

Structural Design of a Novel Longitudinally Split Precast Segmental Cap Beam

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Abstract: Based on an expressway project in Shanghai, the structural behavior, construction procedure, and engineering applicability of a novel longitudinally split precast segmental cap beam were investigated. The proposed cap beam is divided longitudinally into two precast segments for lifting and erection. After erection, the segments are connected by a cast-in-place wet joint placed on a specially designed bottom formwork, while the cap beam is connected to the columns through grouted sleeve connections. For the two-column portal-frame cap beam commonly used in urban viaducts, two tendon-layout schemes were developed. In Scheme 1, all the prestressing tendons are arranged within the precast segments; the wet joint is 500 mm wide, and the single-lift weight is 143 t. In Scheme 2, some of the prestressing tendons are arranged within the wet joint; the joint width is 700 mm, and the single-lift weight is 126 t, approximately 12% lower than that of Scheme 1. The results show that the wet-joint width is strongly coupled with the single-lift weight, available construction space, and tendon arrangement and is therefore a key parameter governing the applicability of the system. Compared with the conventional transversely segmented precast cap beam, the proposed longitudinally split scheme increases tendon consumption by approximately 20% and requires more cast-in-place work but reduces the single-lift weight by approximately 30% and eliminates temporary support and traffic diversion, minimizing disruption to under-bridge traffic. The proposed system therefore demonstrates good engineering applicability and provides technical support for the prefabricated and rapid construction of urban viaducts.

Keywords: longitudinally split precast cap beam; two-column portal-frame cap beam; engineering applicability; urban viaduct

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1 Introduction

With the continuing acceleration of urbanization, the scale of urban viaduct construction has increased steadily. Prefabricated structures have been increasingly adopted because of their rapid construction, limited disruption to traffic, and reduced onsite workload [1]. Precast superstructures, including hollow-core slab girders, small box girders, and composite steel plate girders, have been widely used in urban viaducts, whereas the development and application of corresponding precast cap-beam systems have progressed relatively slowly. To promote the engineering application of precast cap beams, Sha et al. [2] proposed a vertically layered scheme for inverted-T cap beams, in which the flange is precast, and the web is cast in situ to reduce the lifting weight, although additional on-site formwork is required. Xu [3], Li [4,5], Ye [6,7], Wu [8], Li [9], Zhou [10], and other researchers subsequently proposed precast steel cap beams, precast lightweight UHPC cap beams, UHPC-RC composite cap beams, and precast assembled composite cap beams. These systems have not been widely adopted because of their complex detailing, construction difficulty, and high cost. In recent years, segmental precast cap beams have been increasingly used in practice. Most are transversely segmented and assembled on site by connecting the segments through joints [11], including cast-in-place concrete wet joints [12], epoxy joints incorporating corbels or shear keys together with prestressing [13,14], and composite joints [15]. Urban viaducts

generally carry heavy traffic and have wide decks; consequently, portal-frame cap beams are typically used in the substructure and have relatively large spans and dimensions. Because urban surface structures are complex and underground utilities are dense, even larger cap-beam spans may be required to avoid conflicts. Cast-in-place cap beams and fully precast cap beams are both commonly used in bridge substructures. Cast-in-place cap beams have long construction periods and substantially affect under-bridge traffic, making them unsuitable for many urban viaduct projects. Fully precast cap beams, in contrast, are constrained by lifting capacity and are difficult to apply to large-span prestressed cap beams; they also involve greater construction difficulty and safety risks. Considering their technical feasibility, under-bridge traffic, project cost, and construction difficulty, precast segmental concrete cap beams are therefore preferable. For example, the Wuxi Fengxiang Road Expressway Reconstruction Project, the Shanghai S3 Highway New Construction Project, and the Middle Section Project of Lianggang Avenue all adopted transversely segmented precast cap beams assembled on site. Large-scale applications of segmental precast cap beams also include projects such as the reconstruction and expansion of the Fengpu East Bridge on the Shanghai S4 Highway and Phase II of the north section of the Jiamin Elevated Road.

Overall, existing precast segmental cap-beam projects adopt transverse segmentation. By precasting the cap beam in segments along the transverse bridge direction and assembling them on site, this approach effectively addresses the transportation and lifting difficulties associated with very heavy cap beams. Nevertheless, transversely segmented cap beams generally require temporary support, which occupies ground space beneath the bridge during construction and consequently affects traffic. In particular, when a viaduct crosses important roads or underground structures, the installation of temporary supports may introduce considerable safety risks.

Accordingly, on the basis of the two-column portal-frame cap beam of a completed expressway project in Shanghai, this study investigates a novel longitudinally split precast segmental concrete cap beam. The cap beam is longitudinally divided into two precast segments, which are lifted into positions and then connected into an integral system by casting a wet joint. This arrangement eliminates temporary construction support and minimizes disruption to under-bridge traffic. A finite element model was established in Midas Civil to investigate the structural behavior and engineering applicability of the two tendon-layout schemes and to compare them with the original scheme, thereby providing technical support for the prefabricated and rapid construction of urban viaducts. The proposed system has been successfully implemented in the Traffic Function Enhancement Project for the eastern section of the Shanghai Outer Ring Road (Huaxia Middle Road–Longdong Avenue), where 41 cap beams were constructed using longitudinally split precast segments connected by post-cast wet joints, demonstrating the feasibility and reliability of the scheme.

2 Project Overview

The standard section of the expressway has six lanes in two directions and has a design speed of 80 km/h under City-A loading. The standard main-line span is 30 m and uses small box girders, whereas the substructure consists of two-column portal-frame cap beams supported on $\phi 1,000$ mm bored cast-in-place piles. The existing cap beams are transversely segmented precast concrete structures, whose overall arrangement is shown in Figure 1. The bridge is 25.1 m wide; the cap beam is 29.016 m long and 2.2 m wide, with a midspan depth of 2.6 m. The cap beam is connected to precast assembled columns. The column center-to-center spacing, which is also the span of the portal-frame cap beam, is 25.640 m. Each column measures 1.5 m in the transverse direction and 2.0 m in the longitudinal direction, with pile caps and bored cast-in-place piles beneath. To avoid temporary supports

that would interfere with under-bridge traffic and metro shield tunneling, a novel concrete cap-beam system assembled from longitudinally split precast segments is investigated in this study [16]. Its cross-sectional layouts are shown in Figure 2a) and 2b). The widths of the precast segments and wet joints are jointly governed by structural requirements, lifting weight, and available construction space.

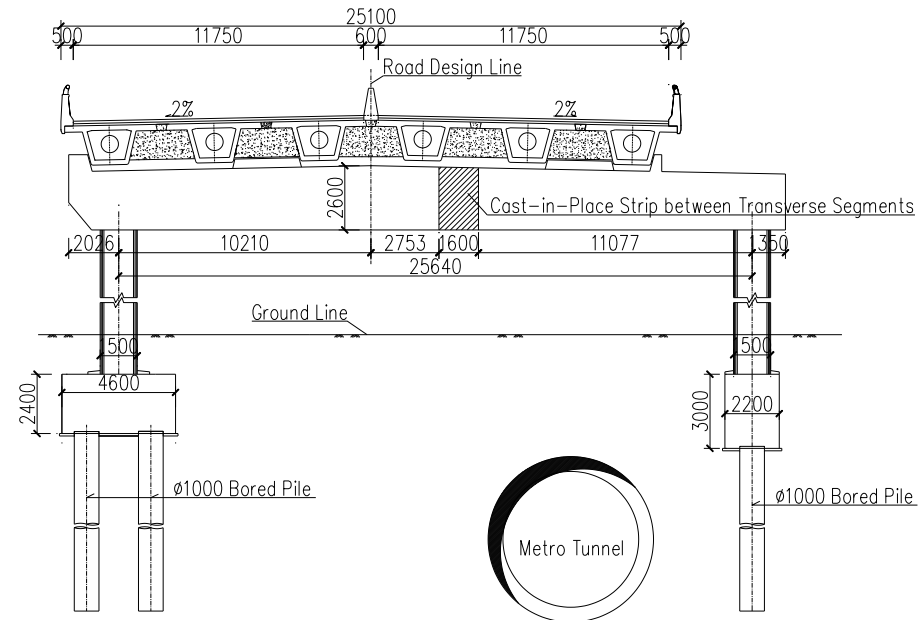


Figure 1 Overall layout of the existing transversely segmented precast cap beam (unit: mm)

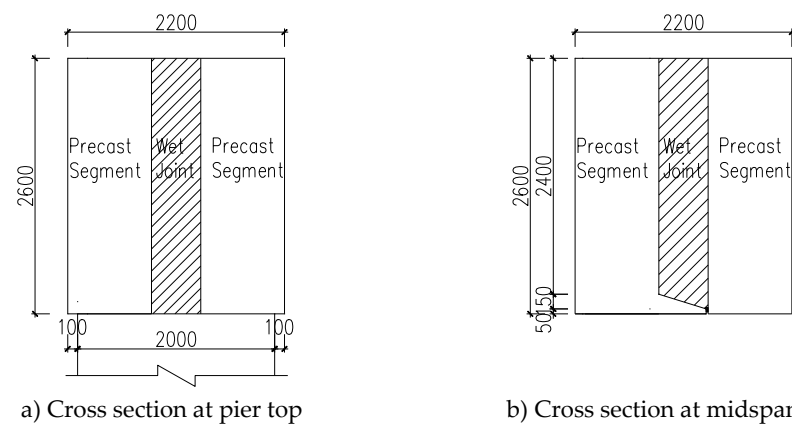


Figure 2 Cross-sectional layout of the proposed longitudinal split precast cap beam (unit: mm)

3 Key Details of the Cap Beam

3.1 Longitudinal Connection Details

After the precast cap-beam segments are lifted to the top of the piers, they must be connected in the longitudinal bridge direction so that the two segments can act integrally. Three types of joints are commonly used between concrete segments: dry joints, wet joints, and epoxy joints. Dry and epoxy joints can provide a certain degree of shear resistance through shear-key interlocking and interfacial friction. Although the epoxy adhesive in an epoxy joint has some tensile bond strength, this strength is highly variable and has poor durability, and it can readily deteriorate under long-term repeated traffic loading and temperature effects. Therefore, for structural safety, its tensile contribution is generally neglected in design, making the joint interface unfavorable when subjected to tensile stresses such as splitting forces near the bearings.

In principle, longitudinal prestressing tendons could be tensioned to generate a large longitudinal precompression force, tightly clamping the two cap-beam segments together so that the joint interface remains in compression under loading and interface friction can resist shear and splitting effects. In practical engineering, however, longitudinal tendons can readily conflict with the transverse prestressing tendons and dense conventional reinforcements in the cap beam. Their anchorage zones also require additional complex anchorage blocks or locally thickened sections, making reinforcement detailing extremely difficult. Exposed anchorage ends further impair the appearance of the cap-beam sides and substantially increase construction time and difficulty. A wet joint, by contrast, can provide reliable connections and integral action between the two segments, and its construction technology is relatively mature; it is therefore adopted as the preferred connection method. As shown in Figure 3a), longitudinal U-shaped stirrups are embedded at both the top and bottom of each precast segment. After the two segments are lifted into position, their centers of gravity remain within the pier-top region, and the opposing U-shaped stirrups are welded into closed loops to prevent outward overturning. Transverse reinforcement is then threaded through the closed stirrups and welded in place, after which the concrete is cast in the wet joint to form a longitudinal connection between the two segments [17]. This provides a reliable rigid connection. The post cast concrete firmly anchors the longitudinal load-carrying reinforcement at both the top and bottom; the top connecting reinforcement can directly and reliably resist the tensile stresses at the bearing region and transfer the internal forces smoothly to the precast segments on both sides through reinforcement-to-concrete bonding, thereby avoiding the brittle behavior associated with dry or epoxy joints that rely on interface friction or adhesive tensile resistance. The connection also establishes a reliable shear-resisting mechanism. The closed-loop reinforcement formed by the connected top and bottom stirrups, together with the transverse reinforcement in the wet joint, effectively confines the core concrete. This reinforcement cage resists not only flexural tensile stresses under vertical loading but also longitudinal shear at the joint through dowel action, enabling the two precast segments to deform compatibly and act as integral members.

3.2 Cap Beam-to-Column Connection Details

The connection between the precast cap-beam segments and the columns generally adopts mature and rapid grouted-sleeve technology [18], whereas direct cast-in-place concrete is used within the wet-joint region. As shown in Figure 3b), the positions of the column longitudinal reinforcements are accurately located within the precast cap beam, and the grouted sleeves are embedded in advance. After accurate field positioning and erection, pressure grouting is performed to connect the column longitudinal bars to the precast cap beam. At the wet joint, the concrete is cast on site to connect the cap beam directly to the column longitudinal reinforcement.

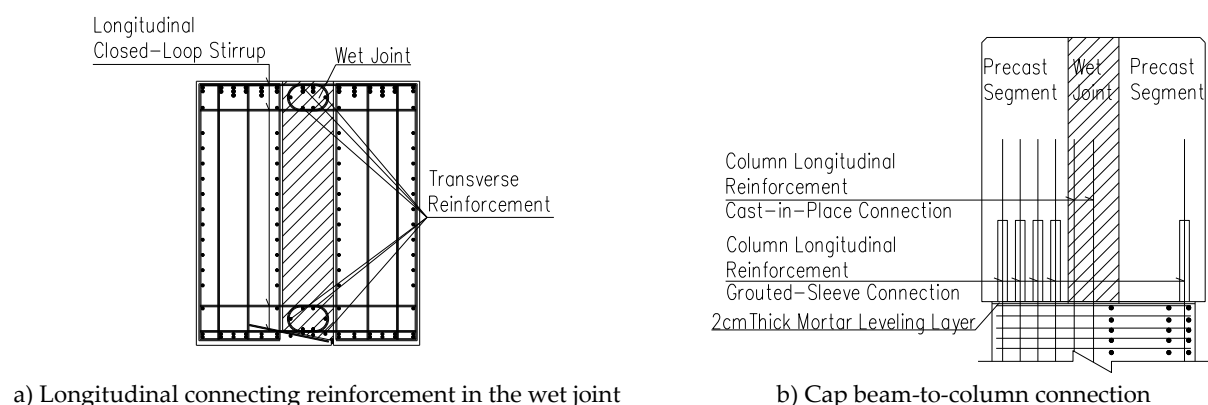


Figure 3 Key connection details of the proposed cap beam

3.3 Bottom Formwork Details

At the pier locations, the pier tops can serve as bottom formwork for casting the cap-beam wet joint. At other locations, corbels projecting from the bottom of the precast segments are used as bottom formwork for the wet-joint concrete. This arrangement reduces onsite formwork and accelerates concrete placement. Two alternative bottom-formwork configurations are shown in Figure 4.

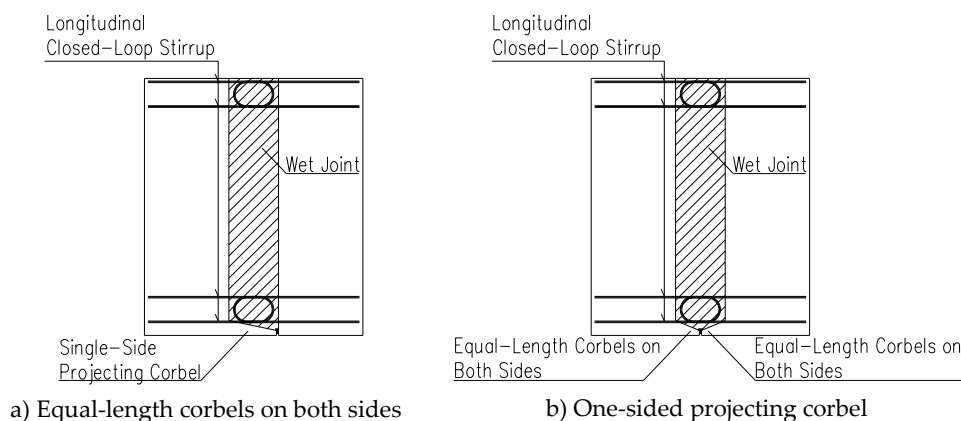


Figure 4 Bottom formwork configurations of the proposed cap beam

As shown in Figure 4a), equal-length corbels project from the bottoms of both precast cap beam segments, with a 2 cm gap between the two corbels. In Figure 4b), a corbel projects from only one of the two precast segments, while a 2 cm gap is likewise retained between the corbel and the other segment. The configuration in Figure 4a) allows identical formwork to be used for the two precast segments, but during erection, the two segments must first be assembled in the air using two cranes and then placed together at the predetermined position on the pier top. If the two segments are erected sequentially, the longitudinal closed-loop stirrups of the first segment would obstruct the installation of the second segment. Although the configuration in Figure 4b) does not allow identical molds for the two segments, the segments can be lifted and installed sequentially, significantly reducing erection difficulty compared with Figure 4a).

Therefore, the precast cap-beam configuration shown in Figure 4b) is adopted. A rubber strip is inserted into the reserved 2 cm gap at the wet joint, and the bottom is sealed with 10 cm-thick fine-aggregate concrete before the wet-joint concrete is cast, as shown in Figure 5.

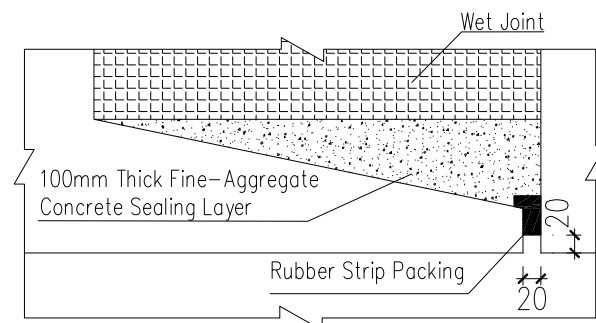


Figure 5 Bottom sealing detail of the wet joint (unit: mm)

4 Finite Element Model of the Cap Beam

As shown in Figure 6, a finite element model of the cap beam was established using Midas Civil. The longitudinal bridge direction is defined as the Y-axis, the transverse bridge direction is defined as the X-axis, and the vertical direction is

defined as the Z-axis. The precast segments and wet joint are modeled separately using beam elements and are connected by rigid links to act together. Rigid links are also used to connect the cap beam and columns, while concentrated soil springs are employed to represent the boundary conditions at the bottoms of the pile caps. The corresponding parameters are listed in Table 1.

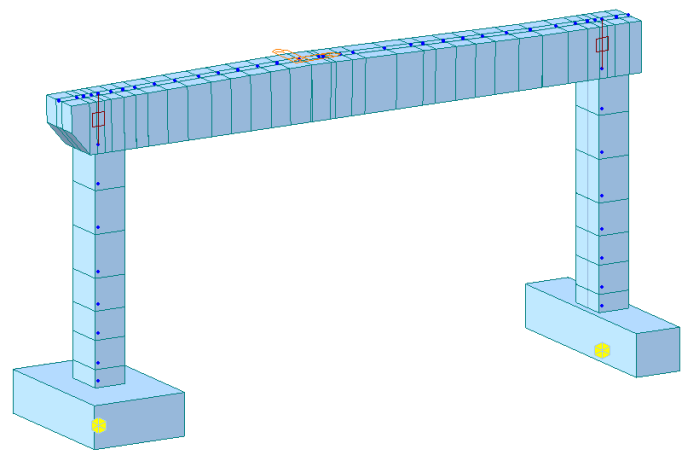


Figure 6 Finite element model of the proposed cap beam

The principal loads included in the cap-beam model are summarized in Table 2. With the exception of the transverse wind load, which is applied to the cap beam as a uniformly distributed load, the remaining loads, including the first- and second-stage dead loads, are applied as concentrated loads at the bearing locations. Lane loading is applied to the cap-beam elements in the form of an equivalent single-lane load, with the multiple-lane reduction factor taken into account. Uniform temperature variation and differential settlement of the overall structural system are also considered. The analysis explicitly accounts for the construction sequence, including the wet weight of the wet-joint concrete carried by the precast segments during casting and the structural-system transformation after the columns and cap beam become rigidly connected.

Table 1 Concentrated soil-spring stiffness

Location	SDx (kN/m)	SDy (kN/m)	SDz (kN/m)	SRx (kN·m/rad)	SRy (kN·m/rad)	SRy (kN·m/rad)
Left pier	269,500	246,600	3,092,000	19,710,000	6,152,000	2,298,000
Right pier	162,300	123,300	1,589,000	14,760,000	1,459,000	1,651,000

Table 2 Model load summary

Load type	Stage-I girder load (kN)	Pavement (kN)	Barrier + noise barrier (kN)	Longitudinal wind W1 (kN)	Transverse wind W2 (kN/m)	Single-lane axle load (kN)
Original scheme	7,976	3,078	1,239	318	38.5	635

5 Cap-Beam Design Schemes

The design adopts the lifting weight of each precast cap-beam segment and the amount of on-site wet-joint construction as the control indices. The wet-joint width

is determined by the minimum working space required for construction, whereas the precast-segment width is selected according to the anchorage space required at the tendon ends and the structural demand. The elevation profile of the prestressing tendons is shown in Figure 7a). Two tendon layout schemes are considered: (1) tendons arranged only within the precast segments and (2) tendons arranged in both the precast segments and the wet joint. The advantages and disadvantages of the two schemes are evaluated and compared with those of the original scheme.

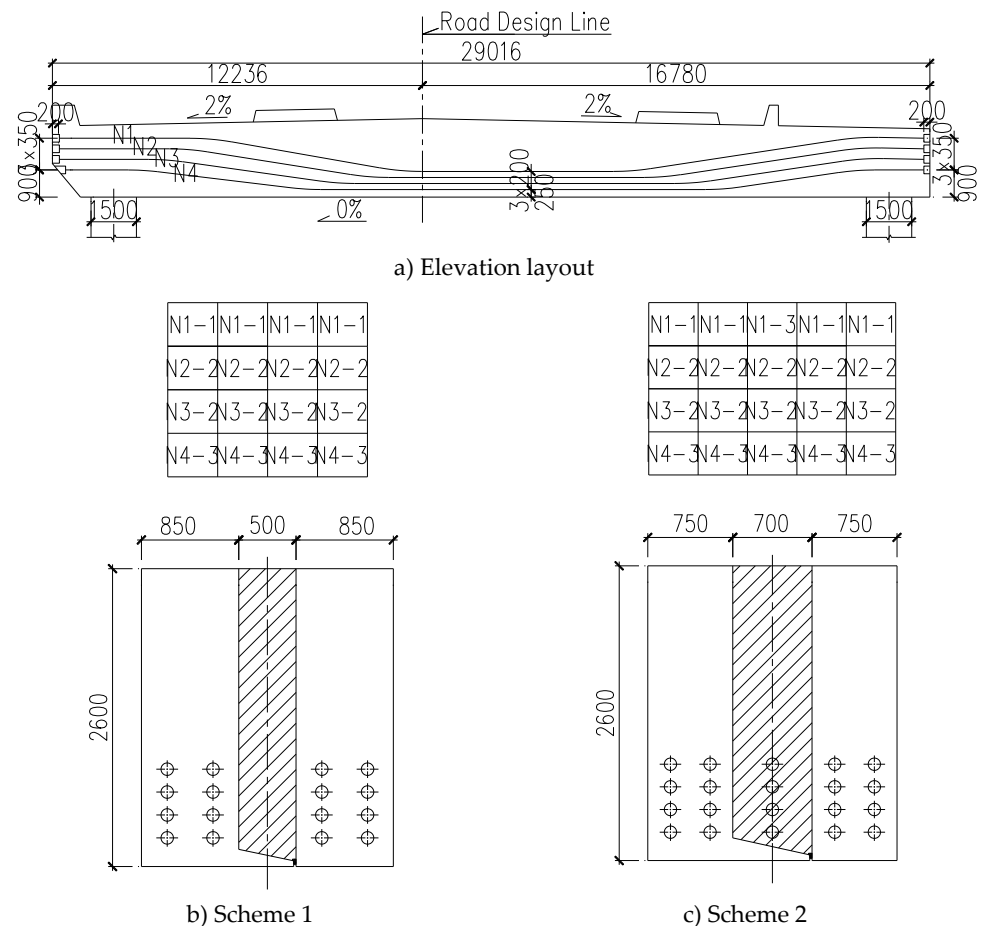


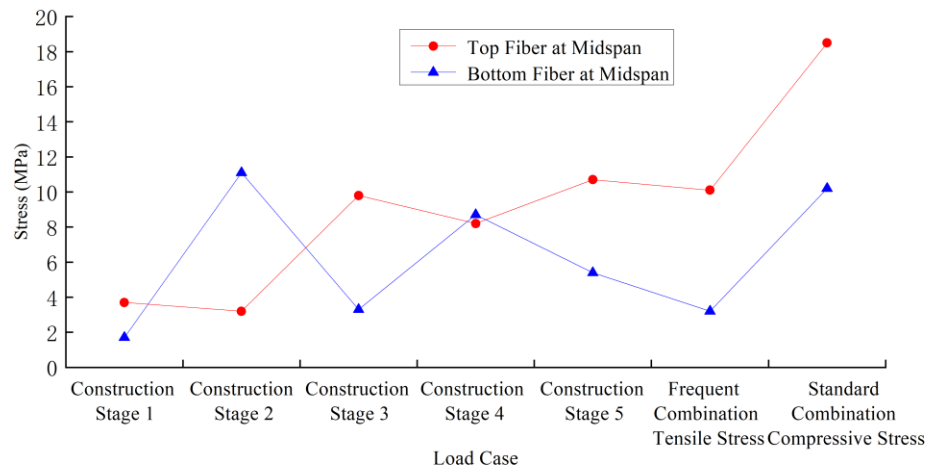
Figure 7 Prestressing tendon layout (unit: mm)

5.1 Scheme 1

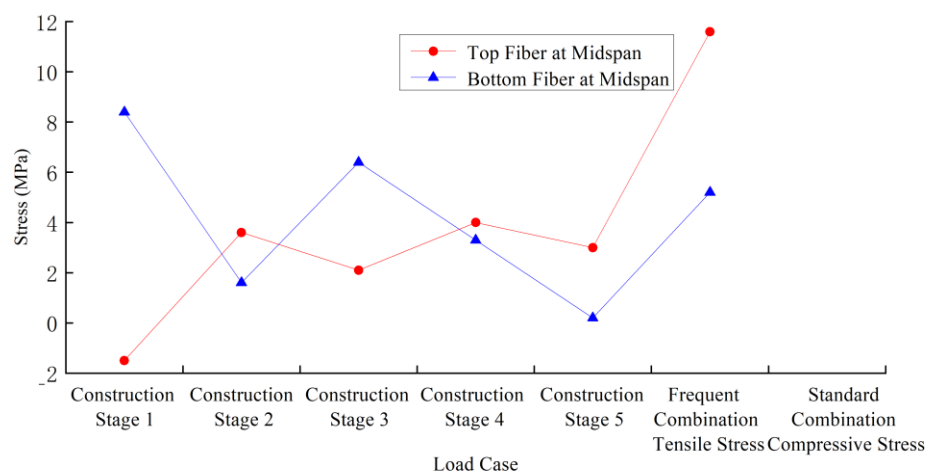
As shown in Figure 7b), Scheme 1 arranges the prestressing tendons only within the precast segments. The wet-joint width is set to 500 mm to provide the minimum space required for reinforcement installation within the joint. Because a certain compressive-stress reserve is also required in the cap-beam wet joint, some of the prestressing tendons can be tensioned only after the wet-joint concrete has been cast and after the two segments have formed an integral member. For comparison, the total cap-beam width in Scheme 1 is kept at 2,200 mm, which is the same as that in the original scheme, and each precast segment is 850 mm wide.

Scheme 1 uses prestressing tendons composed of 19 ϕ s15.2 mm strands, with the elevation profile identical to that in Figure 7a). The construction sequence is as follows: (1) after the precast cap beam is fabricated, the first batch of N1-1 tendons is tensioned; (2) after the wet-joint concrete is erected and cast to form an integral cap beam, the second batch of N2-2 and N3-2 tendons is tensioned, at which point a compressive-stress reserve begins to develop at the lower edge of the wet joint; (3) the superstructure main girders are then erected; (4) the third batch of N1-3 and N4-3 tendons is tensioned; and (5) ancillary structures are constructed. The tensile stress

under the frequent combination and the compressive stress under the standard combination are then calculated. The resulting midspan stresses of the cap beam are shown in Figure 8, where compression is considered positive and tension is considered negative.



a) Precast segment



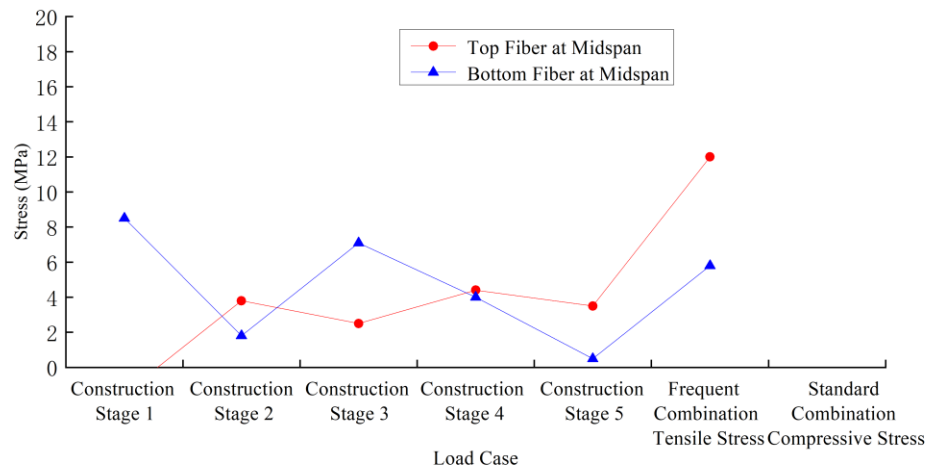
b) Wet joint

Figure 8 Key stress results at the midspan section for scheme 1 (unit: MPa)

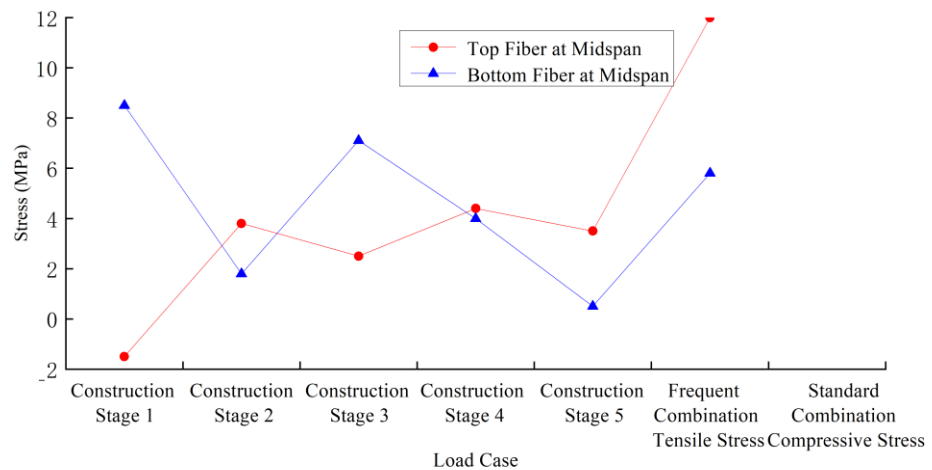
5.2 Scheme 2

As shown in Figure 7c), Scheme 2 arranges the prestressing tendons in both the precast segments and the wet joint. Because tendon positioning and reinforcement construction within the wet joint involve considerable workload and difficulty, the wet-joint width is set to 700 mm, and only one row of prestressing tendons is arranged within the joint. For comparison, the total cap-beam width in Scheme 2 is also kept at 2,200 mm, which is the same as that in the original scheme, while each precast segment is 750 mm wide.

Scheme 2 uses prestressing tendons composed of 15 ϕ s15.2 mm strands, with the elevation profile identical to that in Figure 7a). The construction sequence is the same as that for Scheme 1. The midspan stress results for the cap beam and wet joint are shown in Figure 9, where compression is considered positive and tension is considered negative.



a) Precast segment



b) Wet joint

Figure 9 Key stress results at the midspan section for scheme 2 (unit: MPa)

5.3 Comparative Analysis

The cap-beam stress levels of the two schemes are generally similar. The compressive stress under the standard combination is close to the code-specified upper limit of 19.25 MPa, whereas the lower edge of the wet joint remains in compression under the frequent combination. These results indicate that the dimensions of the precast segments and wet joint, together with the tendon arrangements, are economically reasonable and leave little room for further optimization.

In Scheme 1, the prestressing tendons are arranged only within the precast segments; therefore, the number of strands per tendon must be increased to $\phi 15.2$ –19. This arrangement reduces the dimensions of the precast segments while maintaining an adequate compressive-stress reserve in the wet joint. If the number of strands were further increased or if the precast-segment width was further reduced, the compressive-stress reserve in the wet joint would increase, but the compressive stress in the precast segments under the standard combination would exceed the code limit. In Scheme 2, part of the prestressing tendons are arranged within the wet joint, which effectively reduces the dimensions of the precast-segments while maintaining an adequate compressive-stress reserve in the wet joint. Although Scheme 2 requires more cast-in-place work, it is superior to Scheme 1 in terms of increased weight and tendon consumption.

The key design parameters of the longitudinally split cap-beam schemes and the original transversely segmented scheme are summarized in Table 3. Taking the original scheme as the baseline, four important design parameters are compared: precast concrete volume, cast-in-place concrete volume, single-lift weight, and tendon weight.

Compared with the transversely segmented cap-beam scheme, the longitudinally split schemes require 17–19% more prestressing tendons to ensure an adequate compressive-stress reserve in the wet joint. Nevertheless, they eliminate temporary support and minimize the impact of cap-beam construction on under-bridge traffic. Although the amount of cast-in-place work increases, the precast concrete volume can be reduced by up to 28%, and the smaller precast segments reduce the single-lift weight by as much as approximately 32%.

Table 3 Comparison of key design parameters

Scheme	Precast concrete /(m ³)	Ratio	Cast-in-place concrete /(m ³)	Ratio	Single-lift weight /(t)	Ratio	Tendon weight /(kg)	Ratio
Original	131.2	100%	8.2	100%	185	100%	8,510	100%
Scheme1	107.7	82%	31.7	390%	143	77%	10,128	119%
Scheme2	95	72%	44.4	540%	126	68%	9,995	117%

Compared with the transversely segmented scheme, the longitudinally split cap-beam scheme requires more prestressing tendons and therefore slightly more material. It also requires a larger amount of cast-in-place work, with an average construction period of 7–8 days compared with 4–5 days for the transversely segmented cap beam scheme; thus, it offers no advantage in construction duration or direct project cost. For major transportation corridors such as the Pudong section of the Shanghai Outer Ring Road, however, maintaining traffic beneath the bridge during construction is critical. A transversely segmented cap beam generally requires 9–10 days of traffic diversion, causing substantial traffic disruption and considerable indirect costs. In contrast, the longitudinally split cap beam substantially reduces the single-lift weight and eliminates temporary-support measures, thereby maintaining normal under-bridge traffic operation and safety during construction, avoiding the potentially enormous costs associated with traffic diversion, and reducing major safety risks. Overall, the longitudinally split cap beam exhibits better engineering applicability for urban viaduct projects.

6 Conclusions

With respect to the large-span two-column portal-frame concrete cap beams commonly used in urban viaducts, this study investigated a system in which the cap beam is longitudinally divided into precast segments and subsequently assembled on site. The structural details and connection methods at key locations were examined, and two design schemes were analyzed and compared with the original scheme to evaluate their feasibility and range of engineering applicability. The following conclusions are drawn:

- (1) The longitudinally split precast segmental cap beam is fabricated as longitudinally divided precast segments. After erection, wet-joint concrete is cast to form an integral cap beam. Unlike the commonly used transversely segmented precast cap beam, this system eliminates temporary construction support, preserves the clearance beneath the bridge, and minimizes disruption to under-ridge traffic.
- (2) Closed-loop stirrups are embedded in the precast cap beam segments, and after erection, the segments are connected to the column reinforcement using grouted sleeves. The closed-loop stirrups are then welded on site, transverse through-reinforcement is threaded through them, and wet-joint concrete is cast using

specially designed corbels as bottom formwork, thereby providing a reliable connection.

- (3) In Scheme 1, all the prestressing tendons are arranged within the precast cap-beam segments. Although this scheme requires less work in wet joints, the single-lift weight is relatively high. In Scheme 2, some prestressing tendons are arranged within the precast segments, and the remainder are within the wet joint. This arrangement increases on-site work in the wet joint but reduces the single-lift weight. The results show that wet-joint width is strongly coupled with single-lift weight, available construction space, and tendon arrangement, and is therefore a key parameter governing the scheme's applicability.
- (4) Compared with the transversely segmented precast cap beam, the longitudinally split precast segmental cap beam is less favorable in terms of tendon consumption and the amount of cast-in-place work. Nevertheless, it substantially reduces the single-lift weight, eliminates support measures, and ensures normal under-bridge traffic operation during construction, increasing its engineering applicability in urban viaduct projects.

Conflict of Interest Statement: The author declares that there is no conflicts of interest regarding the publication of this paper.

Data Availability Statement: The data that support the findings of this study are available from the corresponding author upon reasonable request.

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