{S_b} + \frac{M}{S_b} \quad (4)$$

In the equation f_t represents the stress on the top side of the concrete, while f_b denotes the stress on the bottom side. Here, P is the prestressing force at each condition, e is the average eccentricity of the tendon, and S is the section modulus. The following presents the results of the stress examination on concrete based on the conditions of transfer, construction, and service.

Tabel 5. Stress Examination at Mid-Span

Condition	Location	P/A _g (MPa)	P.e/S (MPa)	M/S (MPa)	Total (MPa)	Limit (MPa)	Check
Transfer at Girder	Top	-11,78	19,10	-9,50	-2,17	-21,60	OK
	Bottom	-11,78	-17,81	8,85	-20,74	-21,60	OK
Construction at Girder	Top	-11,78	19,11	-16,47	-9,15	-21,60	OK
	Bottom	-11,78	-17,81	15,36	-14,23	-21,60	OK
Service at Girder	Top	-9,97	16,17	-20,52	-14,31	-20,25	OK
	Bottom	-9,97	-15,08	27,38	2,33	3,35	OK
Service at Slab	Top	-	-	-4,44	-4,44	-13,50	OK
	Bottom	-	-	-2,72	-2,72	-13,50	OK

The table indicates that the cross-section of the girder experiences compression in all conditions except for the service condition, as evidenced by the negative values. Furthermore, overall, the stress values remain within the safe limits specified by AASHTO LRFD 2007.

In segmental bridges, AASHTO LRFD 2007 prohibits the occurrence of tensile stress at each joint of the bridge girder. Therefore, it is necessary to conduct a stress examination at these locations to ensure compliance with the specified requirements.

Tabel 6. Stress Examination at Girder Joints

Connection Location (m)	Section Location	Prestress (MPa)	Check
6,60	Top	-11,78	OK
	Bottom	-4,99	OK
13,60	Top	-14,88	OK
	Bottom	-0,75	OK
21,40	Top	-14,80	OK
	Bottom	-1,11	OK
28,40	Top	-11,72	OK
	Bottom	-5,84	OK

From the specified girder joints, no tensile stress occurs at these locations. This examination is necessary due to the linear variation in prestress losses along the girder caused by cable friction and slip during anchorage. The findings of this study are also in agreement with previous research conducted by Puspitasari et al. (2023), which reported the absence of tensile stress at the girder joints. The presence of tensile stress at the joint locations could lead to girder displacement and potentially result in the failure of the bridge structure.

Analysis of Bridge Deflection

Vertical deformation in prestressed bridges can occur in opposite directions due to stressing, which causes camber, and loading, which induces deflection. In general, the calculation of camber and deflection in bridges is based on the following equations.

$$\Delta_p = \frac{P_{transfer}}{E_{cig} I_g} \left[\frac{e_{mid} L_b^2}{8} - \frac{(e_{end} - e_{mid}) \left(\frac{L_b}{2}\right)^2}{6} \right] \quad (5)$$

$$\Delta_L = \frac{5}{384} \cdot W \cdot \frac{L_b^4}{E_{cig} I_g} \quad (6)$$

In the equation, Δ_p represents the camber that occurs at the time of transfer, while Δ_L denotes the deflection due to the applied loads. Here, E is the modulus of elasticity of concrete, and I is the moment of inertia of the girder cross-section. The following presents the results of the deflection that occurs.

Tabel 7. Examination of Deflection

Condition	Deflection (mm)	Direction	Limit (mm)	Check
Transfer	147,93	Up	-	-
Girder Load	48,85	Down	-	-
Dnc	39,28	Down	-	-
Utility	25,52	Down	-	-
Construction	175,90	Up	-	-
Traffic Load	28,34	Down	43,75	OK
Service	39,42	Down	116,67	OK

Although no strict regulations govern allowable deflection limits in bridge design, AASHTO LRFD 2007 provides recommended deflection criteria of $L/800$ for live load and $L/300$ for service load conditions. Based on the deflection calculations for the girder, it was found that the maximum deflection occurred due to the self-weight of the girder, measuring 48,85 mm. Additionally, the stressing process on the girder resulted in a camber of 147,93 mm upwards, which mitigated the downward deflection during service to a value of 39,42 mm. The deflection is limited to conditions caused by vehicle loads and service, with thresholds of 43,75 mm for vehicles and 116,67 mm for service conditions. In this study, both deflections remained below the established maximum limits, categorizing them as safe.

Analysis of Flexural Capacity

In the structural analysis phase of a bridge, determining the flexural and shear capacity is a critical step in evaluating the resistance of structural elements to the loads they are subjected to. This calculation aims to ensure that structural components, such as girders, can withstand moments and shear forces without experiencing failure. The following equation represents the method for determining flexural capacity.

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2} \right) \quad (7)$$

Where A_{ps} is the total cross-sectional area of the tendons, f_{ps} is the average stress in the tendons, d_p is the effective depth of the section, and a represents the height of the equivalent compression block. By applying a reduction factor, Φ_f of 0.9, the flexural capacity was calculated to be 17,686.29 kNm. This value exceeds the largest ultimate moment, M_u , from strength I, which is 13,320.61 kNm. Furthermore, this value also surpasses the minimum flexural reinforcement limit produced by 1.2 M_{cr} , which is 16,823.89 kNm.

Analysis of Shear Capacity

The determination of whether the section requires shear reinforcement must be based on the value of the ultimate cracking shear force in the girder, V_{ucr} . If this value exceeds $0.5 (V_c + V_p)$, then the section necessitates shear reinforcement. This assessment is crucial for ensuring the structural integrity and safety of the girder under applied loads.

$$V_p = P_{eff} \frac{(y_{end} - y_{mid})}{r_{tendon}} \quad (8)$$

$$V_c = 0,083 \beta \sqrt{f'c} b_v d_v \quad (9)$$

Where V_p represents the contribution of shear resistance provided by the prestressing steel, and V_c is the shear resistance contributed by the concrete. In the equation, r_{tendon} refers to the average length of the tendon in half of the span, b_v is the effective shear width, and d_v is the effective shear depth. Here, β is a reinforcement factor assumed for the minimum reinforcement condition.

Tabel 8. Examination of Shear Reinforcement Requirements

Contribution of Shear Force (kN)			V _{ucr} (kN)	V _{s_req} (kN)
Concrete, V _c	Strands, V _p	0,5 (V _c + V _p)		
880,59	191,17	535,88	1379,04	766,96

Based on the results of the shear capacity evaluation of the concrete and prestressing steel, the section requires additional shear capacity. Therefore, non-prestressed reinforcement, V_s with the specification D13-200, is utilized to fulfill the required shear capacity.

CONCLUSION

The redesign of the bridge superstructure in Padang Pariaman, which had deteriorated due to a flood disaster and the expiration of its service life, was carried out using a load combination of Strength I and Service II, with adjustments made to reflect the regional seismic response spectrum. The AASHTO LRFD 2007 provisions impose limits on allowable service stress, thereby constraining the magnitude of prestressing force. This necessitates careful optimization of strand quantity and tendon configuration to ensure structural performance while maintaining stress within permissible bounds. Based on the design analysis, a PCI girder of type H-170 was selected, utilizing 64 prestressing strands distributed across 4 tendons. The total prestress loss, evaluated based on AASHTO LRFD 2007, was 17,94% or 333,77 MPa at the midspan tendon, resulting from multiple contributing factors. The resulting stresses in the concrete section remained below the allowable compressive and tensile stress limits, thereby ensuring that the section remains uncracked. Additionally, a camber of 175,90 mm is expected during construction, and the deflection due to vehicular and service loads remains within the allowable limits, indicating sufficient serviceability. The design also requires shear reinforcement with D13-200 specifications. Flexural and shear capacity evaluations confirm that the provided capacities exceed the ultimate moment and shear force demands, thereby validating the structural safety of the redesigned system.

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