Precast segmental box girder bridges with external prestressing
- design and construction -
Prof. Dr.-Ing. G. Rombach
Technical University, Hamburg-Harburg, Germany

Summary
Segmental box girder bridges externally post-tensioned are one of the major new developments in bridge engineering in the last years. In contrast to 'classical' monolithic constructions a segmental bridge consists of "small" precast elements stressed together by external tendons (fig. 1). The many advantages of this type of structure like fast and versatile construction, no disruption at ground level, high controlled quality and cost savings have made them the preferred solution for many long elevated highways, especially in South East Asia (see [1], [2]), and bridges. Design and construction of precast segmental hollow box girder bridges will be mentioned in this paper.

1 Introduction
The greatest segmental bridges had been build in South East Asia resp. Bangkok. This region of the world suffers under a big lack of sufficient infrastructure e.g. roads. In the big cities like e.g. Bangkok the traffic nearly collapsed. There is a great need to change this bad situation rapidly. A Master Plan had been developed for the Bangkok region which lead to many big train and highway projects (table 1).

<table>
<thead>
<tr>
<th>Table 1 Projects in Bangkok</th>
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<tbody>
<tr>
<td>Name of Project</td>
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<tr>
<td>Hopewell (SRT-CT, BERTS)</td>
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<tr>
<td>SST</td>
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<tr>
<td>Ramindra Asmarong Expressway</td>
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<td>Second Stage Expressway System</td>
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<td>Bang Na Expressway</td>
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<td>Sector C+</td>
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Fig. 1 Segmental bridge under construction

Table 2 Restraints

<table>
<thead>
<tr>
<th>Conditions</th>
<th>Solution</th>
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<tbody>
<tr>
<td>no space at grade</td>
<td>==&gt; elevated highway</td>
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<tr>
<td>traffic jams</td>
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<tr>
<td>flooding</td>
<td></td>
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<tr>
<td>bad soil condition</td>
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<tr>
<td>short construction time</td>
<td>==&gt; precast system</td>
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<tr>
<td>transportation problems</td>
<td></td>
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<tr>
<td>cost</td>
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<tr>
<td>flexible system</td>
<td>==&gt; segmental hollow box girder</td>
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Table 3 Second Stage Expressway System Part I, Sector B
2 Structural elements of segmental bridges

Segmental bridges are mainly built as single span structures to avoid coupling of post tensioning cables. Furthermore in single spans the greatest shear force is not located in the same section as the greatest bending moment. Though the joint between the segment is always closed. A typical span is shown in fig. 2.

A standard span has a length of appr. 45m. It consists of 14 segments. Dry joints are used in this project (no epoxy glue). No continuous reinforcement is provided across the match cast joints between the segments. Due to the external post tensioning (fig. 3) 3 different segments are needed (fig. 4):

- **Pier segment:** heavy end diaphragm required to stiffen the box section and for anchorage of p.t. cables
- **Deviator segment:** required to deviate tendons
- **Standard segment:** thin webs (35cm)
Figure 3 Tendon layout
D3 : 1090 - 1560 cm
D2 : 700 - 1190 cm

Pier Segment

Deviator Segment

Standard Segment

Shear Keys

Figure 4 Type of segments
3 Construction

3.1 Making of precast segments:

No free space between the segments is allowed. Therefore the segments are poured in line.

Two different methods are used to make the segments:
- Long line match casting method
- Short line match casting method

The short line match method is more flexible and needs less construction space.

Figure 5: Substructure

Figure 6: Short line match casting

3.2 Assembling of Segments
Figure 7: Assembling of segments

Figure 8: Overslung Truss
4 Advantages and Disadvantages of Segmental Bridges

Table 4 Advantages and Disadvantages of Segmental Bridges

<table>
<thead>
<tr>
<th>Disadvantages</th>
<th>Advantages</th>
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<tbody>
<tr>
<td>safety (e.g. in case of fire)</td>
<td>short construction time</td>
</tr>
<tr>
<td>extra cost (more prestressing required, single spans, truss)</td>
<td>(segments are prefabricated while the substructure is being built)</td>
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<tr>
<td>high construction loading (overslung truss)</td>
<td>no interruption of traffic</td>
</tr>
<tr>
<td>new construction method – technology</td>
<td>precast 'mass' production</td>
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<tr>
<td>(e.g. geometry control of segments, design)</td>
<td>- cost efficient</td>
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<td>- good, controlled quality</td>
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<td></td>
<td>- shapes</td>
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<td>weather independent construction (dry joints)</td>
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<td>small light segments</td>
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<td></td>
<td>hollow box section</td>
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<td></td>
<td>reduced dead load</td>
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<td></td>
<td>cost (reduced reinforcement)</td>
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<td></td>
<td>recycling</td>
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Table 5: Advantages and disadvantages of external prestressing

<table>
<thead>
<tr>
<th>Disadvantages</th>
<th>Advantages</th>
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</thead>
<tbody>
<tr>
<td>additional mild reinforcement required ( (\Delta \sigma_p) )</td>
<td>replacement of tendons possible</td>
</tr>
<tr>
<td>additional Cost for ducts, anchorage, etc.</td>
<td>inspection of tendons possible</td>
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<td>only straight tendon layout</td>
<td>easier Installation of longitudinal tendons</td>
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<tr>
<td>diffusion of post-tensioning forces</td>
<td>good corrosion protection of p.t. cables</td>
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<tr>
<td></td>
<td>less dead load (thin webs)</td>
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<td></td>
<td>pouring is facilitated (no p.t. ducts)</td>
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<td></td>
<td>less friction (no wobble losses)</td>
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<tr>
<td></td>
<td>prestress forces can be modified after construction (spare ducts)</td>
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<td></td>
<td>greater permissible p.t. stresses</td>
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5 Design
5.1 Longitudinal Design

General Requirements
The design of a segmental bridge has to be done for the serviceability and the ultimate limit state with the following distinctions to monolithic structures:

Serviceability Limit State
- Full prestressing – min. compressive stress 1 MPa
- Shear transfer in the joints

Ultimate Limit State
- Opening of the joints has to be considered
- Load transfer in the joints

Under service condition the concrete compression in the dry joints has to be greater than 1,0 – 1,4 MPa. Therefore the whole structure is under compression during normal loading. As there is no tension within the concrete, forces and moments can be calculated based on a linear elastic behaviour of the structure. In addition to monolithic bridges, all joints have to be designed for shear loads (ULS). The shear force is carried by shear keys and by friction between the joint surfaces.

Under ultimate loads the joints between the segments will open. The resulting decrease of the structural stiffness has to be considered in the design. This can be done by analytical (moment-curvature relationship) or numerical methods (finite element methods).

Critical sections
- midth of span greatest bending moment
- first joint after support greatest shear force but prestress force not uniformly distributed in cross-section
- diaphragms high concentrated loads due to anchorage of tendons
- deviators high concentrated loads due to tendons.

Numerical investigations
The load – deformation characteristic of a segmental construction is different from a monolithic one due to the dry unreinforced joints between the precast elements. Examinations of the behaviour of a segmental bridge and the forces in the joints finite element calculations had been conducted taking into account the non-linear behaviour due to the opening of the dry joints under tension. In contrast to known numerical investigations, the fine indentation of the joints had been modelled which is of great importance regarding torsion effects (fig. 9).

Figure 9 Finite element mesh of a segment

A real existing single span segmental bridge with external post-tensioning, a standard span of the elevated highway 'Second Stage Expressway System' in Bangkok [2] had been modelled (fig 2, 3, 9). This structure is used as data from a full-scale test [5] is available to verify the results of the complex numerical simulations. The opening of the dry joints is modelled by interface elements.
Fig. 12 shows the calculated moment-deflection curve which is typical for a single span segmental bridge with dry joints. At the beginning of loading the whole structure is under compression due to the high post-tension normal forces. Thus the structure behave like a monolithic one. The deflection increases linear with the load. At a midspan moment of $M \approx 37 \text{ MNm}$ due to live load the first joint near midspan starts to open rapidly resulting in a great decrease of stiffness. The lever arm of the inner forces keeps nearly constant. Thus the moment deflection curve is again nearly linear. The structure fails due to crushing of the concrete in the top slab. Nevertheless a ductile behaviour of the segmental bridge can be seen.

![Figure 12](image)

**Figure 12** *Comparison between full-scale test and numerical results*

Only 3 of 13 joints are open under failure load. Thus a great part of the bridge keeps under full compression.

Further shown in figure 12 are the results from a full-scale test carried out in Bangkok. A good agreement between the numerical results and the test data can be seen. This demonstrates that the finite element model is capable to model the real behaviour of a segmental bridge.
Several load combinations corresponding to bending, shear and torsion are examined to determine the stresses resp. the forces in the joint [6]. In a single span bridge the joints near the support are always closed due to the small bending moment. As the behaviour of an open joint is of main interest also a single span bridge restraint on one side with a modified tendon profile has been modelled.

Fig 13 shows the resulting shear forces in the first joint close to the support in the webs and the slabs due to torsion with increasing load. The results from three different numerical models are presented. The first one is a monolithic girder which behaves always linear. Further the shear forces for a segmental bridge with smooth and keyed joints are shown.

There are no differences between the models as long as all joints are closed. When the joint starts to open, the force in the top slab (tensile region) degrees. A great difference in the behaviour of a bridge with plain and keyed joints can be noticed. Smooth joints can only transfer forces when they are under compression whereas keyed joints can still transfer forces until a certain gap is reached. Even bigger differences can be seen in the webs. The plain joint reach the limit condition $\lim F_z = 0.7\sigma_y$ just after the joint opens whereas the force in the keyed joint still increases.

The results emphasise that the shear keys have a significant influence on the behavior of a segmental bridge under torsion loads. Calculations with plain joints are insufficient when torsion effects become significant.

![Figure 13 Forces in the webs and the slabs due to torsion](image1)

![Figure 14 Joint opening due to positive resp. negative bending moments](image2)
5.2 Design of segmental joints

There is a great uncertainty regarding the design of the joints between the segments (see fig. 18). This is surprising as the behaviour of the joint is of critical importance for the safety of a segmental structure.

The shear capacity of a keyed joint is a combination of the friction between the plain surfaces and the shear capacity of the keys. The latter one is neglected in the German regulations.

5.2.1 Existing design models

The joints of many segmental bridges had been designed according to the AASHTO Recommendations [4]. Equation 1 is mainly based on tests with small specimens having usually one shear key only [7] similar to that shown in figure 16 [8].

\[
V_j = A_{sm} \cdot \sigma_n \cdot f_k \cdot \left(12 + 2,466 \cdot \sigma_k\right) + 0,6 \cdot A_{sm} \cdot \sigma_s
\]

where:
- \(\sigma_n\): average compressive stress across the joint
- \(A_{sm}\): area of contact between smooth surfaces in the failure plane
- \(f_k\): characteristic concrete compressive strength
- \(A_{key}\): min. area of the base of all keys in the failure plane

According to the German recommendations for design of segmental bridges [3] only the frictional forces should be considered in the design. The load bearing of the shear keys is neglected as only epoxy joints can be used. Please note the difference between eq. (1) and (2) regarding the frictional area \(A_{sm}\) resp. \(A_T\).

\[
V_j = \mu \cdot A_T \cdot f_k
\]

where: \(A_T\): effective shear area

The results of both models will be discussed together with the proposed design concept in section 5.2.3.

5.2.2 Tests and numerical verification

To develop a design concept for the joints tests with specimens, similar to that described in [7] having one or multiple shear keys (fig. 16) were conducted to calibrate the finite element model. The study includes dry and glued joints. The dimensions of the shear keys are representative for segmental bridges. The non-linear material behaviour of the concrete like e.g. crushing and cracking and the interaction between the indented surfaces (bond, slippage, friction) has been considered in the numerical model.
Figure 16 Test specimen

The test specimens are first stressed normal to the joint and than loaded with a vertical force up to failure. Fig. 17 shows the experimental and calculated load-deformation curve. The behaviour of the joint and the ultimate load are well predicted. The highly complex concrete behaviour near the failure load has not been modelled as this region is not relevant for the load bearing capacity of a joint.

Figure 17 Test results versus numerical results for a dry and epoxy joint

5.2.3 New design model

After the verification of the finite element model, a numerical parametric study had been conducted with various number and shapes of shear keys, concrete qualities etc. [6]. The results lead to a design model that differs from the existing concepts. The shear capacity of a keyed dry joint \( V_{d,j} \) is a combination of a frictional and a shear part. For the first one the total area of the joint \( A_{\text{joint}} \) is used and not only the smooth parts \( (A_{\text{Sm}}) \) like in AASHTO recommendations. The load bearing capacity of the keys depends on the concrete tensile resp. compressive strength and the area of the failure plane \( A_{\text{key}} \).

for dry joints: \( V_{d,j} = \frac{1}{\gamma_f} (\mu \cdot \sigma_n \cdot A_{\text{joint}} + f \cdot f_{ck} \cdot A_{\text{key}}) \)  \hspace{1cm} (3)

where: 
- \( \mu = 0.65 \) coefficient of friction 
- \( \gamma_f = 2.0 \) safety coefficient 
- \( \sigma_n \) average compressive stress across the joint 
- \( A_{\text{joint}} \) area of the compression zone 
- \( f_{ck} \) characteristic concrete compressive strength 
- \( f = 0.14 \) factor for the indentation of the joint 
- \( A_{\text{key}} \) min. area of the base of all keys in the failure plane 
- \( h_{ne} \) height of keys, with \( h_{ne} \leq 6b_n \) 
- \( b_n \) width of the keys
The failure plane $A_{key}$ will have the least area of key breakage. A relatively high safety coefficient of $\gamma_f = 2.0$ should be used as the failure of the joint is brittle.

For glued joints only the frictional part can be used (eq. 4). Experiments showed a relatively small increase in strength of appr. 20% between a glued and a dry joint. Furthermore a sufficient quality of the glue can not be guaranteed on site.

\[
V_{d,j} = \frac{1}{\gamma_F} \cdot \mu \cdot \sigma_s \cdot A_{joint}
\]  

(4)

To compare the results of both models, the shear stress $\tau = \frac{V_{d,j}}{A_{joint}}$ is calculated for a standard segment of the segmental bridge in Bangkok [2]. The relevant joints are fully closed. The concrete compressive strength is $f_{ck} = 40 \, \text{MPa}$.

Fig. 18 shows the load bearing capacity of a keyed joint according to various design models. The great differences between AASHTO and the German regulations can be seen. The first model can not be used for high compressive stresses, which may occur near the ultimate design load of a multispan segmental bridge. Furthermore it seems to overestimate the load bearing capacity of a joint.

**Figure 18** *Comparison between different design models (standard segment [2])*
5.3 Types of joints

Figure 19  Types of Joints

Figure 18 Bang Na – Bang Pli Bang Pakong Expressway
REFERENCES


