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FIELD INVESTIGATION OF JOINTS IN PRECAST POST-TENSIONED SEGMENTAL CONCRETE BRIDGES

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Abstract. The precast segmental construction is widely used in the bridge structures erected by balanced-cantilever method. The joints are obviously of the utmost importance in connecting the precast segments of bridge girders. As well as the girders, the joints between segments should be capable of transferring compression, shear, and torsion forces. The basic requirements for match-cast joints are summarised. The joints between segments in many precast concrete bridges constructed by balanced cantilever method were inspected during construction and in service. The results of field survey that are presented in this paper included 5 major bridges. The spans are all precast box girders of constant or varying cross-section. Studies were conducted to determine the geometry and mechanical properties of grouted and match-cast joints. Classification of joint defects is presented. Statistics are given on the defects observed in some box-girder segmental joints. Photographs of typical joint problems are included.

Keywords: segmental concrete bridges, match-cast joints, erection requirements, defects.

1. Introduction

Segmental post-tensioned concrete box girder bridges, both cast-in-place and precast, were introduced in bridge construction during the post World War II reconstruction period. Numerous successful applications in this period can be found in almost all European countries as well as in North America. The development of precast segmental construction is generally attributed to France and former USSR. Several precast bridges and overpasses of this type were built successfully in Lithuania. The frame and continuous beam bridges, made of precast segments, were erected by the balanced cantilever method of construction using internal post-tensioning and mainly epoxy-jointing techniques.

The segmental construction was proposed on the basis of the following advantages: speed and ease of erection when compared to cast-in-place concrete; elimination of the falsework over waterways or when traffic must be maintained under the bridge during construction; better control of concrete strength, shrinkage and creep because segments are normally cured to full strength in the factory-controlled conditions. Probably one of the main factors encouraging the design of precast segmental bridges in Lithuania was also the shortage and high relative cost of the structural steel.

The main disadvantage of precast segmental construction is a high degree of accuracy and geometry control during fabrication and erection of segments. Quality of erection is essential. It is evident that the joints are obviously of the utmost importance in connecting the precast segments of bridge girders. Poor joint erection job will detract the performance of the structure as a whole.

At the same time, it is recognized that precast segmental box-girder bridges often show unexpected long-term deflections (Kamaitis 1995; Radic *et al.* 2004), web concrete cracking (Kamaitis 1996; Okeil 2006), and prestressed tendons corrosion (Podolny *et al.* 2001; Wouters *et al.* 1999). Long lasting deformations and concrete cracking mainly are related with the serviceability limit state. Prestressing steel corrosion which is difficult to provide good monitoring and inspection for post-tensioned structures can have the disastrous consequences. All these disadvantages are mainly related to the specific of precast segmental construction and the complex time-dependent behavior of the concrete.

The problems with precast segmental bridges can be related also to poor construction practice of the joints between segments. Joints are either wide or match-cast. Wide joints are cast-in-place with concrete. Match-cast joints are normally bonded with epoxy (sometimes with high strength mortar or grout) or are dry. Frequently single or multiple shear keys are used to resist the shear forces. The match-cast joints and particularly the epoxy joints, however, are mostly used. Epoxies in the joints between precast segments are being employed as a stress distribution bonding/sealing layer. As well as the girders, the joints between segments should be capable of transferring axial loads, shears, moments, and torsions from member to member.

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The major factors determining the integrity of joints are selection of appropriate adhesive, joint configuration, adhesive layer thickness, preparation of the bonding surfaces, and strict quality control in bonding operations. Not hardened epoxy glues were found in the joints after failure of 63 m long precast segmental continuous bridge superstructure in Russia in 1987 (Kamaitis 1995). The collapse of the footbridge in Hampshire in 1967 (after 15 years in service) and the bridge in West Glamorgan in 1985 (after 32 years in service) can also be mentioned (Woodward, Williams 1988). Both structures were segmental construction with thin mortar joints permeable to moisture and chlorides. The collapse of the bridges was due to corrosion of longitudinal tendons at the segmental joints.

Numerous cases of prestressing tendons corrosion of segmental bridges in European countries are reported in Podolny *et al.* (2001). For instance, in 1992, a bridge in Belgium collapsed without warning as a result of corrosion of post-tensioning tendons. There was considerable loss of life. Similar problems related to segmental bridges including and segmental joints were being identified in USA and Japan. Corrosion of steel tendons mainly is due to poor construction quality and ingress of water with chlorides. Not completely hardened epoxy, imperfect sealed and leaking segmental joints and failed tendons also were revealed in a number of segmental bridges.

As a result of recent findings of condition state of segmental bridges built in USA and Europe, stricter requirements for design, detailing, inspection and maintenance are adopted. In particular the AASHTO *Guide Specifications for Design and Construction of Segmental Bridges* published in 1999 prohibits the use of internal tendons in dry joints. External unbounded tendons are easier to inspect and protect from corrosion and simpler to replace (if damaged). In some states the dry joins in general are not recommended. The standards in USA require the inspection of all bridges with reports of bridge condition ratings at least once every 2 years. The inspection frequency ranging from 1 to 6 years is adopted in many European countries.

These failures provide an important lesson in designing and erection of segment joints. It should be pointed out that little (if no) attention is paid during standard inspections on the state of segmental joints of bridge stock in Lithuania. It is essential to consider joint problems and their ramifications at all stages of their life, from conceptual design through construction and under service conditions.

This rapport is based upon a survey of segmental box-girder bridges in Lithuania. The purpose of this paper was to illustrate and point out the cause of instances of segmental joint condition state which are all too common. It is hoped that this presentation may afford more general recognition of a condition which contributes to condition state of segmental joints and thereby be helpful to bridge inspectors and designers.

2. Principal requirements for match-cast joints

Since 1990 the author has had considerable experience in inspection, testing and retrofiling of various type bridges. He has also a good opportunity to observe glued joint problems. This experience has led to a joint philosophy which favours joint erection requirements. Detailed requirements for match-cast joints in national design guidelines or standard procedures for segmental structures are lacking. In selecting the match-cast joint type, their design and erection the following considerations should be taken into account:

- the type and configuration of the joint and jointing material shall be carefully selected to provide the erection tolerances allowing correction in the alignment of the superstructure's longitudinal and transversal profile;
- the bearing capacity of the joints had to fit with that of monolithic structures; the long-term behaviour of joints should be guaranteed;
- preferably the single or multiple keys in the webs would be erected, in order to facilitate alignment and to transfer shear during erection;
- the strength of the joint filling material should be not lower and the deformability not higher than that of the concrete in adjacent segments;
- the joint width should be as low as possible, in order to provide min influence of the joint on the over all behaviour of the segmental structure; the strength of the epoxy joints is higher as the joint thickness is lower;
- for all types of joints the faces of adjacent segments before joining operations must be dry, clean, free from grease, oil or other contaminates;
- the post-tensioning shall be applied when the joint filling material reaches sufficient (design) strength; in epoxy joints after application of the epoxy adhesive the uniform glue pressure (at least 0.2 MPa) between faces at the joint is required;
- there shall be no tensile stresses in extreme fibers of the joints when all possible design loads (including the effect of prestress) are acting; for full prestressing min compression of 1 MPa is recommended;
- preferably the joints should be erected at positive ambient temperatures; epoxy joints should be heated (at least to +5 °C) to cure properly, if the erection of superstructure continues through the winter.

Unfortunately, not all of the joining systems presently being used meet the above requirements. To satisfy the above requirements, the design and quality erection of segmental joints should be carefully controlled.

3. Status of segmental bridges in Lithuania. Classification of defects

Segmental precast post-tensioned structures were first used in the early 1964 for construction of bridges and highway overpasses in Lithuania. The first application was the three-span frame bridge with dry joints. The Žirmūnai Bridge of 1965 (Fig. 1) is one of the first segmental structures having wide concrete joints. The bridge is in a location where it can be seen from all sorts of viewpoints and appearance was one of the important considerations. Since then, more than 30 structures have been built.

Fig. 1. One of the first PC precast segmental bridge with concrete joints over Neris River (bridged in 1965)

The bridges over water with spans of up to 100 m and interchange highway viaducts (Fig. 4) on land sides of the typical frame system with the main span of 48 m were designed and constructed from precast concrete segments, as a rule, with epoxy resin joints.

During erection deviations of segments from the desired profile of superstructures are observed that is probably inevitable. From time to time the alignments of segments were adjusted to compensate for erection errors by using cast-in-place concrete (microconcrete) without reinforcing bars extended across the joints. All bridges are continuous or frame box girders of constant or variable depth, post-tensioned with internal grouted tendons. A few bridges have structural hinges at the middle of the largest spans. Generally, all bridges are constructed by balanced-cantilever method.

In more recent years, some problems have been reported, particularly web cracking in the box girders, excessive long-term deflections of superstructures and corrosion of some post-tensioned tendons with an inherent loss in serviceability, durability and long-time safety. As was mentioned above, the problems of precast segmental bridges are mainly related to the specific of precast segmental construction and the complex time-dependent behaviour of the concrete. Probably many of these distresses are also a result of inadequate attention to joints.

Unreinforced joints between the segments can have a critical importance for the behaviour of precast segmental structures. It should be stressed that improper consideration for design, construction and maintenance leads to reduced performance of segmental bridges.

Many prestressed concrete bridges were inspected during construction and in service with regard to condition state of grouted and match-cast joints between precast segments. Geometrical parameters, material properties, and defects of joints were investigated. The main defects and their causes of the joints between precast segments are classified and presented in Table 1.

Table 1. Main defects and their causes of segmental joints

Defects	Causes
Epoxy adhesive not fully hardened or slow gaining strength	• poor quality of components • errors in formulation, poor mixing and hardening • seepage of water (rain) during application of adhesive • slow hardening of the epoxy in low temperatures
Porosity and inadequate strength of concrete in the joints	• inadequate compaction and improper curing • shrinkage of joint concrete • low spacing between segments
Variable joint width or not completely filled joint	• deviation of segments from desired profile • not uniformly placed and compacted concrete on hand-coated segment surfaces • rain during application of adhesive • not uniform contact pressure during prestressing of tendons • penetration of joint filling material into the ducts
Opening of the joint	• insufficient bond strength between joint filling material and concrete (improper cleaning of concrete surfaces, presence of laitance, excessive moisture) • permanent prestressing losses • overloads
Spalling or cracking of concrete surrounding the joints	• errors during erection operations • absence of full contact between bearing areas and not uniform distribution of bearing forces
Leakage through the joint	• poor waterproofing of bridge deck • not completely filled joint • joint filling material is permeable to moisture

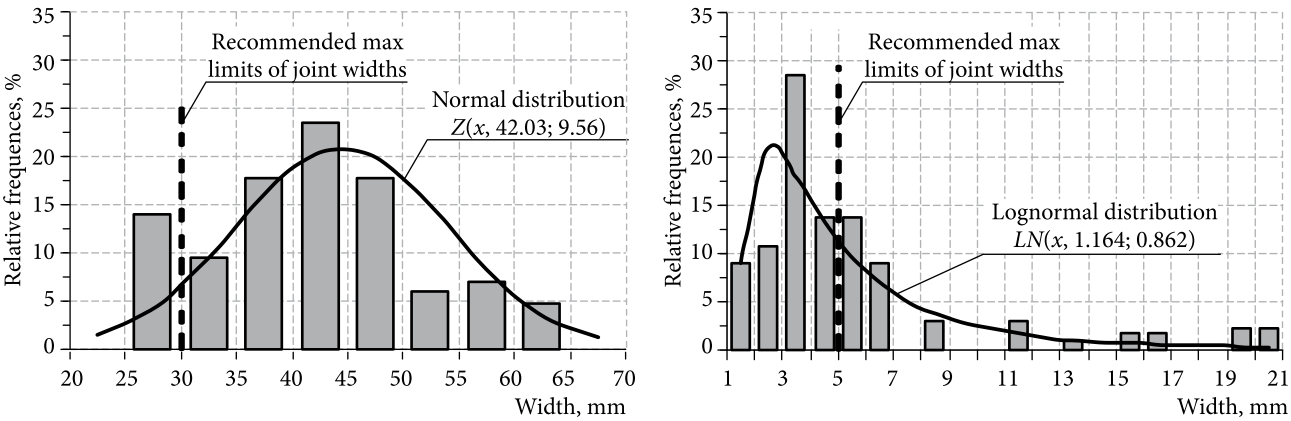
4. Results of field investigations

The field survey included 5 major bridges (Table 2).

From 1993 repeated visual and instrumented inspections for each bridge were made. There was an evidence of excessive deflections and web cracking of box girders as well as tendons corrosion on any of the inspected bridges, as was mentioned above. Because of space limitations not all survey results for every bridge are included in this pa-

Table 2. Data of inspected bridges

Bridge No.	Type and spans of the bridge	Opened to traffic	Type of joints
1	Frame 20.7 + 100 + 20.7 m	1965	Wide concrete
2	Frame 50 + 100 + 50 + 29.7 m	1969	Epoxy + grouted
3	Continuous beam 62 + 100 + 62 + 3 × 36.5 m	1972	Epoxy + grouted
4	Continuous beam 30 + 40 + 92 + 40 m	1979	Epoxy + grouted
5	Continuous beam 42 + 84 + 42 m	1997	Epoxy + grouted



a b

Fig. 2. The distribution of the joint widths for cast-in-place (a) and epoxy (b) joints

per. In the next sections we will present some typical examples of each kind of defects observed. Note that similar results of condition state of joints are obtained approx in all inspected bridges.

4.1. Width of joints

As an example we present the survey data for bridge No. 5. The number of investigated joints was 336, including 324 epoxy and 12 cast-in-place concrete joints.

Fig. 1 shows the histograms of joint widths. This distribution is fitted with a normal distribution for cast-in-place joints and lognormal distribution for epoxy joints. Mean of 42.03 mm and variance of 9.56 mm were found for grouted joints. For epoxy joints the mean is 4.93 mm and variance is 33.26 mm. The max epoxy joint thickness is about four times and that of cast-in-place joints about two times of the prescribed by some codes or recommendations (Table 3). Probability of thicker joints over the recommended by codes and guides is very large.

Table 3. Max widths of segmental joints (no reinforcing bars extended across the joints)

Standards and recommendations	Max thickness	
	match-cast joints, mm	grouted joints, mm
DIN 4227, part 3E (Germany)	4	4
CHBDC (Canada)	3–4	3–4
SNiP 2.05.03-84 (Russia)	5	30
PCI recommendations (USA)		25
AASHTO (USA)	2	

4.2. Properties of joint filling material

The strength and the deformability of the joint filling material should be identical to that of the concrete in the adjacent segments, and this factor may be critical design consideration for the structure.

Standard cub specimens were normally cast at the concrete plants to control the segments concrete quality and on the construction sites to control the joint grout and epoxy glue properties. The strength of concrete in the segments was also determined with the Schmidt hammer on the construction site just before moving the precast segments to their position and post-tensioning operations. The evaluated concrete strength in precast segments and cast-in-place joints took into account the variation in material properties, as well as uncertainties due to casting, compacting and curing of cast-in-place joints.

As an example, the test results for a bridge are presented in Table 4. The strength of the concrete of two bathes (I and II) in the joints is lower than that of the concrete of segments and this factor may be critical for structures safety. Mean of 52.4 MPa and COV of 12.9% were found for concrete of arbitrarily selected 10 precast segments. For precast joints mean of 38.4–28.8 MPa and COV of 15–17% were observed. In addition, the strength of cast joints was of 47–49 MPa at the level of the top slab and 16–30 MPa in the webs of box girders. Note that clear spacing between segments is to low to permit good compaction of cast concrete. The joints were cast in simple suspended formworks. Not completely filled joints and porous joint concrete can be also mentioned. All concreted joints of 20–50 mm in width were found to be very difficult to achieve uniformly good workmanship.

Of primary interest with regard to serviceability behaviour of segmental structures under loading, including prestressing, is the relation between deformability of joint material and that of segmental concrete. The short-term and long-term modulus is employed in the deflection analysis of concrete structures.

Table 4. Strength values of concrete of the segments and joints

	Precast segments		Joints	
	plant after 28 days	field after ~6 months	after 28 days (I)	after 28 days (II)
n	22	22	11	11
$\overline{X}$, MPa	43.9	52.4	38.4	28.8
±σ, MPa	6.42	6.78	6.30	4.90
COV, %	14.6	12.9	15.7	17.0
E_{cm}, MPa	34290	36160	32940	30220

The short-term modulus for normal or high strength concrete was determined by the empirical equation (Henry, Smith 2007):

$$E_{cm} = 22 \times \left(\frac{f_{cm}}{10} \right)^{0.3}, \quad (1)$$

where E_{cm} – the short-term modulus for normal or high strength concrete, GPa; f_{cm} – mean compressive strength in 28 days, MPa.

The results of computation are presented in Table 4. The ratio between short-term modulus of joint and segment concrete is approx 0.84. It follows that deflection (curvature) of the joint is approx 16% higher than that of the adjacent segment sections.

Long-term deformations, which generally are of primary importance, are due principally to the effects of creep. The long-term modulus

$$E_{cm}(t) = \frac{E_{cm}}{1+\Theta_t}. \quad (2)$$

The creep factor may be determined as follows:

$$\Theta_t = \beta(f_{cm}) \times \beta(t_0), \quad (3)$$

where $\beta(f_{cm})$ – a factor of concrete strength; $\beta(t_0)$ – a factor of concrete age t_0 at loading.

Taking into consideration both factors for precast segments, we obtain $E_{cm}(t)$ = 19 120 MPa and for the joint concrete $E_{cm}(t)$ = 11 960 MPa. Hence, time dependent modulus ratio is of 0.63.

Let us consider now the epoxy joints. During erection of all bridges epoxy adhesives have been tested in flexion, in tension and transverse shear of bonded concrete prisms with notation of whether failure occurs in the joint or in the concrete. It was obtained that flexural strength is of the order of 240 MPa and flexural modulus is of 6000 MPa. It seems that strength of epoxy compound is higher than that of the segment concrete. However, the deformability is much higher of polymer compounds. The ratio of the short-term modulus is only of 0.16.

The tests of bonded concrete prisms demonstrated that tension bond of 3 MPa and shear bond of 1.9 MPa and greater is readily obtained. However, sometimes only 15% of tests specimens (total number is 20) were capable of developing full tensile strength of the concrete. Rupture occurred in the adhesive or by stripping of the adhesive from the concrete. It was also observed that the variance of joint thickness could have considerable influence on the bearing capability of bonded prisms as well as segmental structures. The strength of the epoxy joints is higher as the joint thickness is lower. The extensive tests of precast segmental joints showed that an epoxy thickness of 1 or 2 mm gives a greater shear strength then those of 3 mm (Zhou *et al.* 2005). Based on this, it could be concluded that a wide scatter in data of polymer compounds can be obtained in the field with different batches. Good quality control on product and application, however, is required for obtaining good joints in the field.

The low short-term and long-term modulus as well as shrinkage and creep of insufficiently compacted joint filling material can lead to very high deformations during post-tensioning and exterior loading of the completely segmental structure. For instance, it has been shown that shrinkage deformations are one of major causes of defects in bridge structures (Gribniak *et al.* 2007; 2008). Note, that the curvature along span of loaded segmental structure is not the same and varies from section at joints to section at segments.

Let us now consider a unit consisted of 2 adjacent segments of length, l_s, with a single joint of thickness, δ_j, subjected to bending (Fig. 3). Applied to the segment surface due to adhesion between joint filling material and segment concrete, joint is restraint to lateral deformations and the interface stresses inevitable appear. The concrete segments with highest E_c arrest the joint strains. An adverse effect of the joint on segments should be noted. The joint with surrounding segment concrete are in multiaxial state of stresses. These stresses also increase the deformability of segment concrete around the segment joint. Suppose that a is the distribution width of the joint influence zone. Based on the above definition, it is suggested that different flexural rigidity of the joint, joint influence zone and rest of the segment should be considered as is shown in Fig. 3. The flexural stiffness of the segment, joint and influence zone can be computed using the relationships:

$$\left.\begin{aligned} B_s &= E_{cm}(t) I_s; \\ B_j &= E_{jm}(\delta_j; t) I_j; \\ B_s^* &= 0.5(B_s + B_j), \end{aligned}\right\} \quad (4)$$

where $E_{cm}(t)$ and $E_{jm}(\delta_j;\, t)$ – the age-adjusted modulus of segment concrete and the joint, respectively, MPa; I_s and I_j – the moments of inertia of transformed segment and joint sections, respectively, Nm.

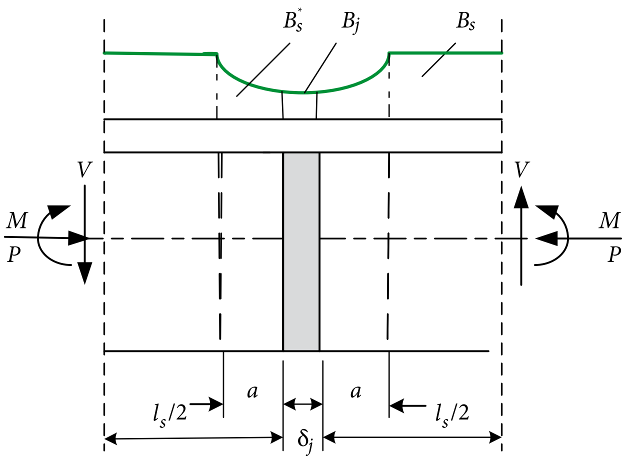


Fig. 3. Idealized model for segmental joint and definition of the joint rigidity problem to be solved

Based on the above discussion, it can be predicted that the behaviour of segmental structure (rigidity, strength, failure mode) will depend on the relative strength and modulus of deformation of the constituent materials. The axial strains and the curvatures not uniformly distributed along the span may be determined by virtual work or by other methods of structural analysis and is beyond the scope of this paper. The further research is conducted to account the influence of segmental joints on the flexural behaviour of segmental structures.

From the values of limited number of tests presented in this section, it is apparent that uniformity of segment concrete and joint filing material properties can be essential for segmental construction.

4.3. Tightness of joints

The joints should be tightly sealed in order to sustain the uniform pressure and to prevent water and salt penetration through the joints into the tendon ducts when the waterproofing of bridge deck is not effective. Hence, at joints where discontinuities in concrete cover and ducts are present the epoxy and grout should provide corrosion protection of the tendons.

The results of survey showed that serious problem is leakage of joints to moisture and chlorides from the bridge deck causing prestressed tendons and anchors on the joints faces of the precast segments corrosion. The results of the inspections for inspected bridges showed that the joints appeared to have some leakage along with leaching coming from the top slab. Fig. 4 is a view of the underside of this joint. Schematic representation of the effect of leaching is also shown. Here water draining from the deck has found its way through the transverse joint followed by dissolving calcium hydroxide at joint locations with the result of staining, efflorescence, encrustation and stalactites.

The leaking joints in all bridges represent approx no more than 4% of the total number of inspected epoxy joints. However, very intensive leakage of joints is observed in the exterior girders (Table 5). The number of leaking joints in these girders can achieve approx 30%. In exterior girders of two bridges, the active corrosion and rupture of some wires of tendons were observed (Table 5). All these bridges have been repaired and strengthened.

Table 5. Number of leaking joints in the bridges

Bridge No.	Total No. of bridge joints	Percent of leaking joints in the edge girders	Observed rupture of tendons single wires
1	128	15.6	not yet
2	400	16.7	yes
3	488	26.3	yes
4	300	33.3	not yet
5	336	8.3	not yet

Significant corrosion of prestressed tendons was also noted in some spans of highway viaducts. To prevent the probable collapse of the whole structure, shoring technique was used (Fig. 5). However, it is not the best solution. It should be pointed out that serious durability problems may occur in the future with tendons in these bridges because of the difficulty of detecting and quantifying any deterioration. Corrosion of prestressed tendons can lead to dramatic failures.

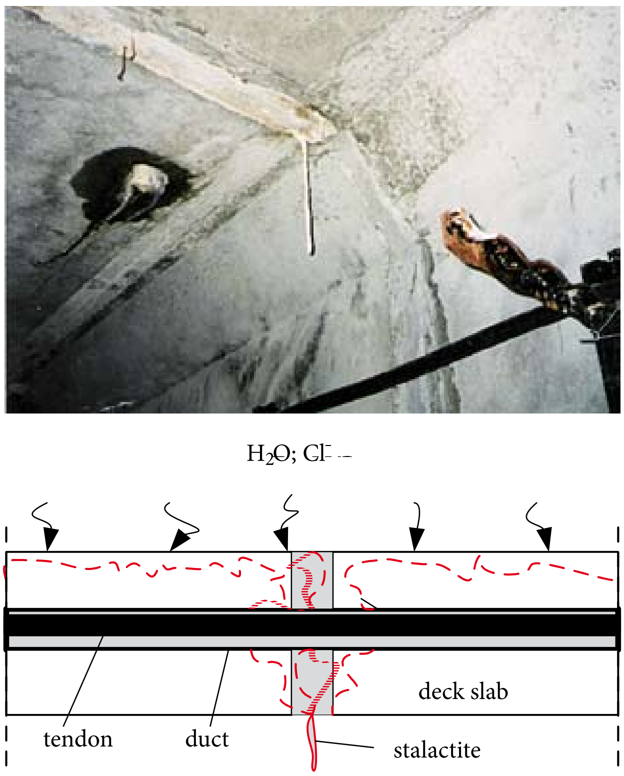


Fig. 4. The effect of leaching on the underside of concrete deck of box girder due to leakage of epoxy joint

Fig. 5. General view of shored span

As an example, collapse without prewarning of bridge staircases for pedestrians due to prestressed tendons corrosion and rupture can be mentioned (Fig. 6). The joints between the central tower and the precast stairs were left without epoxy (dry joins) and the de-icing salts were used in abound. Fortunately, no loss of life was involved.

In some bridges water and concrete samples taken from tendon ducts for laboratory analysis showed clear presence of chlorides (in some places up to 0.2% of concrete volume). Note, that the use of de-icing salts in

Fig. 7. Flowing out of the epoxy glue from the too large joint

Fig. 6. Failure of bridge staircases due to corrosion of prestressed tendons in the dry joints

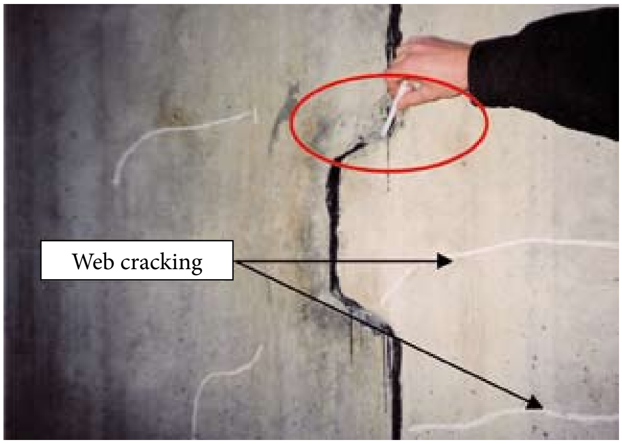


Fig. 8. Spalling of the concrete at the joint and concrete cracks in the box girder web

abound in winter conditions of Baltic states is inevitable (Aavik 2006; Žilionienė, Laurinavičius 2007).

Finally, it is necessary to stress that the durability of prestressed tendons in the segmental joints still remain under discussion.

4.4. Cracking of precast segments

The epoxy joints have to be not only watertight but also free from localized stress concentrations. Match casting allows little room for adjustment the dimensional inaccuracies during erection. It has been noted that in small erection corrections the joints are well filled with epoxy. In larger corrections (more than ~ 5 mm) the epoxy flows out from the joint (Fig. 7) and presence of voids, insufficient filling of joints and concentration points were detected by drilling cores in the joints through the thickness of the girder web.

It was observed that in the epoxy and concrete joints due to reduced and not uniformly joint contact area, producing high-localized creep and stresses, local cracks and spalls around the segment joints can appear. These cracks are approx perpendicular to the joint surface and appear at early ages often right after construction. Typical example of horizontal cracking is shown in Fig. 8.

It is evident that web vertical stirrup reinforcement cannot restrict the web cracking. Usually these cracks are fine, their width is about 0.2 mm or less and they penetrate only a matter of few meters into the end of segment. Not lengthening or widening of these cracks was observed. It seems that this type of cracking would not be detrimental to the performance of segmental girders. Although these cracks can become the potential points of vertical stirrup corrosion (Jokubaitis 2007).

It is very difficult to predict and compute local stresses and to determine the factors causing these stresses. It is evident that not uniformly compression over the entire joint area by means of temporary or permanent longitudinal post-tensioning during construction, service loads and probably secondary effects (thermal, shrinkage, creep, settlements) are their main causes.

Occasional damage or loss of whole shear key occurred sometimes during stripping or handling. Loss of shear keys is not to be detrimental to erection and they can be repaired after erection but before post-tensioning.

In one of inspected bridges opening of the epoxy joints up to 2 mm as expected under overload during replacement of the deck roadway was observed.

5. Conclusions and recommendations

Based on field observations and the results of this work, the following conclusions can be drawn:

1. Erection geometry control and vertical or lateral correction of alignment discrepancies is inevitable and essential disadvantage in precast segmental construction. The joints between precast segments have to permit a smoothing out of alignment and erection errors and to provide adequate bending and shear capacity. Detailed requirements for segmental match-cast joints in national design guidelines or standard procedures are lacking. Recommendations which have to be taken into consideration during design and erection of much-cast joints are presented.
2. Site investigations of 5 major bridges during construction and in service over the past 15 years showed that the joints between segments perform adequately if proper construction procedures are followed. Unfortunately, the joints often represent the weakness zones of the structure. The common sources of trouble are excessive width or inequality of joint thickness, not adequate strength and deformability of joint filling material, not completely filled joints leading to local creep and concentration of stresses. Local concrete cracks and spalls around the segment joints and leakage of joints to moisture and chlorides can appear. Quality control and retrofitting of deficient joints is frequently difficult. All these deficiencies should inevitable have the influence on the durability of the segment structures.
3. The problems with the segmental mach-cast joints generally are due to poor quality construction and lack of maintenance. Most existing defects in segmental structures could have been prevented by more careful construction and improved maintenance procedures. Weakness of the segmental structures can be also related to discontinuities in concrete cover and internal ducts that are not always fully grouted as well as lack of mild steel reinforcement crossing the segment-to-segment joints. It is important to adopt more stringent erection requirements as well as maintenance and inspection procedures in Lithuania similar to what has been done in the USA or other European countries.
4. Based on the survey data and analysis of joint condition state presented in this paper a question may be asked: which is contribution of joints and their condition state on the overall behaviour of segmental structures and what lessons to be learned. It is expected that the internal forces and deformations of the precast concrete segmental superstructures should be dependent on the different from concrete segments strength, elastic, rheological (viscoelastic), and thermal properties of the joint material. The main parameters of joints affecting performance of segmental structures would be joint material properties (strength, deformability, density, and adhesion), joint width, and probably the number of joints in the structure. Traditionally the segment joints are designed only to shear capacity. The effects of strength and long-term deformability of joint filling material in compression and tension are not considered. Future investigations should include analytical and experimental studies to determine the behaviour of segmental structures with possible effects of bonded joints.

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