Price: 10 EUR

HUNGARIAN GROUP OF *fib*

CONCRETE STRUCTURES

ANNUAL TECHNICAL JOURNAL

2001

Vol. 2

CONCRETE STRUCTURES
Journal of the Hungarian Group of *fib*

Founded by: Hungarian Group of *fib*
Publisher: Hungarian Group of *fib*
(*fib* = International Federation for Structural Concrete)

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Editing of online version:
Ádám Damokos

Technical editing and printing:
RONÓ Bt.

Price: 10 EUR
Printed in 800 copies

ISSN 1419-6441
online ISSN: 1586-0361

Cover foto:
Art of reinforcement,
Budapest, Hungary
Photo by Pál Csécsei

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VIADUCTS ON THE HUNGARIAN-SLOVENIAN RAILWAY LINE

DESIGN AND CONSTRUCTION OF THE VIADUCTS

János Barta – Péter Wellner – Tamás Mihalek – János Becze – József Fodor

Two viaducts were constructed as part of the new railway line between Hungary and Slovenia. In our last issue you can find an article about the project's preparation (Vörös, J., CONCRETE STRUCTURES, 2000, pp. 24-28). Details of the viaducts, their design and construction are described herein.

Keywords: prestressed concrete viaducts, incremental launching, prestressing tendon, expansion joint, pile, pier, technology, launching nose, construction deck, hydraulic pushing jack

1. INTRODUCTION

General descriptions have been already given about the new viaducts on the Hungarian-Slovenian railway line in the last issue of the Journal *CONCRETE STRUCTURES* (Vörös, 2000). This included the review of the location and details of the viaduct. Further details are given in Refs. Wellner-Mihalek (2000), Mihalek-Wellner (2000), Fodor (2000) and Becze (2000).

The tender was awarded to the ZALAHIDAK Consortium. Beside the construction the tender contained the design so the viaducts. These designs were carried out by the Technical Department of Hídépítő Co. (leader of the consortium), co-operating with the Stabil-Plan Ltd. and some sub-designers. During this design phase the construction technology and equipment specification had to be taken into consideration as well.

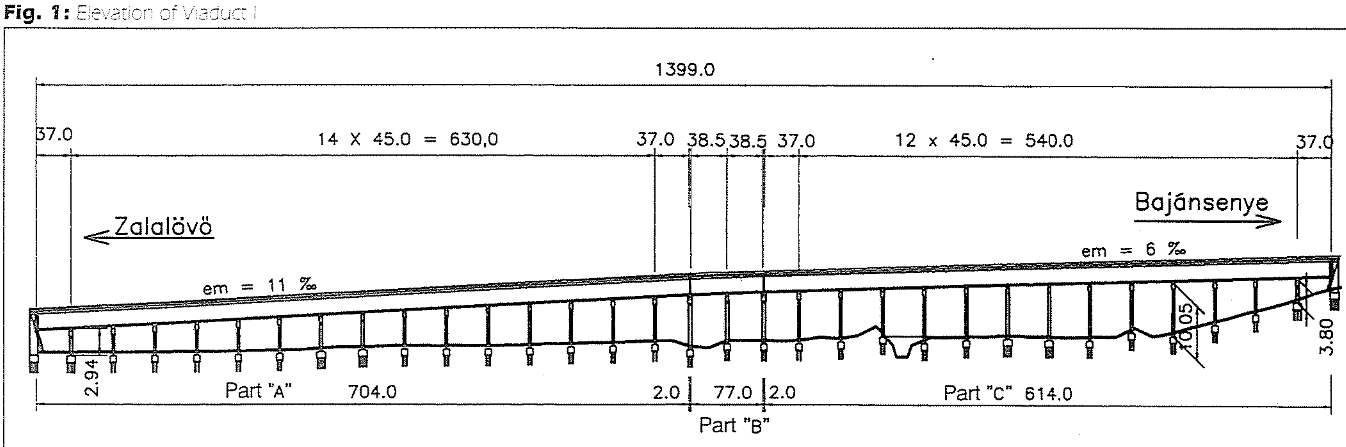
All elements of the design were produced using computer technology and appropriate software: the statics calculations, the processing of the results, the design, the drawings and plotting. The following software were used, the first two for the calculation of statics and final one for the drawings:

- "PONTI" of RIB, a software company from Stuttgart, Germany (owned by one of the sub-designers, Stabil-Plan Ltd.). This was used for the dimensioning of the superstructure. This software is a finite element programme to calculate 3D truss and beam grillage structures, especially developed for structures of prestressed reinforced concrete.
- "AXIS-3D" of INTERCAD Ltd., Budapest, Hungary. For the dimensioning of the substructure and some details of the superstructure. This software is a finite element programme to calculate 3D frame, truss, beam grillage, plate and shell structures.
- AUTOCAD of AUTODESK, USA. For the designing and drawing of almost all the plans.

2. GEOMETRICAL DATA OF THE VIADUCTS

The railway line is horizontally straight over 772 m, then lies in a transition curve over 154 m and finally is in a simple circular curve over 474 m. This alignment could not be modified at all. The longitudinal profile of the railway line had to be slightly modified in order to harmonise it with the longitu-

Fig. 1: Elevation of Viaduct I



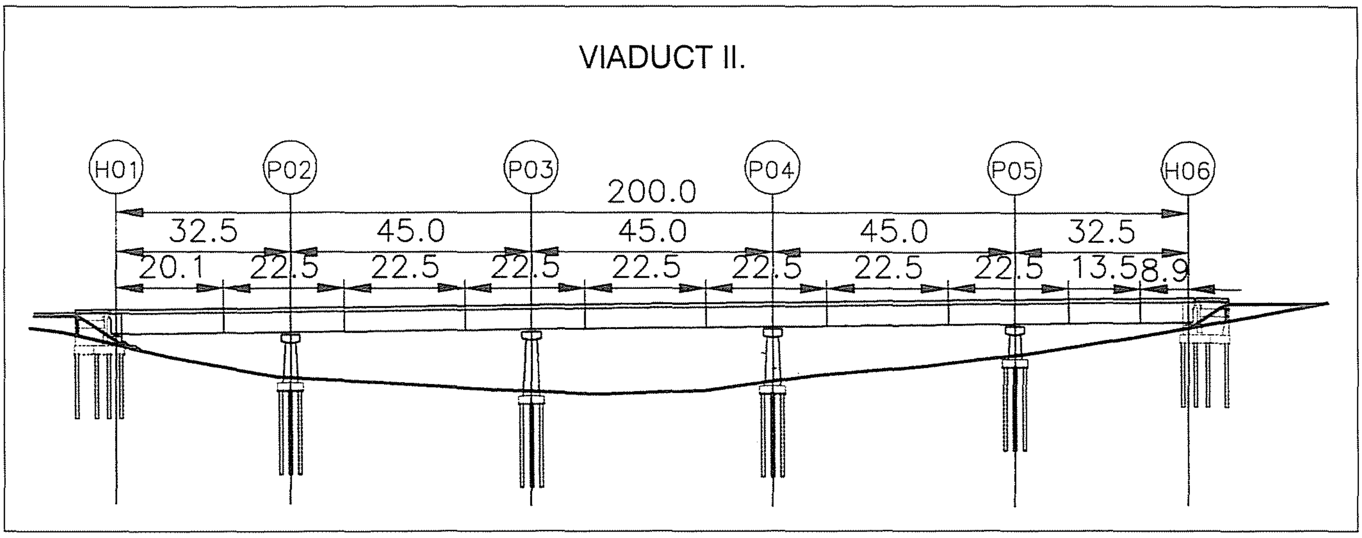


Fig. 2: Elevation of Viaduct II

dinal arrangement of the viaduct. As a result the railway line ascends 11 permill over a length of 682 m, followed by an 85 m long convex slope with a 17000 m radius, and finally a 633 m long section that ascends 6 permill.

The winning bid of ZALAHIDAK Consortium, led by Hídépítő Co., proposed that the viaducts be built using the incremental launching method. Hídépítő Co. first used this technology in Hungary in 1988. Since then 16 superstructures (on 12 bridges) have been constructed by the company employing this method, all of them road bridges. However, this was the first time in MÁV's (Hungarian Railways) history that a railway bridge was built with a prestressedreinforced concrete superstructure using incremental launching technology.

The fact that the superstructure can be pushed using this technology either in straight line or in a clear curve was taken into consideration during forming the general arrangement of the bridge. Furthermore, this was a consideration for both the horizontal and vertical plains.

Last but not least, a very important aspect was the short construction time of 11 months. If the construction proceeded from both ends of the bridge, parallel work at the same time was achievable.

So the solution was as follows:
A 704 m long straight viaduct with an 11 permill ascend from the direction of Zalalövő (designated part "A") and a 614 m long curved viaduct (with 2400 m radius) with a 6 permill descend from the direction of Bajánsenye (part "C") were built. These viaducts could be constructed by incremental launching very efficiently. The 77 m section between the two viaducts, which is in the convex slope, was constructed on scaffold and cast-in-situ (part "B").

Where the railway line runs through a 2300 m clear curve and in transition curve we used a substituting curve of 2400m radius for the box axis of the superstructure. This radius was determined so that the distance between the axis of the railway line and the axis of the superstructure (statical axis) should be as small as possible in order that the eccentricity of the railway loads could be minimised.

At the connection of two cca. 700 m long superstructures the horizontal movements caused by thermal expansion, shrinkage and creep can be cca. +200/-500 mm. Beside the geometrical considerations, the claim to reduce these movements is the other reason for applying the cast-in-situ concrete segment between the two longer pushed viaduct structures. So at the expansion joints the significant movement of the long segment and the lesser movement of the monolithic segment are summarised. The expansion movements were minimised using this solution in that the double fix supports would be in the middle of the long segments so the expansion lengths were even further reduced. The elevation of the 1400 m long viaduct (Viaduct I) is shown in *Fig. 1*.

On the section of the short viaduct (Viaduct II) the case is much simpler: the railway line is straight in its total 200 m length, and ascends 12 permill (its arrangement can be seen in *Fig. 2.*). These parameters are ideal for incremental launching without any modification.

3. SUBSTRUCTURES OF THE VIADUCTS

In case of Viaduct I (1400 m long structure) the substructures can be divided into five groups:
- abutments at the two ends of the viaduct,
- normal piers,
- common piers at the connections of the segments of the superstructure
- fix piers to take the horizontal loads,
- pushing piers, from which the incremental launching was executed.

In case of Viaduct II (200 m short structure) we can find only two of these groups:
- abutments at the two ends of the viaduct, and
- normal piers.

At all groups the substructures can be separated into four structural parts (*Fig. 3.*):
- deep foundation (piles),
- foundation body (pile-cap),
- pier- (or abutment-) wall,
- structural beam.

3.1 Foundation (piles and pile-caps)

The design of the foundation of the viaducts faced several difficulties: for example the soil under the foundations was even more diversified than generally in Hungary; the 160 km/h

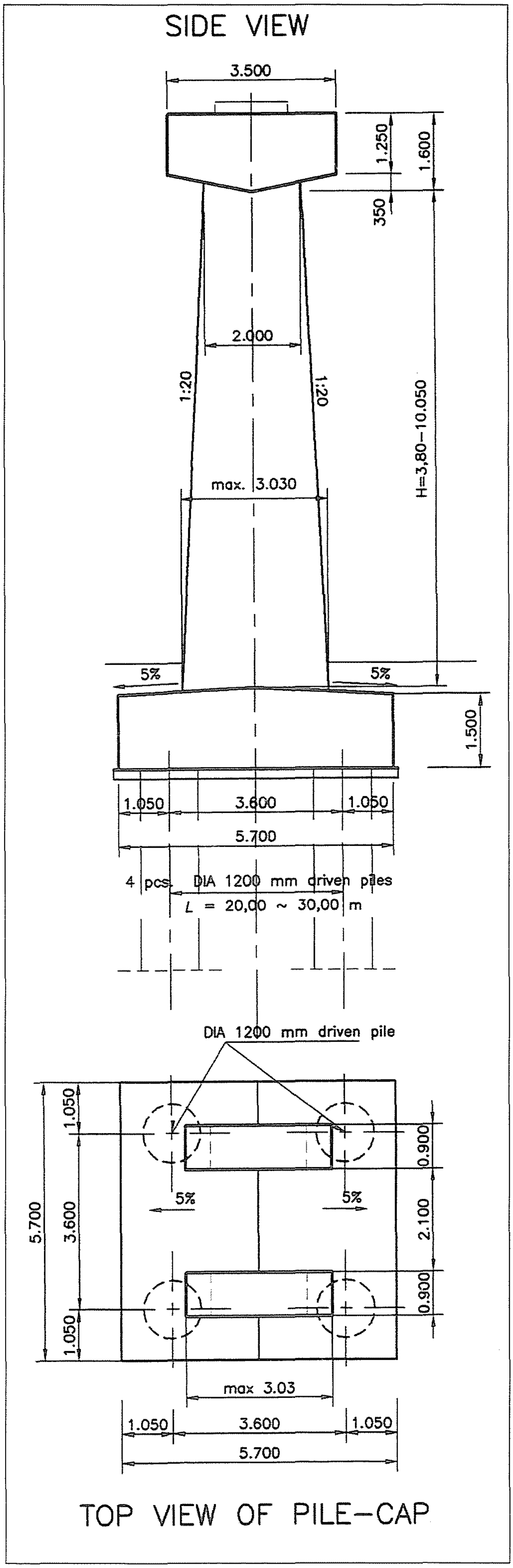


Fig. 3: Side view of pier and top view of pile-cap

planning velocity on the viaduct, the 1400 m length of the structure, etc. The basic idea of the foundation design was that the difference between the settlement of two neighbouring supports should not be more than 10mm, satisfying the demand of the superstructure. However, this demand is not particularly special as it is a requirement of every bridge structure; independently of its structure and the road or railway traffic it carries.

The design of the foundation started with detailed geotechnical preparations. At each pier, geotechnical survey bores and dynamic sound tests were executed. Using the results of the laboratory tests 15 piers were chosen from the 33 supports, where test piles were loaded.

Static calculations were executed to discover the maximum loads of the piles. The final lengths of the structural piles were determined from the load capacity results of the test piles, so the above-mentioned settlement condition was satisfied.

In case of the Viaduct I the structural piles are bored piles of large diameter using SOIL-MEC technology. Their diameter is 1200 mm, their length varied between 18 and 31 metres. The height of the reinforced concrete pile-caps above them is 1.50 m and 1.80 m.

Four piles were designed under each normal pier. The distance between their centre was 3D, according to common practice. This ensures that the pile-caps do not suffer unnecessarily great stresses. Naturally, under the fix and pushing piers more (six or eight) piles were designed.

In case of the Viaduct II the structural piles are driven piles using FRANKI technology. Their diameter is 600 mm and their length varies between 12 and 16 m. The thickness of the reinforced concrete pile-caps above them is 1.50 m. Sixteen piles were designed under each pier. The distance between their centres was 3D.

Beside the vertical load-capacity we also examined the horizontal load-capacity of the pile-foundation of the fix supports.

3.2 Pier-walls

The piers consist of two 0,9 m thick rectangular pier walls which were placed directly under the bearings. Looking at their side view the pier walls have a 1:20 slope. The higher the pier, the greater cross section which connects to the pile-cap and it is even aesthetically advantageous. The neck of the pier-walls, which connects to the structural beam is 2.00 m wide at the normal piers, and 3.00 m at the fix, pushing and common piers. Following the change of the terrain level, the heights of the pier-walls vary between 3.80 m and 10.05 m.

3.3 Structural beams

The structural beams were constructed on the top of the pier-walls. As the pier-walls are directly under the axis of the bearings, the structural beams suffer no bending moment from the vertical loads. Their sizes were determined mainly by the demands of the launching technology of the superstructure. We designed bearing-stools on the structural beams. During the construction the sliding equipment was placed on them, and when the superstructure was completed, the final bearings were put onto them. Beside the bearing-stools a suitable space was ensured which was enough for hydraulic lifting jacks to be placed there at any time (during construction, if needed; when placing the final bearings; and later in case of changing the bearings). The size perpendicular to the axis of the viaduct was also determined by the space requirement of the lateral guidance needed during pushing. On normal piers two different bearings were placed: one, allowing all direction movements, and another allowing only the longitudinal movements, but which takes the perpendicular forces. Both types were MAURER bearings with PTFE sliding surfaces. Steel structures were built into the structural beams of the fix piers to take the horizontal forces. Another type of steel structure was built into the structural beams of the pushing piers which were needed for the execution of the launching method. These steel structures were linked in with DYWIDAG prestressing rods. At these piers the bearing-stools could be made only after the completion of the superstructure. At the common piers four bearings were placed on the structural beams: two for each connecting parts. Because of this these beams are longer than the others.

4. SUPERSTRUCTURES OF THE VIADUCTS

To lead the one-track railway line through a one-cell prestressed reinforced concrete box girder was chosen for the superstructure. The superstructures were constructed from segments with the following lengths:

- "A" part: 8.90 m + 18.00 m + 29x22.50 m + 15.70 m + + 8.90 m, altogether 33 segments,
- "C" part: 8.90 m + 18.00 m + 25x22.50 m + 15.70 m + + 8.90 m, altogether 29 segments,
- Viaduct II.: 8.90 m + 13.50 m + 7x22.50 m + 20.10 m, altogether 10 segments.

The segment-lengths, mostly of half span, 22.50 m, ensured the most elements of the same type. Generally we had to construct two types of segments: above the supports and between them. Naturally the two segments at the ends of the viaducts are different. The bottom width of the box-girder is 4.50 m and its height in the axis of the viaduct is 3.75 m. The webs of 450 mm are vertical. Two reasons accounted for this arrangement. First, the tilting of the construction deck's outside formwork on the concave side would have been very complicated at the curved part of the superstructure. Second, the appropriate size of the compressed concrete cross sectional area over the supports and the space needed for the anchorage of the tendons at the end of the segments required adequate width for the bottom slab.

The bottom and top slabs are 250-260 mm thick. This thickness was determined by the diameter of the prestressing ducts in these slabs, and they could not be considerably lessened even using higher strength concrete.

We strengthened the bottom and top slabs by applying a 250/800 mm haunch in the inner corners of the closed rectangular cross section. The service footways were placed on 180 to 530 mm thick cantilever slabs along the two sides of the box girder. The side-rails were fixed onto the ends of the cantilever slabs. The typical cross section of the superstructure is shown in *Fig. 4*.

The designed strength class of the concrete of the superstructure was C35-24/semiplastic.

The segments were produced in construction decks behind the abutments. When the concrete reached its required strength, they were connected to the completed part of the viaduct with prestressing tendons. The superstructure was moved forward parallel with the axis by lifting-pushing jacks placed on the

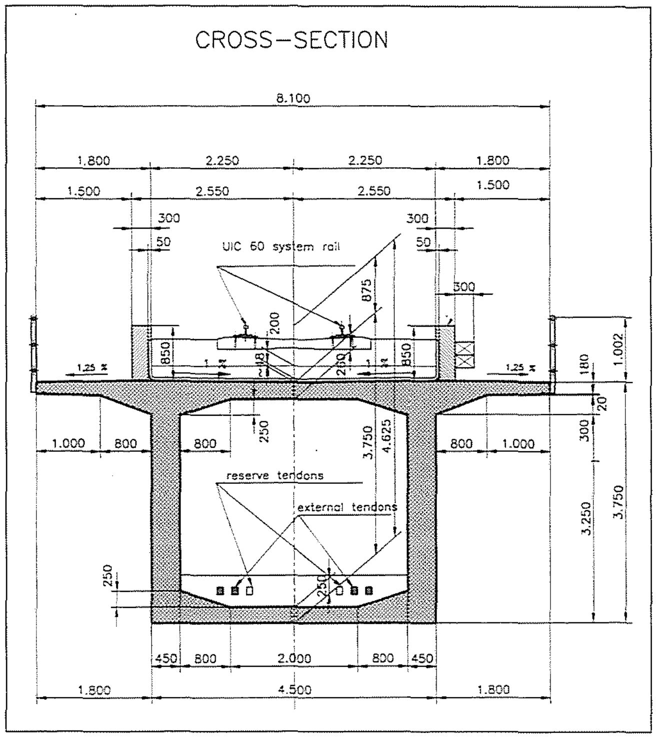


Fig. 4: Cross section of superstructure

top of the pushing piers (two for each bridge part). On the top of the other piers the superstructure slid on built-in slipping equipment, on PTFE sheets fed by hand.

The straight tendons used for the incremental launching were placed into the bottom and top slabs. Blocks were attached at the end of the segments for anchorage. Eight tendons were placed in the top slab and six or eight tendons in the bottom slab.

Beside these straight tendons, curved tendons were placed in vertical alignment and straight tendons were designed into the webs as well. Each of these tendons consists of 15 prestressing strands of 0,6" diameter and are to St 1630/1860 quality. The tendons were anchored into DYWIDAG MA 6815 (DSI, 1998) type anchoring heads with ribbed outside surface. Beside the two curved tendons in both grids, two straight tendons were also placed in the middle section of the viaduct parts at the height of the centre of gravity of the cross section. The purpose of these is to take the braking forces of the railway vehicles (Eibl-Buschmeyer-Kobler, 1995. Guyon, 1991., Virlogeux, 1992).

To carry the live load of the railway, so called "external cables" were used inside the box, led through bottom and top deviators, as shown in *Fig. 5*.

These cables were composed of four pieces of double protected VT-CMM 04-150 D tendons (each containing four strands), a product of the company VORSPANNTECHNIK. Therefore, each tendon consists of 4x4 pieces St 1570/1770 strands. The low frictional resistance of the strands is ensured by the grease layer mixed with graphite inside the inner protecting plastic tube. They were anchored in VT-CMM 16x150 type anchoring heads.

Two tendons next to each web were led on 160-190 m length inside the superstructure. The tendons went through the deviators in galvanised steel structures. Their friction was decreased by PTFE strips placed in the steel structures between the tendons and the steel.

The deviators were placed above the supports and at one-third of the spans. To ensure the possibility to strengthen the viaducts in the future empty reserve places for two extra tendons were also made.

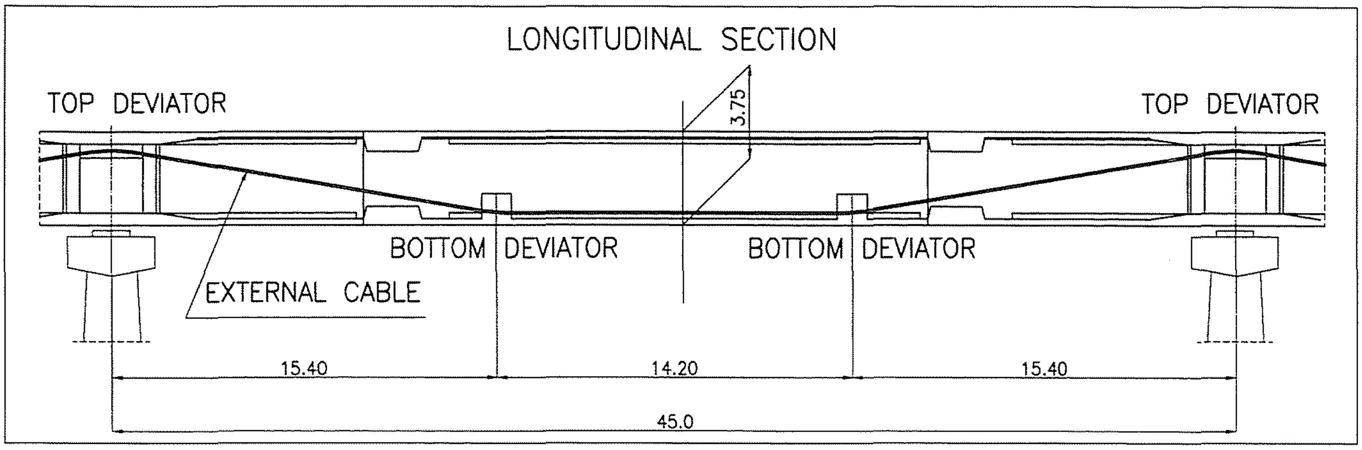


Fig. 5: Deviators of external cables

Two 250 mm thick and 0.90 m high reinforced concrete walls were built on the two sides of the top surface of the box, 4.50 m from each other and parallel with the axis. These two webs which support the ballast of the permanent railway also lead through the viaduct. Under the ballast water-insulation, a protecting and drainage layer was built in. The water is led away from the ballast through gullies placed near to the piers. The down-pipes, led through the box continue down to the foundation bodies (pile caps) fixed to the pier walls.

The design process of a viaduct can be divided into two well-separated parts: one deals with the phase (or phases) during construction, the other with the final (service) state. This is especially true when talking about superstructures constructed by the incremental launching method. In this case the analysis of the two phases are considerably different.

In the construction state we analysed several phases. In all of them the length and the support conditions of the structure (and certainly the model) are different.

In the final state the ready structure in its final place has to be examined under the most unfavourable circumstances of the different type of loads: dead, live and service loads.

4.1 Construction state

If the ratio of the structural height (h) and the span (l) is $h/l = 1/12 \sim 1/16$ (in our case this ratio is 1/12), about 5 N/mm^2 compression is needed in the superstructure for the incremental launching when executed with the help of a nose with suitable stiffness. For this reason only a few extra cables are needed.

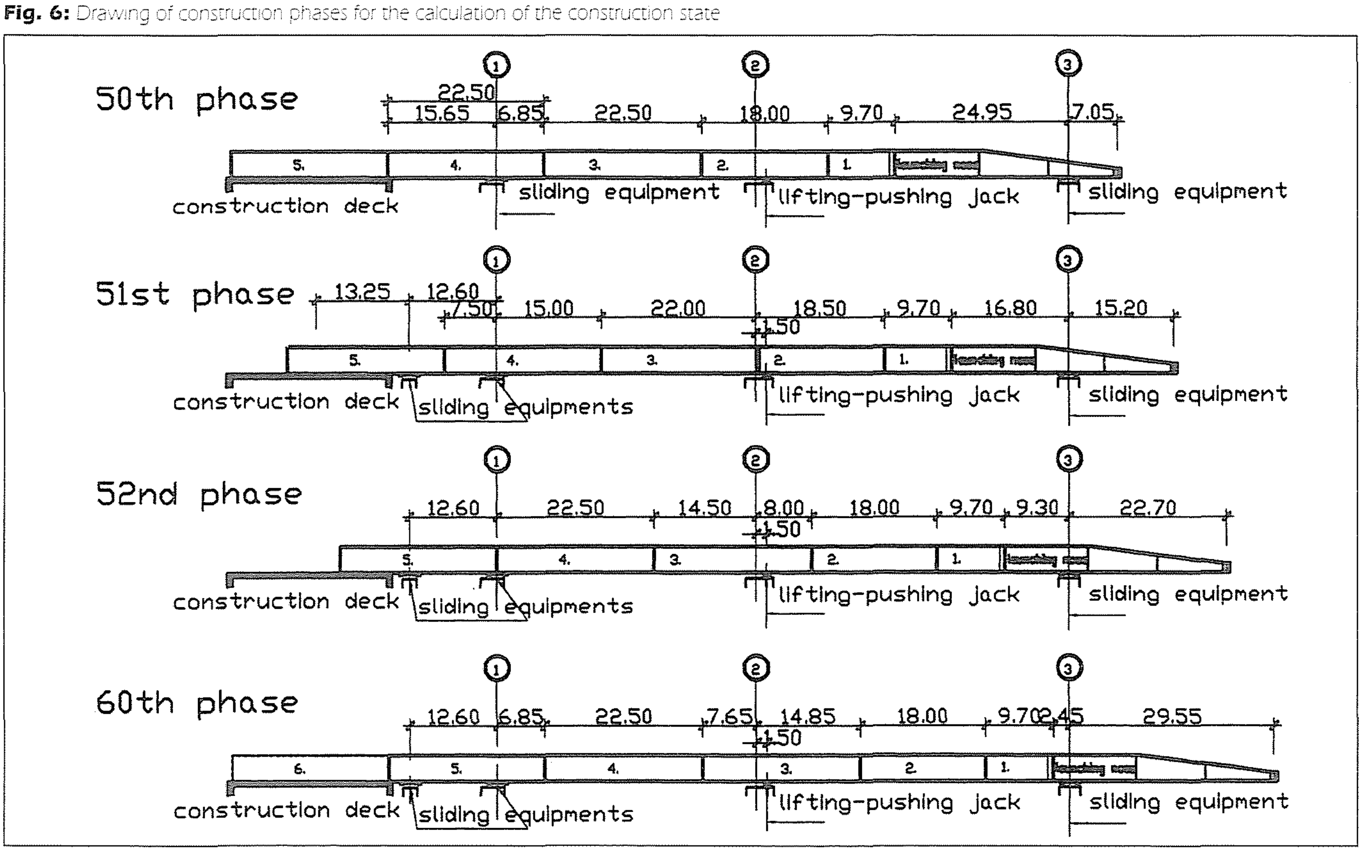
4.1.1 The long viaduct (Viaduct I)

At Viaduct I three separated parts were constructed. The two long ("A": 704 m and "C": 614 m) parts were constructed by incremental launching; the middle one ("B": 77 m) was produced monolithically on scaffold.

4.1.1.1 Parts "A" and "C"

Part "A" and "C" were examined by building phases and checking the stresses of the viaduct parts in several intermediate phases. The concrete quality in the model was generally C35,

Fig. 6: Drawing of construction phases for the calculation of the construction state



but in case of the newest segment it was always C25. An example of the building phases is shown in *Fig. 6.*

The condition of prestressing the superstructure is to achieve at least 26 N/mm² compressive strength of the test cubes. During the preparatory works of the prestressing this value increased up to 28 N/mm², which meant C25 grade concrete. While the segments were concreted in two phases (1st phase: bottom slab and webs, 2nd phase: top slab), this compressive strength value refers to the 2nd phase that was cast later.

In the static calculations we checked the stresses which originated from the following loads:

- the dead weight of the superstructure with varying number of spans,
- the effect of uneven changes of temperature (values at temperature differences:
  reinforced concrete box girder ± 5C°,
  steel launching nose ± 15C°),
- combinations of the height differences of the supports (sliding and lifting places),
- effect of prestressing tendons.

The condition of suitability of a prestressed structure is that the stress in the concrete should not reach the permissible stress. In the case of incremental launching the superstructure is not homogenous because of the contact surfaces between the segments. According to the design practice at these joints, half of the value of the permissible stress of the homogenous concrete can be taken into consideration. This means $\sigma_t = 0.5 \times 1.6$ N/mm² $= 0.8$ N/mm² (considering C25 younger concrete) permissible stress in the concrete in the tensile part of a bended girder in the construction state. This condition was satisfied in our structure in all phases.

4.1.1.2 Part "B"

This part is a two-span (2x38.5 m=77 m long) structure. Its cross section is the same; the prestressing system is similar (straight tendons in the bottom and top slabs, straight and curved cables in the webs and external cables inside the box) to the launched bridges. The prestressing system is shown in *Fig. 7.*

While the construction technology was different (building on scaffold) from the other parts, the calculation of the superstructure of part "B" also differed from the other elements. Here the superstructure became bearing after the prestressing of the hardened reinforced concrete box girder and the scaffold may be removed after prestressing the straight and curved cables. No tensile stress occurred in the superstructure (first laying on the scaffold, then raising up) in any phases of the stress examinations (prestressing the cables pair by pair).

4.1.2 The short viaduct (Viaduct II)

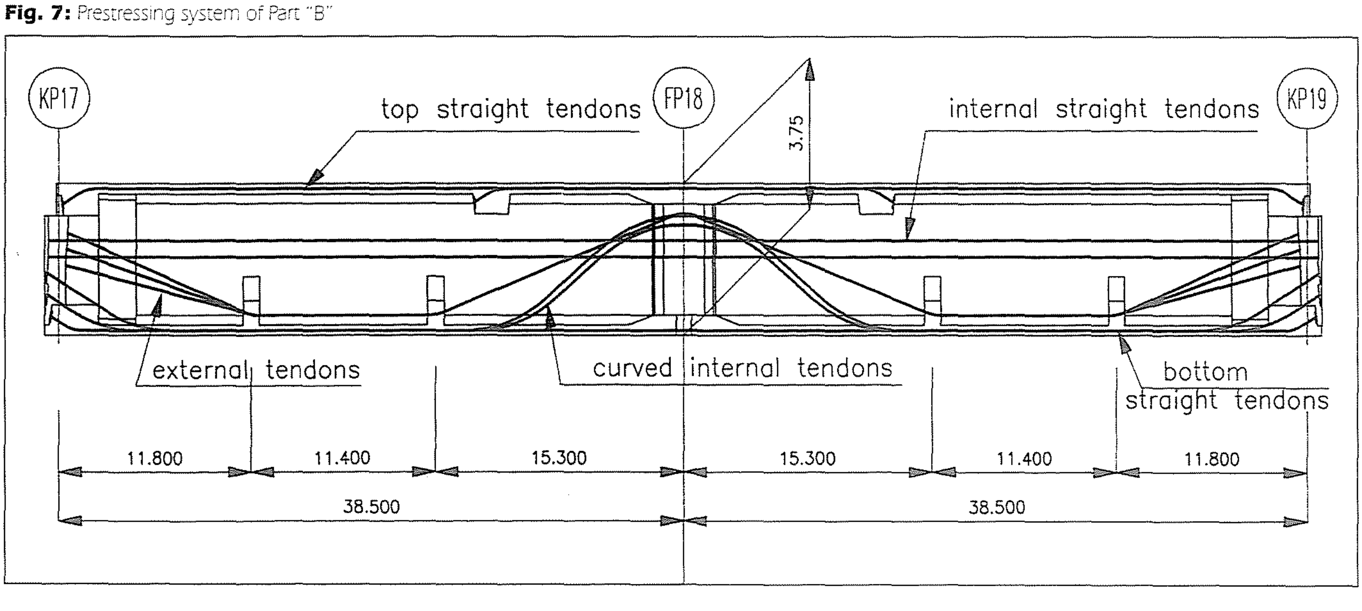
The cross section and the prestressing system of Viaduct II are the same while the length (number of spans) and the constructing and launching system is different from Viaduct I. While the construction decks of parts "A" and "C" were behind the abutments, the equivalent of Viaduct II was in front of the abutment (between the abutment and the first pier). While part "A" and "C" were launched from the top of structural beams with the help of lifting-pushing jacks, Viaduct II was pushed from the back of the superstructure with skew pushing jacks, leaning against a gear rack in the bottom formwork of the construction deck. Because of the different length, construction and launching system and support conditions, Viaduct II needed an extra statical calculation.

As a matter of curiosity let us mention that for the production of the last segment in the construction deck the bridge was over-pushed by about 5 m. After disassembling the launching nose the pushing equipment was transported behind the other abutment and the super structure was launched back to its final position.

4.1.3 Analysis of the attachment of the launching nose

To reduce the cantilever moments of the reinforced concrete girder a steel nose is applied to the front of it when using the incremental launching technology. 6+6 ∅36 mm DYWIDAG prestressing rods in the top slab ensured the connection between the superstructure and the nose and 4+4 prestressing tendons (12x0.6" strands each) in the bottom slab. The shearing force is taken by steel boxes (shearing teeth) welded on the end face of the nose and concreted into the first segment. This arrangement was first applied by the Technical Department of Hídépítő Co. in 1994, and has been successfully used by them since on several occasions. When using this system the safety of the attachment of the nose is at least n=1.5.

Fig. 7: Prestressing system of Part "B"



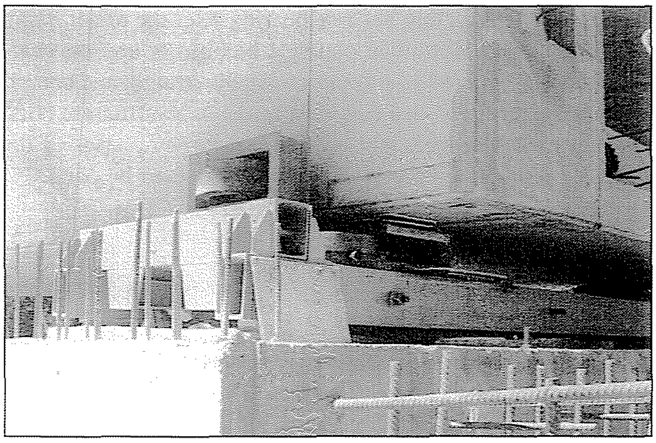


Fig. 8: Rubber wheels of lateral guidance

4.1.4 Lateral guidance

In the case of the straight parts on both sides and the case of the curved part on the inner side, the lateral guidance was ensured with rubber wheels in a vertical axis. (*Fig. 8.*) They could be adjusted even during the construction so the horizontal deviations could be corrected immediately. The steel frame of the lateral guidance was fixed to the bearing stools.

In the construction state of the curved part the radial forces at the supports had to be taken into account as well. For this purpose lateral guidance with vertical PTFE plates were fixed to the outer side of the sliding equipment on the construction beams of the piers. We used a multi-span girder model with a curved axis to calculate the lateral forces developing at the supports during the launching process. As the result of this calculation the maximum lateral force became $F_{l,max}$=96.6 kN (at pier KP21). The load capacity of the lateral guidance fixed to steel frames constructed around the bearing stools on the top of the structural beams was $F_{l,c}$=564 kN, at each support.

4.2 Final (service) state

The completed superstructure on its final place had to be dimensioned to the "U" and "NJ" railway-loads according to Hungarian standards. In addition to the vertical forces the effect of the horizontal forces (lateral railway loads, centrifugal forces, wind load) had to be taken into consideration in the calculation. The geometry of the curved part (the different radii of the railway track and the bridge superstructure) resulted in an eccentric load with alternating direction (max. ±320 mm). An extra eccentricity (±100 mm) must be added to it (used also in case of the straight parts) as the result of incidental displacement of the track. An extraordinary load position is the case of derailed trains. We checked the stresses and the load capacity of the superstructure considering all these circumstances, even their torsion effect.

An extra process was the dimensioning of the floor plate of the box girder. For this we built its total, precise model from shell elements and beside the dead load we loaded it with the standard railway load.

An interesting case was the effect of the external tendons. With an extra calculation we determined the forces of directional change at the deviators (after subtracting the losses), and in the calculation of the superstructure we added them as loading forces.

As the result of the stress calculations we showed that in the contact surfaces between the segments no tensile stresses occur.

5. SUPPLEMENTARY INVESTIGATIONS

5.1 Analysis of failure load

We examined the effect of the braking force using the total model of the superstructure built up from shell elements. The braking force acted horizontally parallel with the bridge axis on the top of the box. We determined the stresses developing in the slabs and webs and added them to the stresses we got in the service state. We calculated the necessary extra (supplementary) tendons for this sum of stresses. They were placed in the webs near to the centre of gravity (centric cables), while the braking forces can work in both direction and this causes symmetric extra stresses.

We also determined the extra reinforcement of the bottom slab needed around the steel structure taking the braking force above the fix piers.

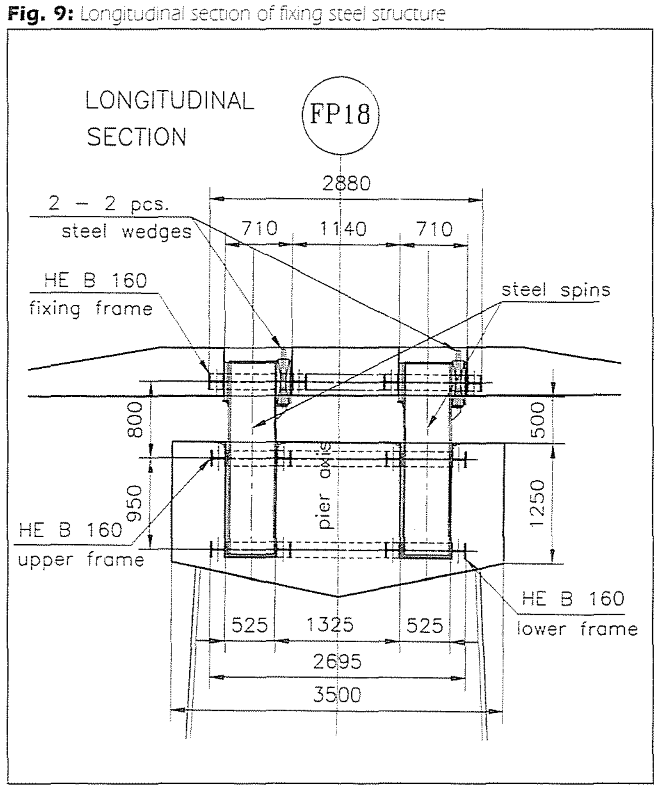
5.2 Fix piers

The fix piers took the horizontal forces parallel with the axis of the part. These are: FP8 and FP9 at part "A", FP18 at part "B", FP25 and FP26 at part "C". In case of Viaduct II the fix support is not a pier, but abutment H01, because at this bridge the horizontal pushing force was taken by this abutment, so it could be formed easily to be able to take the braking force.

At these supports, steel structures (boxes) were built into the structural beams and steel frames into the bottom slab of the superstructure above them. Into these vertical holes steel spins were placed to transmit the horizontal forces from the superstructure to the substructure. Beside these spins 10 mm thick technical rubber plates were built in to make the angular displacement of the support possible. (See in *Fig. 9.*)

According to the Hungarian standards the braking force on a bridge depends on the length of the bridge, but has a maxi-

Fig. 9: Longitudinal section of fixing steel structure



mum of 6000 kN. So in our case the maximum forces to be taken are 6000 kN in case of parts "A" and "C" (that is why we applied two fix piers at each parts, sharing the force between them), 1580 kN at part "B" and 4000 kN in case of Viaduct II.

Checking the fix piers of parts "A" and "C" we also took into consideration the extra horizontal force developing in the superstructure between the two fix piers due to the change of temperature.

5.3 Lateral forces

Lateral forces act on the viaduct structures even in final state. These are:
- wind loads (whisk and wind pressure),
- change of the temperature (even and uneven),
- centrifugal force,
- lateral force of trains,
- shrinkage and creep.

Some of these loads work (centrifugal force) or cause lateral force (even change temperature, shrinkage and creep: the curved structure wants to straighten as their result) only on the curved part. All the others work on the straight parts as well. We examined these effects using a 3D model built up from shell elements. We got the following maximum lateral forces:
- part "A": on the abutment and on the common pier: 600 kN, on other piers: 800 kN,
- part "C": on the abutment and on the common pier: 800 kN, on other piers: 1100 kN.

5.4 Analysis of the natural frequency

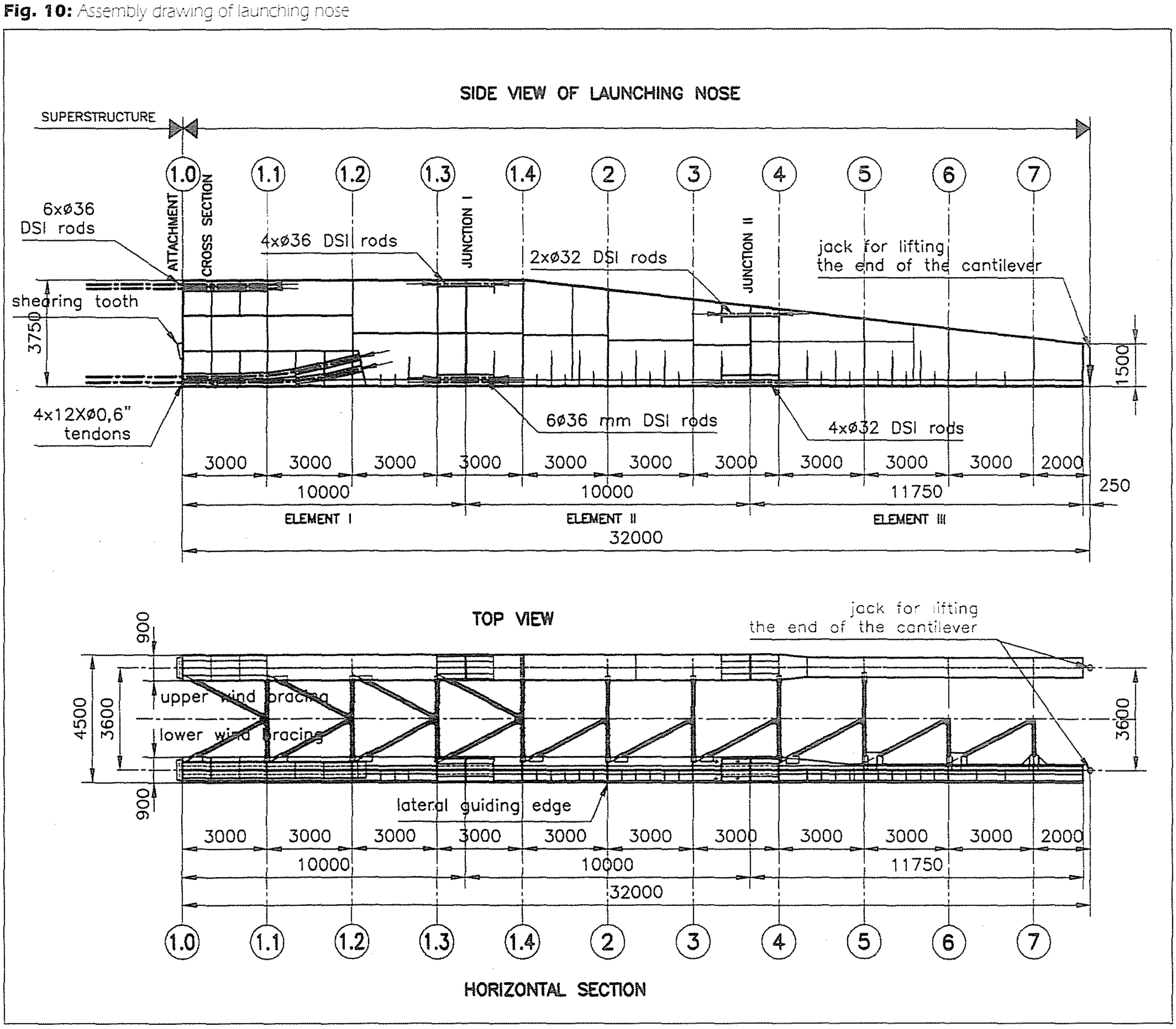
We checked the natural frequency of the viaduct structures with the AXIS-3D program. We built up the total model (37 m+14×45 m + 37 m), and loaded it with the dead loads. We also took into consideration the effect of the prestressing. According to the standard the natural frequency should be between 2.5-5.5 Hz. By our calculation it was to be between 2.5-2.8 Hz, which was proved by the test load of the viaduct.

6. DETAILS OF TECHNOLOGY

6.1 Incremental launching method and its equipment

The development and use of this technology were made possible by two main elements:

Fig. 10: *Assembly drawing of launching nose*



- first: the general use of the high pressure hydraulic equipment,
- second: a plastic material, the PTFE, which has very low friction coefficient under a certain pressure. This pressure is a little lower than the compressive strength of the kinds of concrete generally used for superstructures of viaducts. This means that the compressive stress, given from the PTFE sheet to the superstructure, does not cause local stresses to be too high. (The friction coefficients measured during construction works were m=2.20 %~3.50 %.)

In our case we used two different launching systems at the two Viaducts:

- Viaduct I: lifting-pushing jacks on top of piers (part "A": piers P2 and FP8, part "C" bridge: piers P32 and FP26),
- Viaduct II: skew pushing jacks, leaning against a gear rack in the bottom formwork of the construction deck, pushing from the back of the superstructure.

6.1.1 Launching nose

When the end of the superstructure passes a support during the pushing it becomes a cantilever. Getting closer to the next pier the cantilever moment would grow up to very high values. This is reduced with the help of the launching nose, which is quite light in comparison to the reinforced concrete structure (the weight of the superstructure is about 205 kN/m, the nose's is only about 30 kN/m).

The length of the nose is 32 m, which is about 70% of the longest span. The height at the end connecting to the superstructure is the same as that of the box: 3,75 m. Its weight is cca. 1000 kN. A nose consists of two main girders and cross binding truss between them. A main girder consists of three transportable parts. These parts are connected to one another by prestressing rods. The connection of the nose to the superstructure is solved by prestressing tendons and rods (see above). The deflection of the front of the nose is cca. 80 mm, when it reaches the next pier. This deflection is lifted back by a hydraulic jack on the front of the nose. (Top and side views of launching nose are shown in *Fig. 10.*)

6.1.2 Lifting-pushing jacks

The Eberspächer lifting-pushing jack (as its name shows) consists of two jacks: a lifting and a pushing one. The operation of the equipment has four steps:

1. The lifting jack lifts up the superstructure from the pier about 10 mm.
2. The pushing jack pushes the lifting jack and the superstructure on it about 250 mm forward. The lifting jacks slides on PTFE plates. (The friction coefficient between the jack and the superstructure is quite high: 0.70.)
3. The lifting jack lets the superstructure down back on the pier.
4. The pushing jack pulls its piston back 250 mm.

These steps are repeated until the superstructure slides totally out of the construction deck. For a 22.50 m long segment about 90 phases are needed. One phase lasts about 2 minutes, so the velocity of the launching is about 6-8 m/hour. When the first three segments were moved forward, the superstructure was still not above the pushing pier, so they were pulled by high strength prestressing rods. (The arrangement of the lifting-pushing jack is shown in *Fig. 11*)

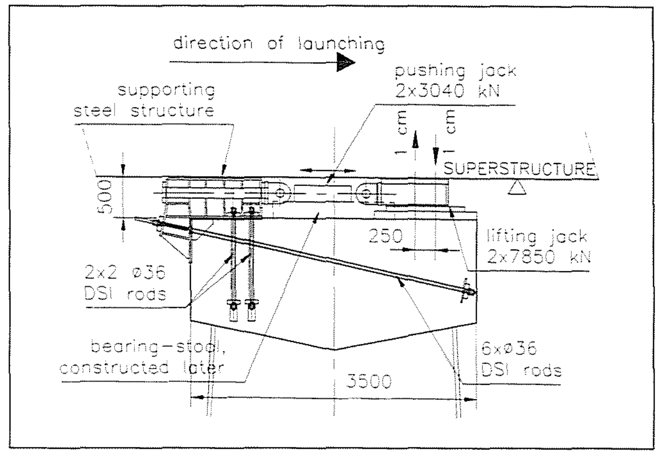


Fig. 11: Arrangement of the lifting-pushing jack

6.1.3 Jack with gear rack

In case of Viaduct II we used jacks with a gear rack. The superstructure was pushed from the back surface of it with skew pushing jacks (2x1000 kN pushing capacity), leaning against a gear rack in the bottom formwork of the construction deck. After the pushing the gear rack is covered with steel formwork plates, which move together with the superstructure when it is pushed out of the deck, and at the end of it they fall down. After collecting they can be re-used.

Using this launching method no lifting is needed during the procedure, the structure moves directly into its final longitudinal profile. The rate of this launching method is about the same as that for the lifting-pushing method.

6.1.4 Temporary supports

When the superstructures of parts "A" and "C" reached the common piers (KP17 and KP19), the superstructure of part "B" was already complete. So there was no place for the launching noses to slide over these piers and they had to be disassembled before reaching part "B". Consequently temporary supports had to be built in the last spans 16-16 m from the common piers. The longest element of the nose was 15 m, so when sliding over the temporary support a part of the nose could always be disassembled. For the last 16 m the superstructure moved without a nose, but while the cantilever was not longer than 16 m, the moments were not greater than earlier in a normal span with nose.

6.2 Construction decks

As part of the incremental launching technology, the superstructures of the viaducts were manufactured in the construction decks. As has already been mentioned above, the construction decks of parts "A" and "C" were behind their abutments (H1 and H33), but in case if Viaduct II it was placed between abutment H01 and pier P02. While part "C" is a curved structure, its construction deck had the same curved shape.

A construction deck consisted of the following parts:

- Foundation (and substructure in case of Viaduct II),
- reinforced concrete (steel in case of Viaduct II) beam-grid,
- bottom formwork,
- outside formwork,
- inside formwork.

The deep foundation of the construction deck consisted of 6×4=24 Ø600 mm Franki piles (l =16 m), with 800 mm high reinforced concrete pile caps on them.

To create a foundation economically without any settlement under such soil conditions is impossible. So it was designed in a way that in case of any occasional settlement it could be re-adjusted to the original designed position. For this purpose two 1400 mm high reinforced concrete beams were constructed on the pile caps parallel with the axis of the bridge together with a reinforced concrete beam-grid 200 mm above them. In this 200 mm high gap there were steel wedges with 100-ton load capacity. These wedges could be adjusted with screw shafts. In case of settlement the beam-grid was lifted back with hydraulic jacks and could be fixed again by the re-adjustment of the wedges. (We would mention that settlement of the construction deck took place during the manufacturing of the first two segments of the superstructure following which no re-adjustment was needed.)

A reinforced concrete cross girder was placed on the top of the foundation beams in front of the beam-grid. Its role was to prevent horizontal movements of the superstructure due to changes in temperature, while during the construction process the superstructure lay on PTFE sheets on every pier (except the pushing piers). So after a completed segment had been pushed forward its end was fixed by Ø36mm Dywidag rods to this cross girder.

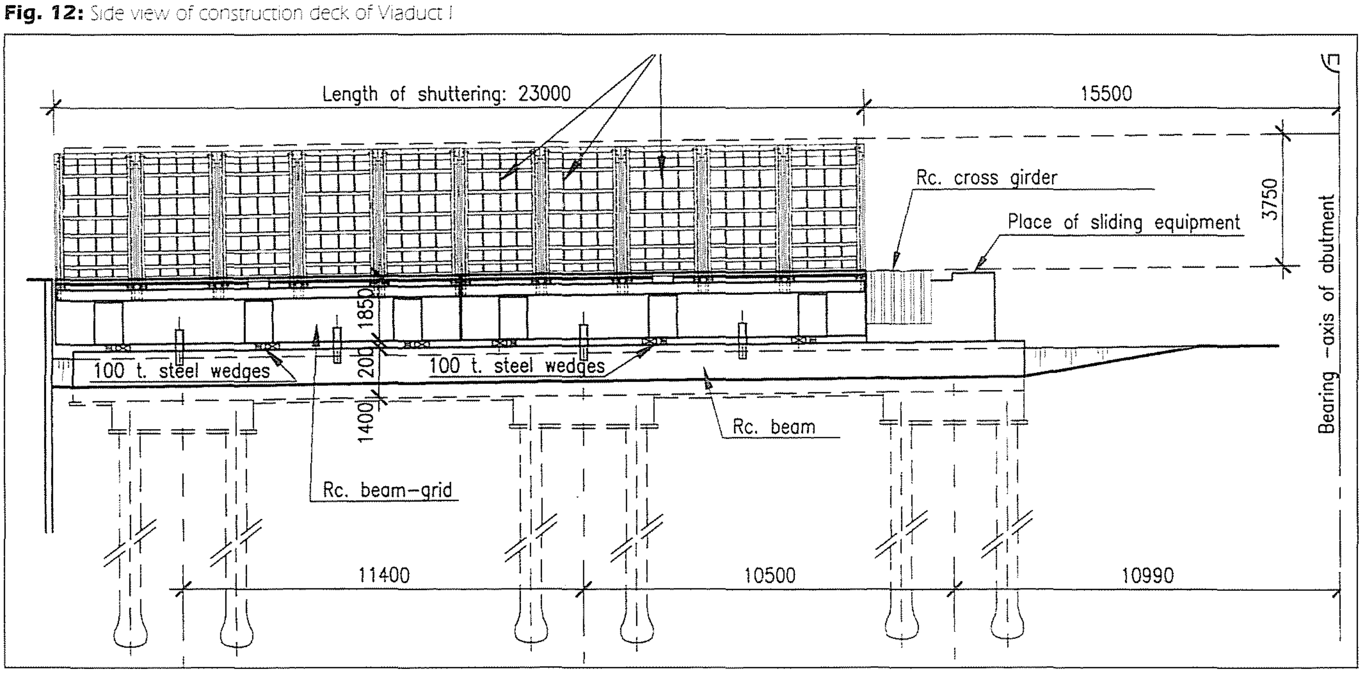
The beam-grid consisted of the longitudinal beams and six cross beams. The longitudinal beams were exactly under the webs of the box girder of the viaduct. In case of part "C" the longitudinal beams were curved according to the curvature of the superstructure and the cross beams were perpendicular to the chord of the axis of the curved construction deck. The longitudinal section of the foundation of the construction deck is shown in *Fig. 12*.

The bottom formwork was supported on the cross beams with 50-ton load capacity steel wedges, and could be sunk down and lifted up by hydraulic jacks. This procedure was needed because of the following. To spare time the reinforcement armature of the first concreting phase was assembled completely behind the construction deck. This was then pushed into its final place on steel U-section rails on Hünebeck rollers. When the rails and the rollers were pushed back the bottom formwork needed to be sunk down. (During this time the reinforcement was supported by the longitudinal beams.) The bottom formwork consisted of 10 parts each of which was made of 8 mm thick steel plates stiffened by U-sections. We left 2 mm gaps between them. The curvature of the bottom formwork of part "C" was created so that these gaps were closed to zero on the inner side of the curve and were opened to 4 mm on the outer side. The gaps were filled with elastic material.

The cantilevers of the cross beams overhanging the longitudinal beams supported the steel structure of the outside formwork. The structure of the outside formwork was determined by the dismantling method. While the commonly used tilting method could not be used here because of the curvature of part "C", we had to pull out the formwork in parallel with the help of hydraulic equipment. The outside formwork consisted of 2×10 parts, each made from 6mm thick steel plates stiffened by cold drawn sections. We left 4 mm gaps between them. The curvature of the outside formwork of part "C" was created so that these gaps were closed to 2mm on the inner side of the curve and were opened to 6mm on the outer side. The gaps were filled with rubber strips. The formwork plate of the cantilever of the box was connected to the vertical plate with a hinge. Both plates were supported by adjustable skew rods with screw shafts. The cross section of the construction deck and the bottom and outside formwork are shown in *Fig. 13*.

Prefabricated inside formwork plates were designed only for the first concreting phase of span-segments and the segments above supports. We also used some of these formwork units with other different segments where we could. At all other locations wooden inside formwork was made in situ. The structure of the steel formwork units was similar to the outside one. The two plates in front of each other were supported by adjustable horizontal rods with screw shafts. These were used when adjusting the formwork before concreting and also to dismantle it. The formwork of the second concreting phase (the floor slab) was not a prefabricated structure. It was created so that when dismantling all parts could be easily carried out of the box by hand. It consisted of light-weight girders and 22 mm thick wooden plates. The girders were placed at every metre, supported by steel structures fixed on the completed webs of the superstructure.

Fig. 12: Side view of construction deck of Viaduct I



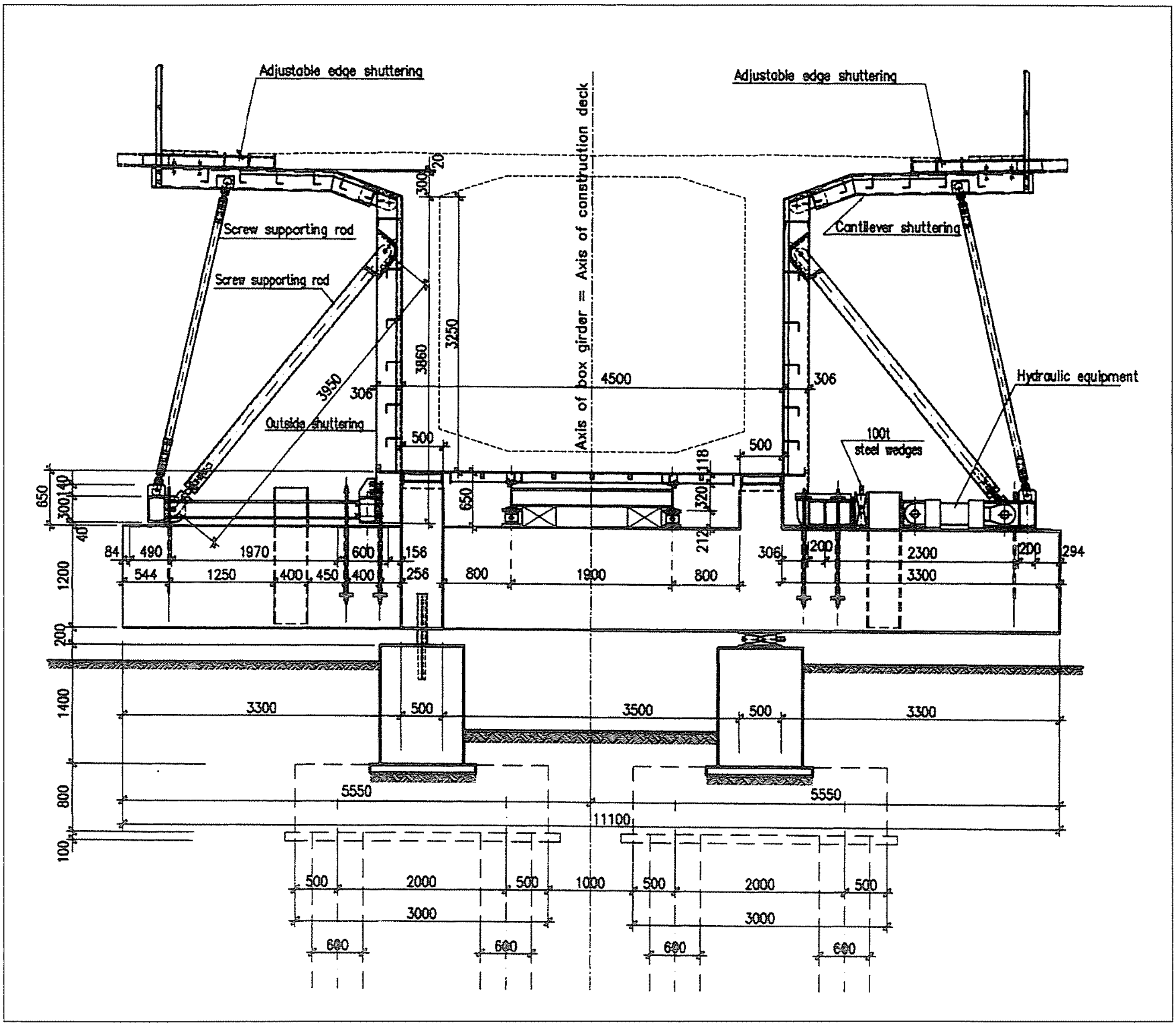


Fig. 13: Cross section of construction deck of Viaduct I

6.2.2 Viaduct II

Considering the construction decks there were two substantial differences between Viaduct I and II:

1. Because of surface relief it would have involved too much earth work to place the deck behind the abutment, so it was placed into the first span between the abutment (H01) and the first pier (P02);
2. No lifting-pushing jacks were used, but jacks with a gear rack to push the superstructure into its final place.

While the deck was in the first span, there was a gap between it and the abutment. If we had wanted to build that part of the superstructure in its final place, expensive extra formwork on scaffold would have been needed. To avoid this we pushed the last but one segment of the superstructure over by 5.45 m. In this way we could manufacture even the last segment in the deck and when it was ready the whole superstructure was pushed back 5,45m from abutment H06.

In the first span the surface relief changed very quickly so the deck became quite highly elevated above the terrain. Due to this a serious substructure had to be designed under it. The deck had three supports: the first (5.30 m from the abutment) lay directly on the foundation and the other two on 2 yoke-type open piers made of 6 Ø324 mm steel pipes. The deep foundation for all of them were Ø600 mm Franki piles capped with R.C. The pile cap of the first support was connected with the abutment. The construction deck transferred its loads to the pile cap directly with the help of 100-ton load capacity steel wedges. In case of the other two supports the deck lay on steel beams, also with the help of 100-ton load capacity steel wedges over the cross girders on the yoke type open piers. The open piers of the same support were connected with transverse bracings to each other. However, no longitudinal bracing were needed between the supports because the open piers carried no longitudinal loads except the movements of the deck itself caused by the changes of the temperature, which could be taken by the steel structure open piers. All other longitudinal forces were transferred to the abutment.

The cross-girder, installed with the aim to prevent the longitudinal movements of the superstructure due to changes of temperature during construction could be found even at this construction deck, but here its material was not reinforced concrete but steel, and was placed right in front of the bottom formwork connecting the two pushing rails (gear racks). The longitudinal section of the construction deck is shown in *Fig. 14* and the cross section is shown in *Fig. 15*.

The sizes, the structure and the operation of the bottom, outside and inside formworks were exactly the same as in the case of Viaduct I, so here they are not described again.

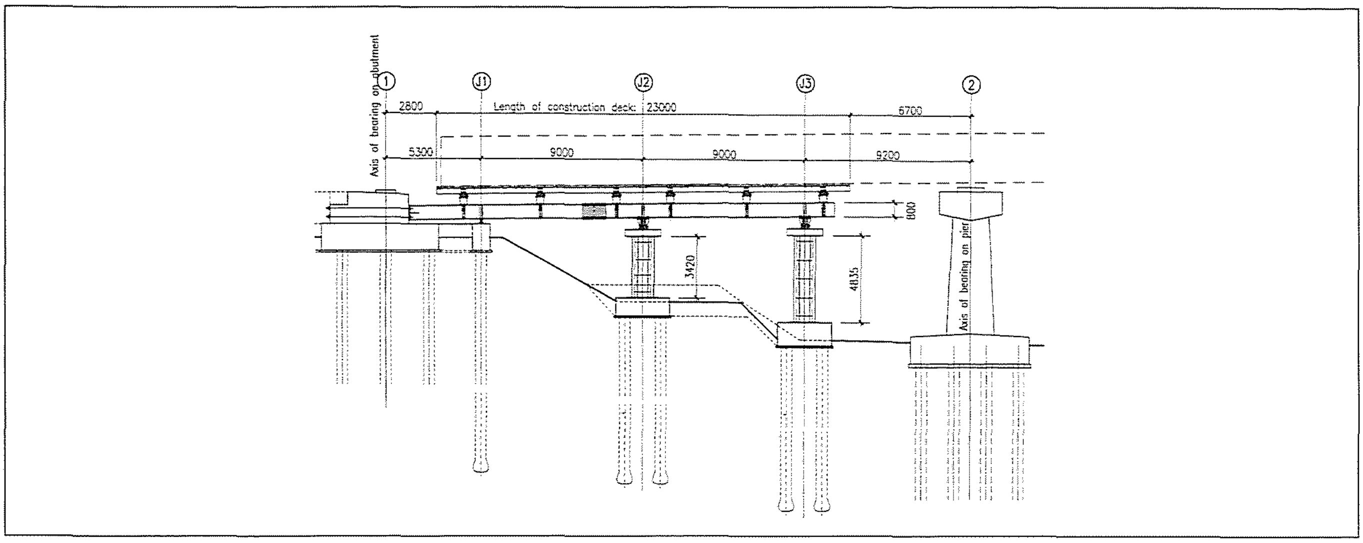


Fig. 14: Longitudinal section of construction deck of Viaduct II

7. CONCLUSIONS

One of Middle-Europe's longest railway viaducts of prestressed concrete and a shorter one were constructed by Hídépítő Co. near Nagyrákos on the new railway line between Hungary and Slovenia. Incremental launching technology proved to be the best construction solution. It meant, that the superstructures were produced on construction decks behind the abutments in 22.5 m long units and were pushed to their final position with hydraulic jacks.

The viaducts were designed from tender design to shop-drawings by the Technical Department of Hídépítő Co. with the help of some sub-designers. During design, not only the dimensioning of the structure was considered, but also the construction technology and the applied equipments. As a consequence the data of the railroad was slightly modified in accordance with the requirements of the construction technology.

The structure was analysed not only during the construction stages but also in its final state. Beside the general dimensioning we had a special task with respect to the examination of the curved viaduct part during the launching and when finally in place. We had to determine the influence of the lateral forces, the wind, the shrinkage and the braking force on the superstructure. We also checked the deformation and the natural frequency of the viaducts.

The engineers of the company completed this unusual task perfectly.

8. REFERENCES

Becze, J. (2000) "Viaducts on the Hungarian-Slovenian railway line – Building-technological interests of the viaducts II.", *VASBETONÉPÍTÉS*, 2000/4, pp. 118-124 (in Hungarian)

DSI-Dywidag Systems International ('998), "Dywidag Bonded Post Tension Systems ¾ Multiple Anchorgae MA"

Eibl, J. – Iványi, G. – Buschmeyer, W. – Kobler, G. (1995), "Vorspannung ohne Verbund ¾ Technik und Anwendung", *Beton-Kalender*, pp.739-803

Fodor, J. (2000) "Viaducts on the Hungarian-Slovenian railway line – Building-technological interests of the viaducts I.", *VASBETONÉPÍTÉS*, 2000/3, pp. 83-92 (in Hungarian)

Guyon, Ph. – Moraison, G. (1991), "Ponts pousses", *Journees Techniques*, pp. 6-47

Mihalek, T. - Wellner, P. (2000) "Viaducts on the Hungarian-Slovenian railway line – Design of the viaducts", *VASBETONÉPÍTÉS*, 2000/2, pp. 53-61 (in Hungarian)

Virlogeux, M. (1988), "Externe Betonvorspannung", *Beton- und Spannbeton* Vol.83, pp. 121-126

Vörös, J. (2000), "Viaducts on the Hungarian-Slovenian railway line – Preparation of the project", *CONCRETE STRUCTURES* 2000, pp.24-28.

Wellner, P. – Mihalek, T. (2000) "Viaducts on the Hungarian-Slovenian railway line – General introduction of the bridge structures", *VASBETONÉPÍTÉS*, 2000/1, pp. 20-25 (in Hungarian)

János BARTA (1968), MSc. Civil Eng. is design engineer at the Technical Department of Hídépítő Co. After graduating at the Civil Engineering Faculty of the Budapest University of Technology he worked for a statical design company for five years. There he designed (as co-designer) building structures consisting mainly of office buildings and blocks of flats. In 1997 he moved to Hídépítő. There he took part in the design works of the sub- and superstructures of several bridges and a pier structure of the Port of Ploče, Croatia. Some of the designed bridges had prestressed reinforced concrete superstructures and were constructed with the incremental launching technology such as the Viaducts on the Hungarian-Slovenian railway line. Meanwhile, between 1992 and 1998, he gave lectures in English for foreign students at the Department of Building Materials of the Budapest University of Technology. He is a member of the Hungarian Group of *fib*.

Péter WELLNER (1933), M. Eng. is Head of Technical Department at Hídépítő. The designing of prestressed reinforced concrete bridges and the associated institutions involved in their technology in Hungary indicates his successful professional background. He received a State Prize for his involvement in the first bridge built using the cantilever mounting method. He also took part in the launching of the method of cantilever concreting in Hungary. The incremental launching technology was initiated in Hungary under his direction. Such structures are now continuously used. He is a member of the Hungarian Group of *fib*.

Tamás MIHALEK (1950), MSc. Structural Eng. He started his designing professionally at Hídépítő and subsequently took part in technological design works beside designing bridges with monolithic superstructures and precast beams. At present he is a leading designer of Hídépítő. In 1988 he took part in the design works of Hungary's first bridge built with the incremental launching technology in Berettyóújfalu. Since 1996 the Technical Department of Hídépítő has been designing the incremental launched bridges (constructed by the company) under his direction. The main fields of his interest are: the design of prestressed reinforced concrete bridges, the influence of the structural materials and applied building technology on the structures and the consideration of influences on static calculations. He is a member of the Hungarian Group of *fib*.

János BECZE (1948), MSc. Civil Eng. He started designing professionally at the Bridge Department of the Road and Railway Design Co. (UVATERV). He took part in the technological design works and the design of the temporary structures of many great bridge structures in Hungary. Since 1987 he has been working at the Technical Department of Hídépítő. His basic task has been the development of the adoption of the incremental launching technology in Hungary and the design of the associated temporary structures. Since 1988 and the design of the Berettyó bridge near Berettyóújfalu, he designed the complete technological process of several structures. In addi-

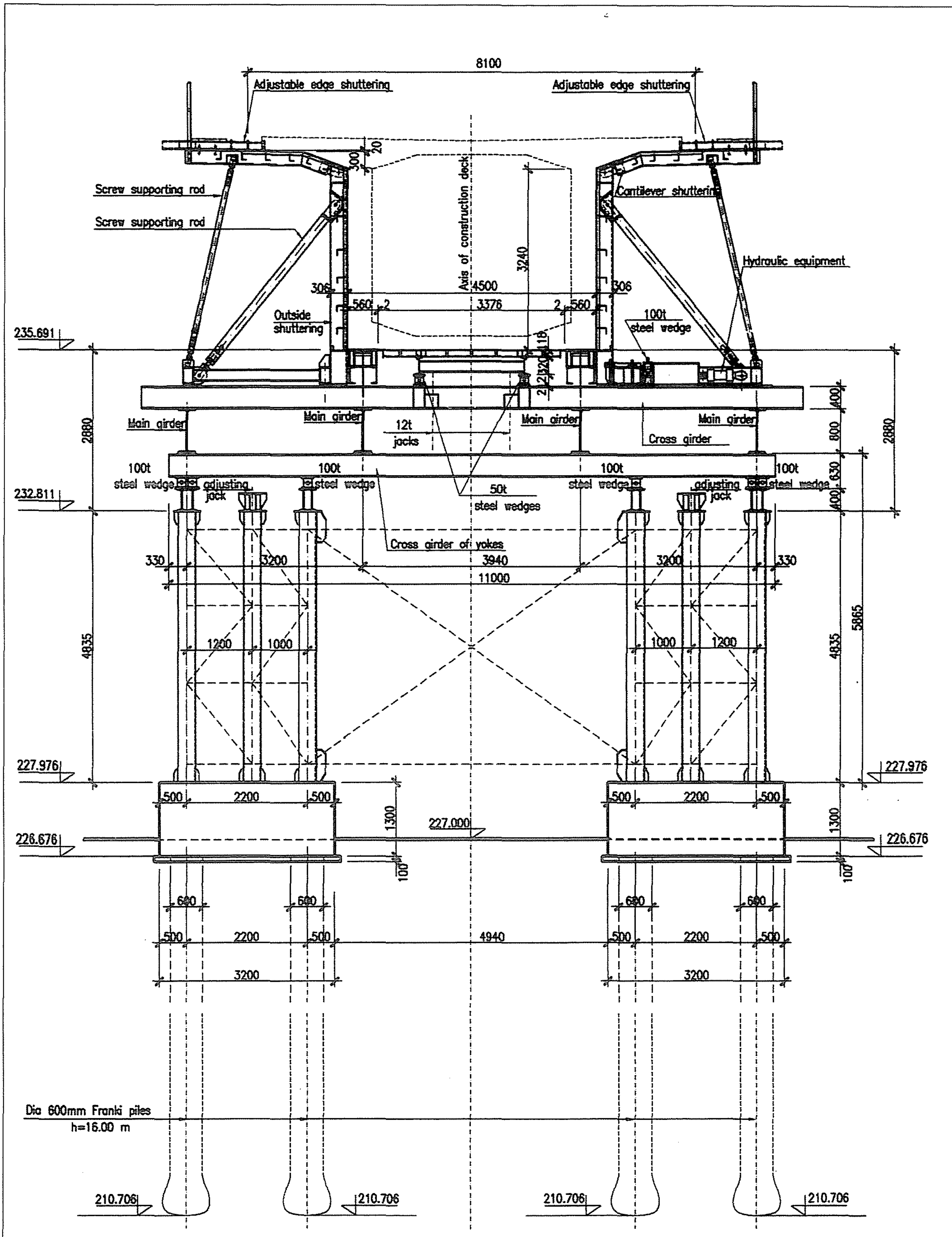


Fig. 15: Cross section of construction deck of Viaduct II

tion he deals with the design of special steel structures and technological tasks. He is a member of the Hungarian Group of *fib*.

József FODOR (1955), bridge builder production engineer. He started his profession design career at Road and Railway Design Co. (UVATERV) in 1976. After designing some small structures as co-designer, his interest turned to the more diversified technological design which provided more novelty. His special professional fields are technological design and temporary structures of the great bridges over rivers (prestressed bridges made with the method of cantilever concreting; composite bridge structures, or steel bridges with ortotropic floor plates). Additionally, he takes responsibility for overall technological design of incremental launching technology. With the exception of four structures, he took part in the design works of all bridges built in Hungary with this technology. He closed his career at UVATERV as Head of Department with the technological design of the Lágymányos Danube bridge in 1994. Since then he has been a design engineer of Hídépítő. His main fields of interest are: research and development of new construction technologies, both in the case of bridge building and any other structural.

SECTION ANALYSIS BASED ON PROBABILITY APPROACH

Dr. Endre Mistéth

The method presented evidently confirms that structural design of communal buildings – which are reliable with low variation regarding the calculations – should be carried out according to the Hungarian prescriptions. However, in the case of industrial buildings or constructions with coefficient of variation of the construction material properties higher than 10% the use of Eurocode seems to be more effective.

Keywords: load-bearing capacity, load, service life, risk, probability, service load

1. PERMANENT LOAD

The mass of buildings involving the weight of the load-bearing and non-structural members should be regarded as permanent loads. The vertical component of the permanent load is disclosed in this paper. The part of load having a permanence lower than 50% should be regarded as an imposed load. The loads which can be adjusted by human intervention (water pressure of basins, load of elevators, etc.) should always be regarded as imposed loads independently of their permanence. The permanent load influenced by the geometrical sizes and the physical parameters should be precisely known at the time of planning. Its measure should be determined on the basis of its expected value. The expected (service) value of the permanent load is a value with at least 50% permanence of the lifetime of the building which can be calculated using nominal sizes and average bulk density.

The ultimate value of the permanent load is a rarely achieved having a probability relating to the lifetime of at least 5%, but at most 95%. This probability, calculated according to Eurocode is at least 1%, but at most 99%. The relative variations of the different permanent loads are divergent. Considering also the desig lifetime, the different values of relative variations are indicated in *Table 1*. All values relate to nominal sizes.

Table 1: Relative variation [v] of different building materials

Denomination	min.	max.	average
Steel structure	2 %	4 %	3 %
Reinforced concrete structure	3 %	5 %	4 %
Concrete structure	4 %	6 %	5 %
Plastered brickwork structure	5 %	7 %	6 %
Stonework	6 %	8 %	7 %
Dense filling	7 %	9 %	8 %
Loose filling	8 %	10 %	9 %
Average according to Hungarian Standards	5 %	7 %	6 %
Average according to Eurocode	10 %	12 %	11 %

The probability distribution of the permanent load can be presumed lognormal. The density function:

$$f(g)=\frac{\exp 4-\left\{\dfrac{\left[\ell n(g-g_0)-u_0\right]^2}{2\sigma^2}\right\}}{\sqrt{2\pi}\sigma(g-g_0)} \quad \text{, for } g \geq g_0$$

$$f(g)=0 \quad \text{, for } g < g_0$$

the expected value: $\bar{g}=g_0+\exp\left(u_0+\frac{\sigma^2}{2}\right)$ (1)

the variance: $s^2=\exp\left(2u_0+\sigma^2\right)\left[\exp\left(\sigma^2\right)-1\right]$

the sqewness: $a=\sqrt{\exp\left(\sigma^2\right)-1}\left[\exp\left(\sigma^2\right)+2\right]$

The parameters g_0, u_0 and v can be calculated by applying momentum or Likelihood methods.

In case of permanent load when $g_0 = 0{,}7$; $\bar{g} = 1$, then $s = v$ and:

$$\sigma=\sqrt{\ell n\left[1+\left(\frac{v}{0{,}3}\right)^2\right]}$$

$$u_0=-1{,}20397-\frac{1}{2}\ell n\left[1+\left(\frac{v}{0{,}3}\right)^2\right] \quad (2)$$

Table 2: Probability characteristics of permanent load

$s = v$	σ	u_0	a	$[g]^{95\%}$	$[g]^{5\%}$	$[g]^{99\%}$	$[g]^{1\%}$
0,03	0,09975	−1,20895	+0,30100	1,052	0,953	1,068	0,937
0,04	0,13275	−1,21278	+0,40237	1,070	0,939	1,105	0,918
0,05	0,16553	−1,21767	+0,50463	1,089	0,925	1,135	0,901
0,06	0,19804	−1,22358	+0,60800	**1,107**	**0,912**	1,166	0,886
0,07	0,23035	−1,23048	+0,71273	1,127	0,900	1,199	0,871
0,08	0,26210	−1,23832	+0,81896	1,146	0,888	1,233	0,858
0,09	0,29356	−1,24706	+0,92700	1,166	0,877	1,269	0,845
0,10	0,32459	−1,25665	+1,03704	1,185	0,865	1,306	0,824
0,11	0,35517	−1,26704	+1,14930	1,205	0,847	**1,343**	**0,823**

$$a = 10v + \left(\frac{v}{0{,}3}\right)^2$$

The values of the permanent load belonging to the 5%, (1%) and the 95%, (99%) probabilities can be calculated by Eq. (2).

$$\left.\begin{matrix}5\%\\95\%\end{matrix}\right\}\left[\exp\left(-1{,}20397-\frac{\sigma^2}{2}\pm 1{,}645\sigma\right)\right]+0{,}7$$

$$\left.\begin{matrix}1\%\\99\%\end{matrix}\right\}\left[\exp\left(-1{,}20397-\frac{\sigma^2}{2}\pm 2{,}326\sigma\right)\right]+0{,}7 \qquad (3)$$

The values of Eqs. (2) and (3) are in *Table 2*.

As indicated in *Table 2* the safety factors are preferably 1.1 and 0.9 according to the Hungarian prescriptions and between 1.35 and 0.82 according to the Eurocode. The Hungarian civil engineering prescriptions are ordered to the average relative variation (v = 0,06) - i.e. to the communal buildings of construction. Eurocode values are ordered to the highest relative variation (v = 0,11), which includes the soil-works and in general the alignment - and occurs at industrial constructions. The values (5%, 95%) may be reasonable at communal constructions while values (1%, 99%) may be reasonable at all other kinds of industrial construction. The higher safety factors also depend on the values of load-bearing capacity.

2. IMPOSED LOAD

Most of the imposed loads related to building construction are distributed loads. The permanence of them during the life time is shown in *Fig. 1*. The load value corresponding to 10% of time probability can be regarded as a long-term load. The time of their simultaneous appearance is $1{,}5 \times 365{,}25 \times 24 \times 0{,}1^4 =$ = 1,3 hours within the lifetime of 1.5 years, so it is possible.

If the distribution function of annual maximums of imposed load is in accordance with the first-upper extreme distribution, the distribution function is as follows:

$$F(p,t) = \exp\{-\exp[-\lambda(p-p_0)]\}$$

where

$$p_0 = p_0 + \frac{\ell nt}{\lambda} \qquad (4)$$

The probability features of the distribution function: expected value:

$$p_t = p_0 + \frac{0{,}577216}{\lambda} + \frac{\ell nt}{\lambda}$$

$$\text{variation: } s = \frac{\pi}{\lambda\sqrt{6}} \qquad (5)$$

sqewness: $a = 1{,}13955$

The $\left(100-\frac{100}{n}\right)$% probability value of the load:

$$p_{\left(100-\frac{100}{n}\right)\%} = \frac{\ell n[(n-\Delta)t]}{\lambda} + p_0' \qquad (6)$$

$$\lim_{n\to\infty}\Delta = \frac{1}{2}$$

Already in case $n = 2$ in Eq. (6) the equality is true. According to the expression the distribution is only followed by the statistic set if its angularity ~1,14. As a first approach a sqewness of ~1,14 is presumed for building construction loads, otherwise the variable should be raised to definite power to reach a = 1,14. If sqewness is higher than ~1,14 the third-upper extreme distribution should be considered. However, this case very rarely occurs. The basic values of loads are indicated in *Table 3*. These are average values of the load maximums occurring every fifty years at buildings designed for fifty years lifetime.

These values can be assumed in one room of the building while the other parts of the building should be loaded to a lesser extent even if that is any of either a flat, an office, a classroom, a theatre or a library. As far as loads of a factory building are concerned, the weight of the heaviest accessory part should be considered on a surface of 5m^2 while on the other parts of the floor a service load of 5 kN/m^2 can be regarded ultimately.

As far as the load configuration is concerned, ultimate loading of one basic span should be regarded, while the other basic spans can be loaded with a service load. This load is 20% of the ultimate load, but for industrial buildings it is at least 5 kN/m^2. The previously mentioned can be illustrated best by means of an example.

3. SECTION QUANTITIES

The section quantity (load-bearing capacity) expected at the end of the designed lifetime is:

$$R(T) = S(T) + \beta_y\sqrt{[s_g(T)]^2 + [s_g(T)]^2}$$

$$\beta_y = \beta_{y,,}(r; a_y, T) \qquad (7)$$

Table 3: Values of service load in function planned life

	s	p_0	λ	p_1	Design life (years)				
					1,5	**5**	**15**	**50**	**150**
Basic value				0,183	0,267	0,518	0,748	1,000	1,230
95-% extreme values	0,2680	0,0621	4,7864		0,767	1,019	1,248	1,500	1,730
99-% extraordinary value					1,108	1,359	1,589	1,840	2,070
Basic value				0,346	0,414	0,615	0,799	1,000	1,184
95-% extreme values	0,2144	0,2497	5,9830		0,814	1,015	1,199	1,400	1,584
99-% extraordinary value					1,086	1,288	0,471	1,672	1,856
Basic value				0,510	0,560	0,711	0,849	1,000	1,138
95-% extreme values	0,1608	0,4373	7,9773		0,860	1,011	1,149	1,300	1,438
99-% extraordinary value					1,065	1,216	1,353	1,504	1,642
Basic value				0,673	0,707	0,808	0,899	1,000	1,092
95-% extreme values	0,1072	0,6248	119660		0,907	1,008	1,099	1,200	1,292
99-% extraordinary value					1,043	1,144	1,236	1,336	1,428

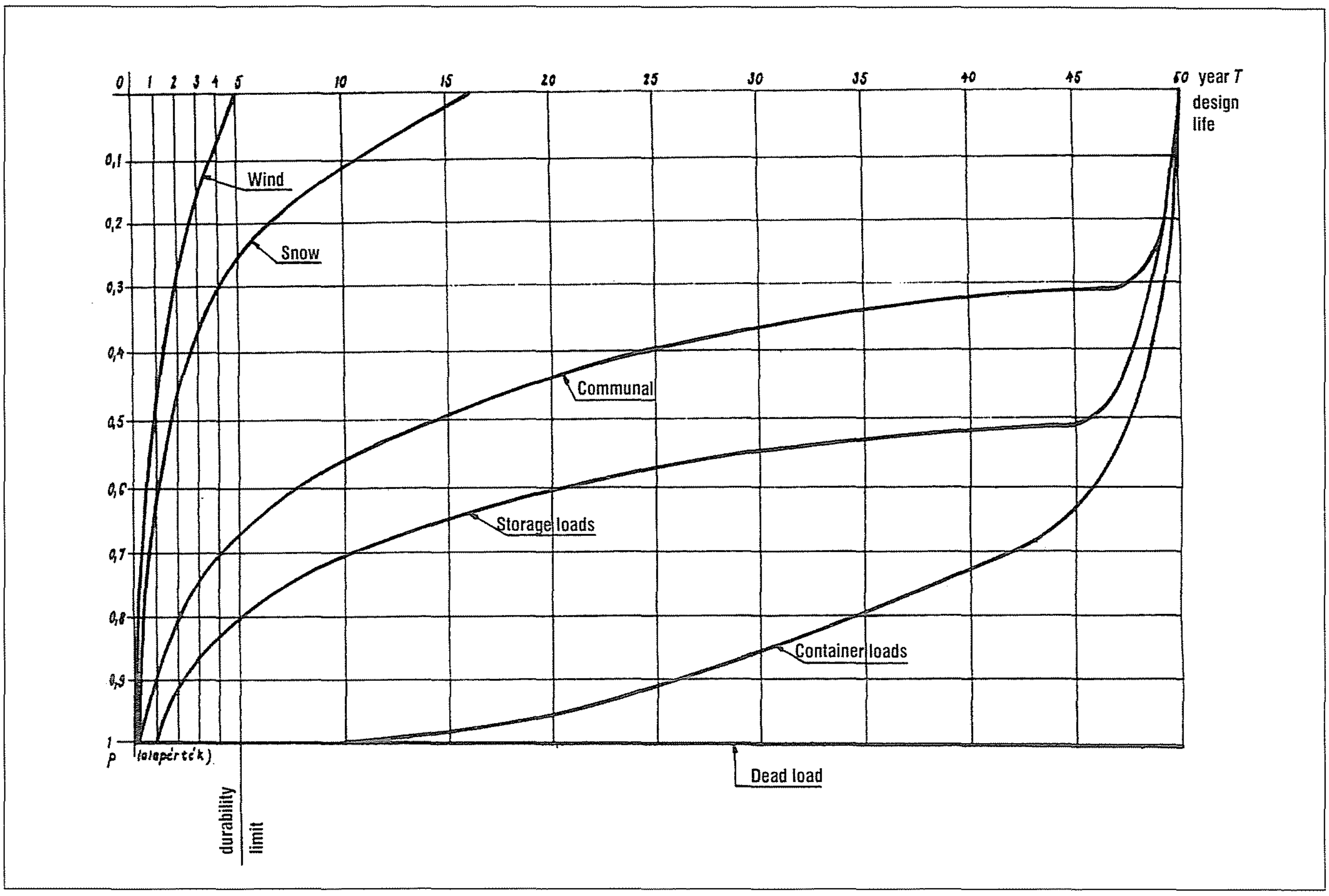


Fig 1: Loads of buildings

In Eq. (7) R is the load-bearing capacity, S denotes the stress caused by the load, s are deviations, b is a numerical value expressing the probability, T is the designed lifetime, r is the inverse of assumed risk and a is the sqewness. If all deviations of Eq. (7) are substituted by the product of relative variation and expected value, Eq. (7) will be the following at the end of the designed life:

$$\overline{R}(T) = \frac{1+\beta_y\sqrt{[v_R(T)]^2+[v_S(T)]^2-\beta_y^2[v_R(T)]^2[v_S(T)]^2}}{1-\beta_y^2[\vartheta_R(T)]^2}\,\overline{S}(T) \tag{8}$$

$$\beta_y = \beta_y(r, a_y)$$

In Eq. (8) v denotes relative variations and b is the numerical value expressing the probability. If both sides of the equation are divided by σ_{B0} (failure stress) and the following expressions are considered

$$R(T) = R_0[\sigma(T)][w(T)][\alpha(T)]$$

$$W_0^R = \frac{R_0}{\sigma_{B0}}$$

$$S(T) = \sum_{j=1}^{\ell} S_{aj} + \ell n T \sum_{i=1}^{n} \frac{S_i^{(1)}}{\lambda_i} \quad \text{then,}$$

$$B(T) = \frac{1+\beta_y\sqrt{[v_R(T)]^2+[v_S(T)]^2-\beta_y^2[v_R(T)]^2[v_S(T)]^2}}{\{1-\beta_y^2[v_R(T)]^2\}[\sigma(T)][w(T)][\alpha(T)]} \tag{9}$$

$$W_0^S = \frac{\sum_{j=1}^{\ell} S_{aj}}{\sigma_{B0}},$$

where denotes the part of the expected value of stress independent of the designed lifetime, and

Table 4: Decrease of load-bearing capacity

t	$\sigma(t)=1-\frac{1}{2}\left(\frac{t}{750}\right)^2-\frac{1}{2}\left(\frac{t}{750}\right)^3$	$w(t)=1-\frac{1}{2}\left(\frac{t}{750}\right)-\frac{1}{2}\left(\frac{t}{750}\right)^2$	$\alpha(t)=1-0{,}15\left(\frac{g+\left(t\cdot\frac{p}{750}\right)}{g+p}\right)^4$	$R=\overset{3}{\underset{i=1}{\pi}} R^{(i)}$
0	1	1	1	1
5	0,99998	0,99664	0,98627	0,98294
15	0,99980	0,98980	0,98360	0,97337
50	0,99763	0,96444	0,98056	0,94345
100	0,98993	0,92444	0,96398	0,88217
150	0,97600	0,88720	0,95254	0,82481
200	0,95496	0,83111	0,93856	0,74491
350	0,84030	0,65778	0,92170	0,50945
440	0,72695	0,53458	0,91310	0,35484

$$\Delta \overline{W}_0^S = \frac{1_n T \sum_{i=0}^{n} \frac{\overline{S}_i^{(1)}}{\lambda_i}}{\sigma_{B0}} \tag{10}$$

If in Eq (10) is a stress unit of the variable imposed load and l denotes the parameter of the first-upper extreme probability distribution, then

$$\overline{W}_0^R = B(T)\left[\overline{W}_0^S + \Delta \overline{W}_0^S\right].$$

In many cases $\Delta \overline{W}_0^S = 0$, then $\overline{W}_0^R = B(T)\left[\overline{W}_0^S\right]$ (11)

If the decrease of strength of the material (steel) is according to

$$\sigma(T) = 1 - \frac{1}{2}\left(\frac{t}{750}\right)^2 - \frac{1}{2}\left(\frac{t}{750}\right)^3 = R_0^{(1)}$$

the elimination of steel strength should be expected after 750 years and its relative deviation is

$$s^{(\sigma)}(t) = 1 + 3\left(\frac{t}{750}\right)$$

The decrease of the section quantity is:

$$w(t) = 1 - \frac{1}{2}\left(\frac{t}{750}\right) - \frac{1}{2}\left(\frac{t}{750}\right)^2 = R_0^{(2)}$$

and its variation is also

$$s^{(\sigma)}(t) = 1 + 3\left(\frac{t}{750}\right)$$

The decrease of long-term strength as a function of time expressed by

$$\alpha(t) = 1 - 0{,}15\left(\frac{g + \frac{t}{750}p}{g + p}\right)^4 = R_0^{(3)} \text{ and } v^{(\alpha)} = 0{,}03$$

where g means permanent load and p denotes imposed load. On this basis the temporal variation of yield strength is shown in *Table 4*.

The β_y value of equations (8) is the following in accordance with the above expressions

$$\beta_y = \frac{R_0[\sigma(t)][w(t)][\alpha(t)] - \sum_{j=1}^{\ell} S_{aj} - 1_u t \sum_{i=1}^{n} \frac{S_i^{(1)}}{\lambda_i}}{\sqrt{\{R_0[\sigma(t)][w(t)][\alpha(t)]\}^2 [\vartheta^R(t)]^2 + \sum_{i=1+n} [s_{Si}(t)]^2}} \tag{12}$$

The expected value $\left(\overline{W}_0^R\right)$ of quantity basing the analysis and indicated in expression (11) can be determined by multiplying the section quantity $\left(\overline{W}_0^S\right)$ belonging to time $t = 0$ with the meanwhile increasing safety factor $[B(T)]$ (9). The $v^R(t)$ value in Eq. (12) is

$$\left[\frac{v_0^{(w)}}{w(t)}\right]^2 \left[1 + 3\left(\frac{t}{750}\right)\right] + \left[v^{(w)}(t)\right]^2 \tag{13}$$

The values indicated in Eq. (13) are the values of the example.

The expression of sqewness decreasing as a function of time is the following:

$$a^{(R)}(t) = a_0^{(\sigma)} \cdot \left\{\frac{v_0^{(\sigma)} \cdot \sqrt{1 + 3\left(\frac{t}{750}\right)}}{v^{(R)}(t) \cdot [\sigma(t)]}\right\}^3 \cdot [1 - d(t)] \tag{14}$$

In this expression the d factor indicates the decrease of the yield stress. Finally, considering that $B_{aj} = \frac{\overline{R}_0}{S_{aj}}$ and $B_i^{(1)} = \frac{\overline{R}_0}{S_i^{(1)}}$ where the meaning of is the stress derived from the unit value of the service load, β_y is as follows:

$$\beta_y = \frac{[\sigma(t)][w(t)][\alpha(t)] - \frac{1}{\sum_{j=1}^{\ell} B_{aj}} - \ell nt \sum_{i=1}^{n} \frac{1}{\lambda_i B_i^1}}{\sqrt{\{[\sigma(t)][w(t)][\alpha(t)]v_R(t)\}^2 + \sum_{i=1}^{\ell+n} \left[\frac{\vartheta_{si0}}{B_{i0}}\right]^2}} \tag{15}$$

4. EXAMPLE

A steel bar having a circular cross-section, a diameter of $d = 100$ mm and an area of $A = 7854$ mm² is given. The expected value of yield strength of steel rolled after 1948 in Hungary according to Korányi (1958) is $\bar{\sigma}_{F0} = 288{,}3$ N/mm², its variation is 30,73 N, its relative variation is $\vartheta_{F0}^{(\sigma)} = 0{,}10659$ and its sqewness is $a_{F_0} = 0{,}679$. The basic value of the permanent load is $S_{g0} = 2{,}7$ kN/m², which is in accordance with the lognormal distribution and the designed lifetime is $T = 100$ years. According to economic considerations the assumed risk is chosen for $\frac{1}{r} \cong 10^{-4}$ at the end of life. The variation of the permanent load is $s = 0{,}06 \times 2{,}7 = 0{,}162$ kN/m² (0,297) and its sqewness is 0,60800 (1,14930). The values in brackets are considered in case of analysis according to Eurocode. The specific value of service load is $\overline{p} = 3{,}0$ kN/m² and is in ac-

Table 5: Calculation of assumed risk decreasing as a function of time (Fig. 2)

t	$b_y(t)$	$a_y(t)$	$v_y(t)$	$\frac{1}{r}$	log r
0	6,67764	0,64505	0,10670	$3{,}66 \times 10^{-6}$	5,437
5	6,14890	0,54456	0,11187	$5{,}48 \times 10^{-6}$	5,261
15	5,89328	0,48601	0,11391	$6{,}41 \times 10^{-6}$	5,193
50	5,34490	0,43058	0,12096	$1{,}78 \times 10^{-6}$	4,750
100	4,68497	0,37440	0,13104	**$6{,}81 \times 10^{-6}$**	**4,170**
150	4,11577	0,31563	0,14154	$2{,}73 \times 10^{-4}$	3,56
200	3,49565	0,25496	0,15204	$9{,}00 \times 10^{-4}$	3,05
350	1,57310	0,16319	0,19904	$5{,}96 \times 10^{-2}$	1,23
440	0,0633	0,11890	0,24592	–	–

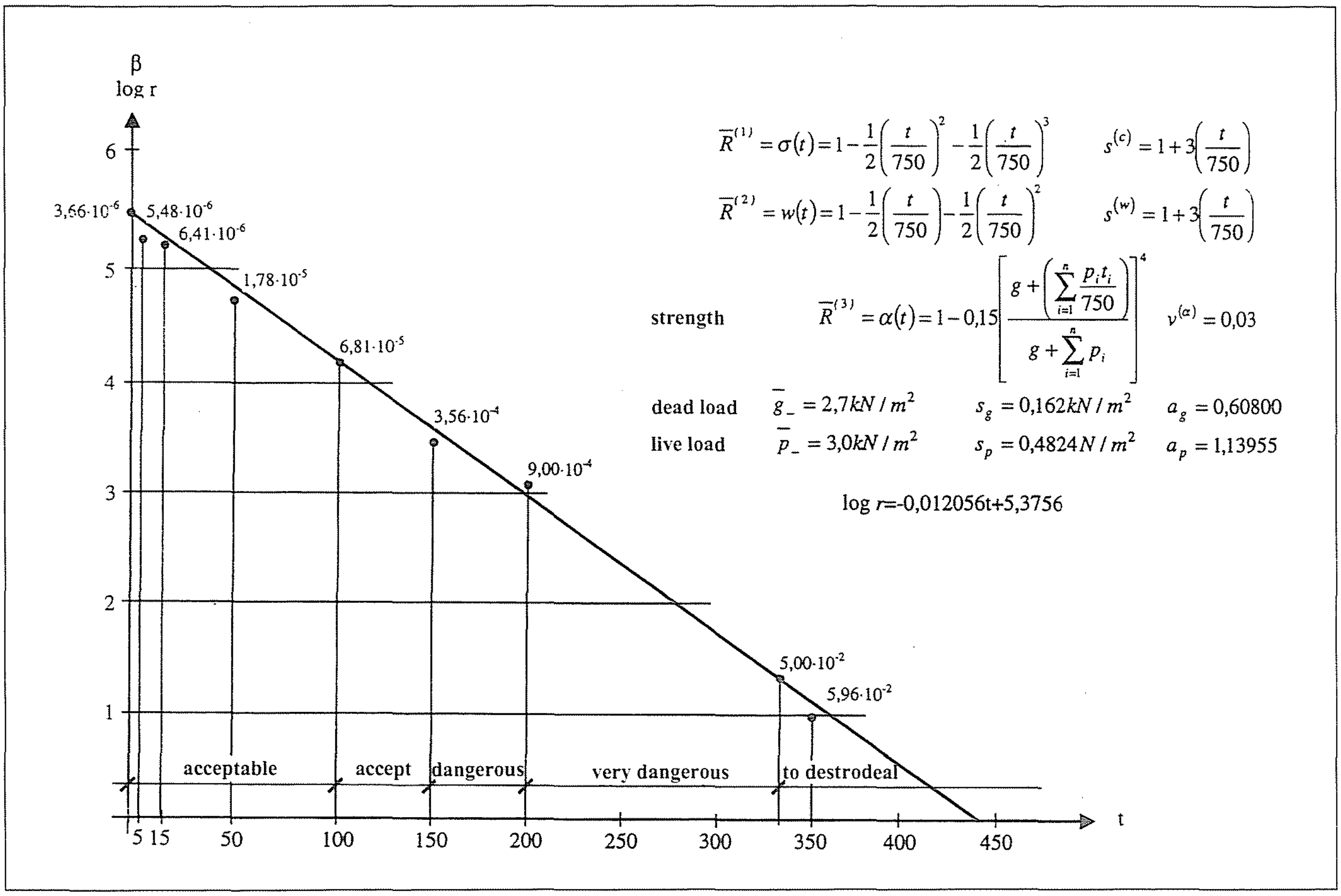


Fig 2: T = decrease of safety of a steel bar planned for a life of 100 years as a function of time

cordance with the first-upper extreme one. Its variation according to *Table 3* is 0,1608 × 3,0 = 0,4824 kN/m² and its angularity is 1,1395. The proportional-action coefficient of load is 228 m² (201 m²). The total loss of steel strength takes place after t_0 = 750 years.

Basic data

Expected value of load-bearing capacity:

$$\overline{R}_0 = 288{,}3 \cdot 7854 \cdot \frac{1}{1000} = 2264{,}3082 \text{ kN}.$$

Expected value of permanent load:

$$\overline{S}_a = 2{,}7 \cdot 228 = 615{,}6 \text{ kN}.$$

$$\overline{B}_a = \frac{\overline{R}_0}{\overline{S}_a} = \frac{2264{,}3082}{651{,}6} = 3{,}67821.$$

Expected value of imposed load: 228 × (1,0) = 228 kN/m².

Expected value of concentrated imposed load:

$$\overline{S}_1 = 3{,}0 \cdot 228 = 684 \text{ kN}.$$

Safety factor of imposed load:

$$\overline{B}_1^{(1)} = \frac{\overline{R}_0}{\overline{S}_1^{(1)}} = \frac{2264{,}3082}{228} = 9{,}93118$$

According to Eq. (15)

$$\beta_{y0} = \frac{1{,}0 - \frac{1}{3{,}67821}}{\sqrt{0{,}10659^2 + \left(\frac{0{,}06}{3{,}67821}\right)^2 + \left(\frac{0{,}1608}{9{,}93118}\right)^2}} =$$

$$= \frac{0{,}72813}{\sqrt{0{,}0113614 + 0{,}0052825}} = 6{,}67764$$

According to Eq. (14)

$$a_{y0} = 0{,}679\left[\frac{0{,}10659\sqrt{1{,}00}}{0{,}10659 \cdot 1{,}00}\right]^3 \cdot 0{,}95 = 0{,}64505$$

and

$$v_{R0} = \sqrt{\left(\frac{0{,}10659}{1}\right)^2 \cdot 1{,}0 + \left(\frac{0{,}005}{1}\right)^2 1{,}0} = 0{,}10670$$

It is shown in *Table 5* that the presumed value of risk of 6,81×10⁻⁵ can be regarded appropriate at a steel bar designed for 100 years lifetime considering the difficulties of estimating the real value of damage caused by a strength loss. A linear line expressed by log r = 0,012056 t + 5,3756 (t = lifetime) can be seated to the logarithms of the inverts of probabilities calculated before by applying regression analysis.. The highest difference is 1,2 × 10⁻³ at t = 150 years.

5. ANALYSIS BASED ON PARTIAL SAFETY FACTORS

The manner of the calculation carried out according to the Hungarian prescriptions is as follows:

$$(1{,}1 \cdot 2{,}7 + 1{,}3p)Te \leq \frac{\overline{\sigma}_{B0}}{1{,}2} \cdot A$$

In the expression g is the constant, p denotes the specific value of imposed load, $T_e^{..}$ is the proportional-action coefficient (the area to be supported by the steel bar); and $\bar{\sigma}_{B0}$ is the nominal value of failure stress in the time $t=0$. In this case:

$$(1{,}1 \cdot 2{,}7 + 1{,}3 \cdot 3{,}0)228 \le \frac{240}{1{,}2} 7854$$

$1566{,}36 \le 1570{,}8$ kN.

If A denotes the cross-sectional area of the bar the analysis should be carried out according to Eurocode in the following way (1,5 value from *Table 3*):

$$(1{,}35g + 1{,}5p)T_c^{\cdot\cdot} \le \frac{\sigma_{B0}}{1{,}15} A$$

$$(1{,}35 \cdot 2{,}7 + 1{,}5 \cdot 3{,}0)201 \le 1639{,}1$$

$$1637{,}1 \le 1639{,}1$$

In case of application of Eurocode the proportional-action coefficient is ~12% smaller than that of $201 m^2$ in the case using the Hungarian prescriptions.

6. CONCLUSIONS

The analysis presented evidently confirms that structural design of communal buildings – which are reliable with low variation regarding the calculations - should be carried out according to the Hungarian prescriptions. However, in case of industrial buildings or constructions with coefficient of variation of the construction material properties higher than 10% use of Eurocode seems to be more effective.

Nevertheless, dividing the constructions into two groups also seems to be economically advantageous. The family of building construction includes so many kinds of buildings that any part of the two groups is much larger than the family of hydraulic engineering structures or bridges. There are also special prescriptions concerning road bridges and railway bridges. (The hydraulic engineering structures built in Europe form altogether a smaller group of constructions than all the communal buildings built in Hungary.)

7. REFERENCES

Korányi, I. (1958), "Analyse der Grenzspannung bei ST (Analysis of tension strength of ST)", 37. *Acta Technika*, Budapest.

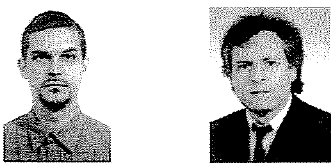
Mistéth, E. (2000),"Theory of Sectional Analysis", (Méretezéselmélet) *Publisher: Akadémiai Kiadó* (in Hungarian)

Dr Endre Mistéth (structural engineer) was born 1912 in Buziásfürdő. He graduated in 1935 from the Technical University of Budapest. His long career consists of technical and political periods. He has designed more than 300 constructions of different materials for many varied purposes. During his career he was active as a layout engineer in Hungary, Egypt, Syria and Iraq. Besides planning reinforced concrete bridge structures, grain elevators, steel construction drawbridges and floodgates, was also designed wooden structure boards and bridges. His political career began in 1944 when Germany invaded Hungary. He became the state secretary of the Industrial Ministry in the government of Mr. Tildy, Minister of Building in the government of Mr. Ferenc Nagy and was then imprisoned, firstly for 8 years and then for 1 year. Upon his release became the head of department, chief clerk and chief engineer firstly of UVATERV and then VIZITERV. He became a university doctor in 1963, a candidate in technical studies in 1969 and then doctor of technical sciences in 1975. He has been expert and lecturer at the Technical University since his retirement.

EDITORIAL NOTE

We appretiate that Dr. Endre Mistéth submitted his paper on Section analysis based on probability approach. Dr. Mistéth wrote his paper at the same time as Eurocodes are about to be adopted and all the parts of the paper are interpreted with the accuracy characterising the author. Since the author has no precise information concerning the background of Eurocode prescriptions, he draws separate conclusions and gives proposals concerning the conditions of adopting Eurocode prescriptions in Hungary in accordance with this.

APPLICATION OF NON-METALLIC (FRP) REINFORCEMENT IN CONCRETE BRIDGES



Adorján Borosnyói – György L. Balázs

In the last decades the considerable deterioration of concrete highway bridges due to corrosion has turned the attention of researchers, designers, producers and owners towards non-metallic and therefore non-corrosive reinforcement. Non-metallic (FRP) reinforcement are applicable as prestressed or non-prestressed reinforcement, as stay cables for cable stayed bridges or as strengthening elements. This paper intends to summarise the applications of this new field and illustrate experiences gained thus far with existing structures.

Keywords: Fibre Reinforced Polymer (FRP), fibre, glass fibre, aramid fibre, carbon fibre, prestressing tendon, durability

1. INTRODUCTION

Concrete is still the most widely applied material for bridges. During more than half a century concrete was supposed to have an infinite service life, therefore design codes did not contain specific recommendations and limitations for durability of reinforced or prestressed concrete highway bridges. Also of note is that in the past the level of industrial air pollution was lower and de-icing salt was not used on bridges.

Because of strong industrial development and the increase in traffic as well as the application of de-icing salts from the 1960's bridges are more endangered. These changed circumstances have a negative effect on the service life of concrete structures and can not be eliminated. Aggressive atmosphere and subsoil water increases the danger of corrosion of embedded reinforcement and the chloride content of de-icing salts accelerates it. The situation is more dangerous for prestressing tendons in thin-walled precast prestressed bridge girders. These tendons are also exposed to stress corrosion. Last but not least, the non-perfectly injected prestressing ducts in case of post-tensioned bonded prestressing tendons increase the risk of corrosion of steel prestressing tendons.

2. CORROSION OF REINFORCEMENT IN BRIDGES

Large amount of $Ca(OH)_2$ – calcium hydrate – is formed during hydration of cement causing the high alkalinity of concrete (pH value between 12,5 and 13,5). In such circumstances a corrosion resistant film a few molecules thick is formed on the surface of embedded steel reinforcement. This monomolecular passive layer protects the reinforcement from corrosion until the pH value of concrete exceeds the value of 9 to 10. However, the surface layers of concrete suffers carbonation due to the CO_2 – carbon dioxide – content of the atmosphere. During carbonation the free $Ca(OH)_2$ content of the hardened cement paste is transformed to $CaCO_3$ – calcium carbonate – leading to a decrease of the pH value. If solutions of de-icing salts with a high chloride content reach the embedded reinforcement, they eliminate the passive layer on the steel surface. In this way the risk of corrosion can be present in strongly alkaline concrete surroundings as well (Balázs, 1991).

The deterioration of highway concrete bridges due to corrosion has an increasing tendency, resulting more frequent surveys and higher maintenance costs. More than 2000 bridges are exposed to regular use of de-icing salts in Hungary during the winter out of a total of about 6000 concrete highway bridges.

Protection of embedded steel reinforcement has always been a major concern of civil engineers. Several improvements have already been tried - ranging from changes in concrete technology to the use of epoxy coated reinforcement. However, these techniques did not always provide the expected results or advantages.

3. NON-METALLIC REINFORCEMENT – FIBRE REINFORCED POLYMERS

3.1 The beginning – Glass Fibre Reinforced Polymer Reinforcement

The idea of using Glass Fibre Reinforced Polymer (GFRP) as reinforcement instead of steel bars in concrete dates back to the 1950's. This idea was actualised in beam tests (Rubinsky – Rubinsky, 1959). The first results were not satisfactory owing to the inadequate bonding characteristics of the GFRP reinforcement available at that time.

After a long period of silence, the possible use of Fibre Reinforced Polymers as reinforcement appeared again in the 1970's. In Germany, Japan and in some other countries research was directed towards the use of Glass Fibre Reinforced Polymer reinforcement.

The German company Bayer AG produced the first commercially available non-metallic prestressing tendon, with the brand name of Polystal® HLV (*Hochleistung-Verbundstab*). The producer developed a full prestressing system with the use of Glass Fibre Reinforced Polymer tendons and anchoring devices. The adaptability of the new material was investigated in full-scale experiments. The first bridge application – a prestressed concrete pedestrian bridge – was erected in 1980 in Düsseldorf. Another three bridges were constructed with Polystal® in the following years in Germany.

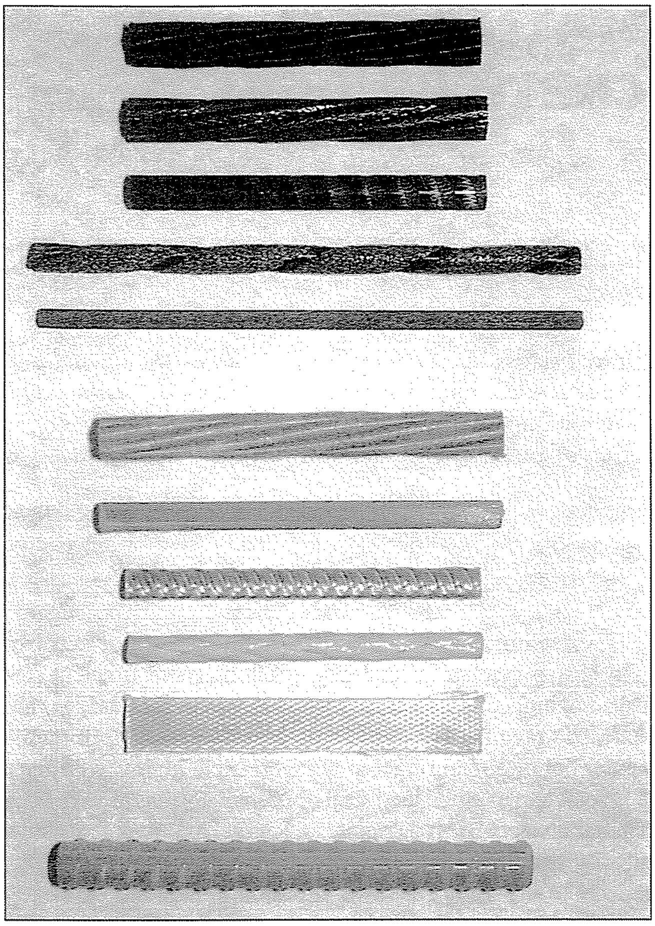


Fig. 1: Surface configurations of FRP reinforcement

In the 1980's a lot of experimental and full-scale applications could be found throughout the world using Glass Fibre Reinforced Polymer reinforcement in bridges (Sweden, Soviet Union, Japan, USA, etc.). However, the widespread use was stopped due to the simple fact, that ordinary glass fibres are not alkaline resistant and therefore may suffer considerable deterioration in the concrete.

Nowadays, there is again an interest in Glass Fibre Reinforced Polymer reinforcement that consists of glass fibres with a special chemical composition as well as a particular resin matrix (e.g. urethane-modified vinylester) to allow the reinforcement's use in an alkaline environment. Producers intend to guarantee the resistance against alkalinity. Such an alkaline resistant Glass Fibre Reinforced Polymer reinforcement is C-BAR® (Marshall Composites Inc., USA) which is developed for non-prestressed use (European producer: Schöck Bauteile GmbH, brand name: ComBAR®). C-BAR® reinforcing bars are available with glass-, aramid-, carbon and glass-carbon hybrid fibres. Bond properties are improved by ceramic ribs stuck to the surfaces of the rebar (Marshall, 1995; Schöck, 1997).

The surface configuration of C-BAR® as well as some other FRP reinforcement introduced hereafter are shown in *Fig. 1*. Their mechanical properties are summarised in *Table 1*.

3.2 Aramid or Carbon Fibre Reinforced Polymer reinforcement

In the 1980's there was a boom in the field of developing new fibrous materials. Considerable efforts were made in Japan and in some other countries. As a result aramid (aromatic polyamide) and carbon fibres have become available. Because of their high price these new fibres were used first in space research and the arms industry (e.g. bullet-proof vests, etc.). A gradual price reduction has made it possible to use in the civil aircraft industrial, automotive sector, electronics (e.g. speakers) and sport equipment (e.g. skis, tennis rackets) applications. For civil engineering purposes (i.e. reinforcement for concrete structures) the first Aramid Fibre Reinforced Polymer bars (FiBRA®, Technora®) and Carbon Fibre Reinforced Polymer bars (CFCC®, Leadline®) were produced in Japan and in the Netherlands (Arapree®). At this time the greatest quantities of these type of reinforcement was still produced in Japan. In Europe one can find producers in Italy (Arapree®, Carbopree®) or in The Netherlands (Carbon-Stress®).

The main advantages of aramid and carbon fibres – in addition to their high tensile strength – is their excellent fatigue strength and insensitivity to electrolitic corrosion. Carbon fibres are fully alkaline resistant, while aramid fibres are considered to be alkaline resistant during their service life.

In North America, Japan and Europe more and more new bridges are constructed using non-metallic reinforcement. With increasing experience of these alternative applications the use of FRPs – mainly CFRP – can spread and may lead to further reduction of the material prices.

The surface configuration of Aramid and Carbon Fibre Reinforced Polymer rebar are shown in *Fig. 1*. Their mechanical properties are summarised in *Table 1*.

Table 1. Characteristics of FRP reinforcing materials

Brand name (producer)	Type of fibre	Resin matrix	Fibre content	Tensile modulus [N/mm^2]	Young's modulus [N/mm^2]	Failure strain [%]	Linear coeff. of thermal exp. [1/°C]
Polystal (Bayer)	E-glass	polyester	68 V%	1670	51000	3.3	$7.0\cdot10^{-6}$
Sportex	E-glass	epoxy (Eskaplast)	n.a.	1600	52000	3.1	–
JITEC (Cousin)	E-glass	vinylester	n.a.	1000–1600	35000–55000	3.8	–
C-BAR (Marshall) (Schöck)	E-glass, aramid, carbon, hibrid	vinylester	60-70 V%	700–1000	38000–42000	2.0	$7.0\cdot10^{-6}$
Arapree (AKZO)	aramid (Twaron)	epoxy	43 V%	1200–3000	53000–91000	3.0	$-1.6\cdot10^{-6}$
FiBRA (Mitsui)	aramid (Kevlar49)	epoxy (Bisphenol)	65 V%	1775	58000	3.1	–
Technora (Teijin)	aramid (Technora)	vinylester (Bisphenol)	65 V%	1765	53000	3.3	–
CFCC (Tokyo Rope)	carbon (PAN)	epoxy (Novolak)	64 V%	2100	137000	1.5	$0.6\cdot10^{-6}$
Leadline (Mitsubishi)	carbon (pitch)	epoxy	65 V%	2250	147000	1.5	$0.68\cdot10^{-6}$
Bri-Ten (Bridon)	carbon (PAN)	epoxy	65 V%	2290	143000	1.6	n.a.
Carbon-Stress (NEDRI)	carbon	epoxy	60-70 V%	2400–3000	155000–165000	1.5–1.9	$0.2\cdot10^{-6}$

3.3 Characteristics of non-metallic (FRP) reinforcement

Fibre Reinforced Polymer (FRP) reinforcement consists of thousands of high strength fibres oriented in parallel with a diameter of 8...10 micrometers embedded into a resin matrix. The purpose of the matrix is not only to keep the fibres together and distribute stresses among adjacent fibres (especially in the vicinity of a local fibre rupture) but also to protect the fibres against transverse actions. The transverse strength of the fibres is considerably lower than that in longitudinal direction.

Fibre Reinforced Polymer products are named according to the type of load-carrying fibres:

Aramid Fibre Reinforced Polymer:	AFRP
Carbon Fibre Reinforced Polymer:	CFRP
Glass Fibre Reinforced Polymer:	GFRP

The tensile strength and Young's modulus of FRPs are a function of the type of fibres, the angle between the fibres and the longitudinal axis, volumetric ratio of fibres (usually around. 60 % per volume), the cross sectional shape and the properties of resin matrix. Their tensile strengths are in the range of 700 to 3000 N/mm^2, Young's moduli are between 70000 and 300000 N/mm^2, while tensile failure strains are between 0,8-4,0%. The characteristics of FRP reinforcement in the longitudinal direction are basically determined by the fibres themselves. However, transverse characteristics are influenced also by the resin matrix as well (Kollár - Kiss, 1998).

FRP reinforcement is typically linear elastic under static loads and show brittle failure, without yielding.

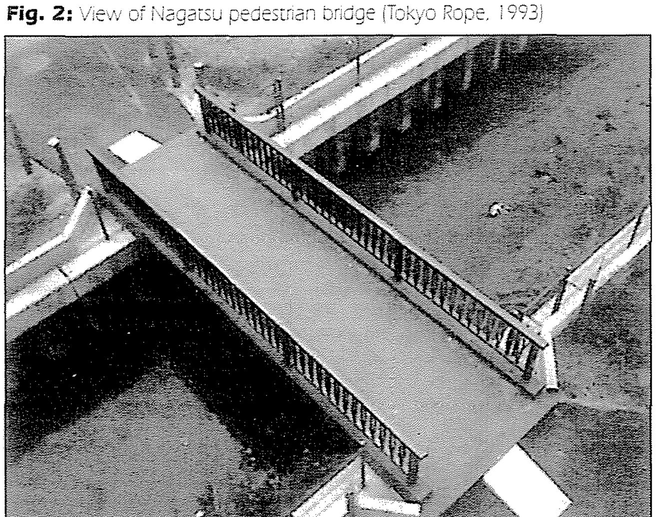
Apart from resistance against corrosion FRP reinforcement also has advantages of low self-weight, magnetic neutrality, high fatigue strength, low relaxation and creep. CFRPs show superior mechanical and chemical characteristics.

The mechanical properties of FRP reinforcement is summarised in *Table 1*.

4. NON-METALLIC (FRP) REINFORCEMENT IN BRIDGES

There are about fifty existing bridges altogether all over the world in which FRP reinforcement has been used. Some of them are pedestrian or bicycle bridges, others are highway or motorway bridges, but we can also find bridge girders of magnetic levitated railways in the list. The majority of these structures are in Japan and North America. Applications in Europe are only about ten (Tokyo Rope, 1993; Taerwe, 1995; El-Badry, 1996; JCI, 1997; Crivelli, 1998; JPCEA, 1998). Experiences available so far are favourable.

Fig. 2: View of Nagatsu pedestrian bridge (Tokyo Rope, 1993)



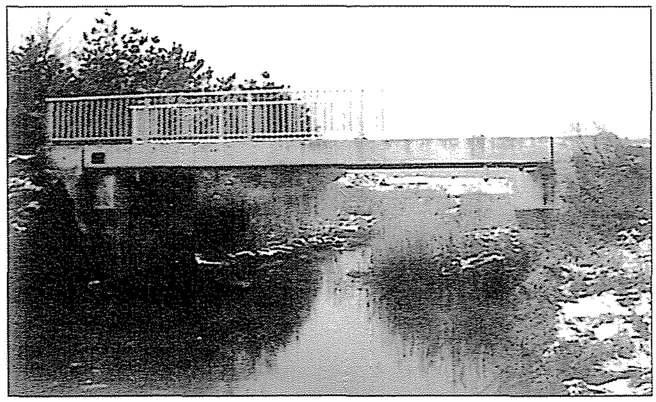


Fig. 3: View of No. 15. bicycle bridge between Hakui and Ganmon (Tokyo Rope, 1993)

In the following paragraphs authors show some examples of bridge applications from Canada, Japan and the USA, which are not intended to be exhaustive. However, these short texts can clearly represent the possible variety of structural uses of these new reinforcing materials.

4.1 Examples from Japan

Japan is the leader, not only in producing non-metallic reinforcement for concrete structures, but in their civil engineering applications as well. Carbon and aramid fibre reinforced polymer prestressing tendons have been available from several Japanese manufacturer since the 1980's (e.g. Tokyo Rope, Mitsubishi Chemicals, Teijin, Nefcom, Mitsui, etc.). Therefore one can find in Japan concrete structures with FRP reinforcement which are more than 10 years old.

Fig. 4: View of Birdie pedestrian bridge (Tokyo Rope, 1993)

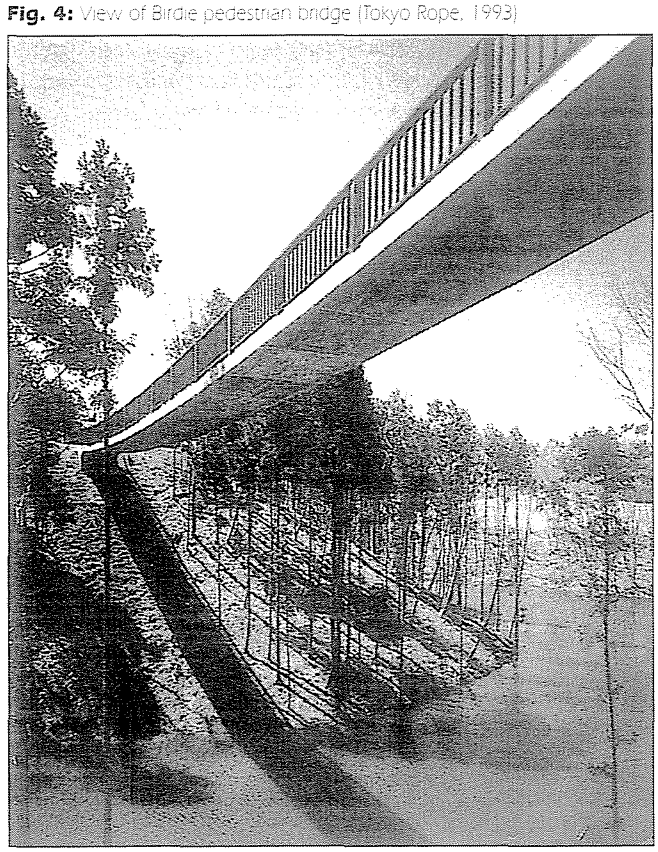


Fig. 5: View of Tsukude Bridge (Tokyo Rope, 1993)

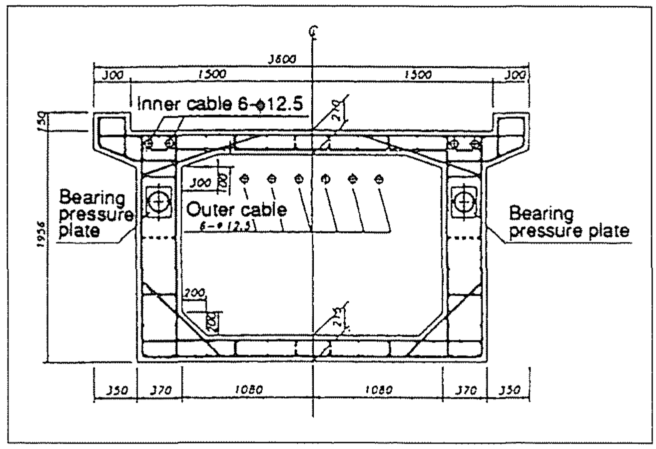


Fig. 7: Cross section of Tsukude Bridge (Tokyo Rope, 1993)

Due to some unfavourable experiences of the European applications in the early 1980's, Japanese glass fibres applications avoided heavy-duty structures at the beginning. The new materials were used mostly in pedestrian and/or bicycle bridges. The Japanese concept was to examine the structural behaviour of the new materials in most types of structures without any risk to humans. According to this demand the greater or more loaded structures were constructed mostly at privately owned premises at the beginning (e.g. golf clubs, national parks, etc.).

The first bridge in the world constructed with Carbon Fibre Reinforced Polymer reinforcement was built in Japan and commissioned in October 1988. The 6.1 m long, 7.0 m wide local road bridge has a superstructure of pretensioned girders with a cast in-situ deck slab (Tokyo Rope, 1993). The following structure was the Nagatsu River Footbridge, constructed in March 1989. This bridge is a monolithic precast pretensioned bridge with no steel reinforcement at all. The bridge has a length of 8.0 m and a width of 2.5 m. A plan view is indicated in *Fig. 2*. Up to the end of 1992 another three low-duty bridges (for bicycle crossing) were constructed with pretensioned-type hollow slab girders, with clear spans of 7.6 to 10.5 m at Hakui Kenmin Bicycle Road. *Fig. 3* shows one of these.

Construction in Japan of longer span bridges with Carbon Fibre Reinforced Polymer reinforcement started at the beginning of the 1990's. A nice example is the Birdie Bridge with a 54.5 m span and 1.7 m width, which can be seen in *Fig. 4*. Its superstructure is a single span suspension deck system with buried formwork. Carbon Fibre Reinforced Polymer ground anchors were also used in this application.

Another special example of a long span large-capacity bridge with Carbon Fibre Reinforced Polymer tendons is the Tsukude Bridge, the construction of which was finished in June 1993. The 111 m long (75 m clear span, 3.6 m width) bridge is a multi-strand post-tension type bridge, the first of its kind in the world. The superstructure has both internal and external post-tensioning cables with the box section. The internal cables consist of 6Æ12.5 mm CFCC® Carbon Fibre Reinforced Polymer strands running in the webs. The external cables consist of 6 pieces of separately anchored ∅12.5 mm CFCC® Carbon Fibre Reinforced Polymer strands running in the box. A bird's eye view of the bridge is shown in *Fig. 5*, while the cross section and the elevation of the bridge is presented in *Figs. 6* and. *7*, respectively.

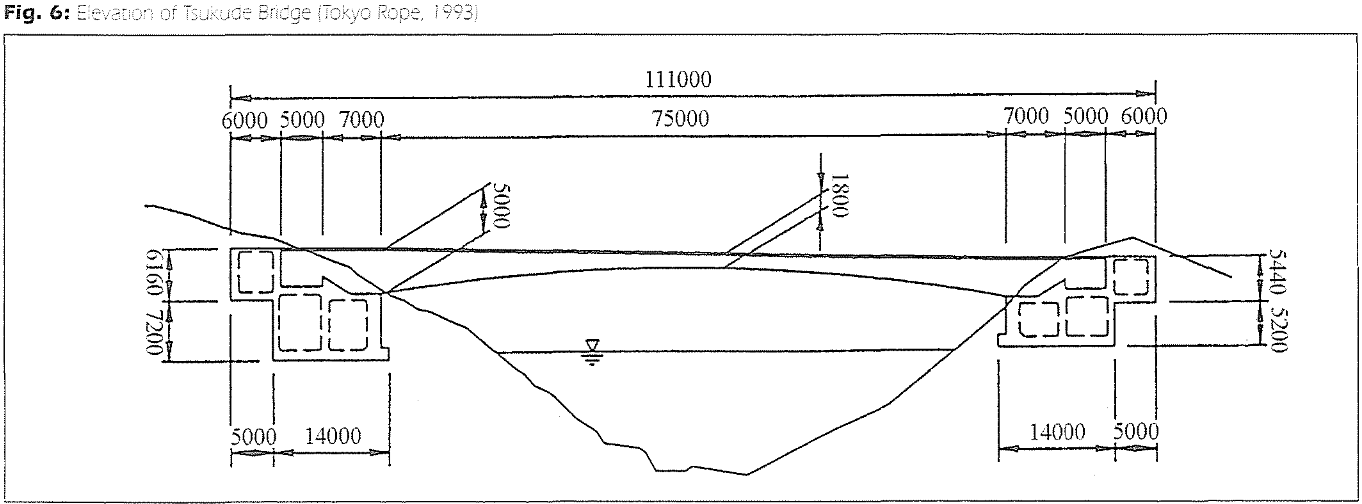
Another interesting example of a two-span curved prestressed concrete bridge can be found at the Haramachi Power Plant. It was constructed from precast segments at the end of 1997 (*Fig 8.*) (FRP International, 1997). The girders of the 12.4 m long span are 7 prestressed precast beams. On the other hand, girders of the 25.1 m long span are constructed from the above mentioned precast segments. Every segment was prestressed pretensioned by 2×∅12.5 mm CFCC® strands and were made continuous on site by prestressing of 6×∅12.5 mm CFCC® strands.

4.2 Examples from North-America

In Canada and the USA numerous bridges can be found with non-metallic reinforcement from three of which are mentioned here briefly.

The Beddington Trail Bridge (*Fig. 9*) was opened in November 1993 at Calgary, Alberta (Rizkalla – Tadros, 1994). This was the first Canadian highway bridge with non-metallic

Fig. 6: Elevation of Tsukude Bridge (Tokyo Rope, 1993)



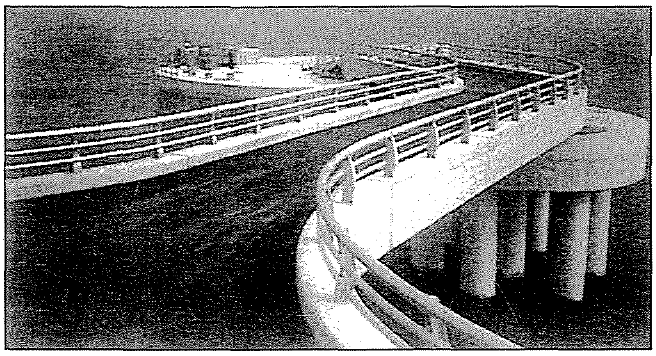


Fig. 8: Bridge at Haramachi power plant, Japan (FRP International, 1997)

Fig. 9: Beddington Bridge, Calgary, Canada (Rizkalla – Tadros, 1994)

reinforcement which has a built-in continuous structural monitoring system ("*smart structure*"). The monitoring system consists of built-in strain gages and thermal sensors which allow the control of the responses of the structure during the complete fabrication and service. The 33° skew-bridge has two spans (22.85 m and 19.23 m) with continuous deck slabs. Bridge girders are 13 precast pretensioned bulb-T beams at each span, from which 6 were cast using Carbon Fibre Reinforced Polymer prestressing tendons. ∅15.2 mm CFCC® strands (Tokyo Rope) were used in 4 girders and ∅8 mm Leadline® tendons (Mitsubishi Chemical) in 2 girders. Design conditions required the same behaviour of CFRP prestressed elements as that of the steel prestressed elements under service load. As a result of this criterion, CFRP prestressed beams were designed to higher load carrying capacity but lower ultimate deflection than that of the steel prestressed girders.

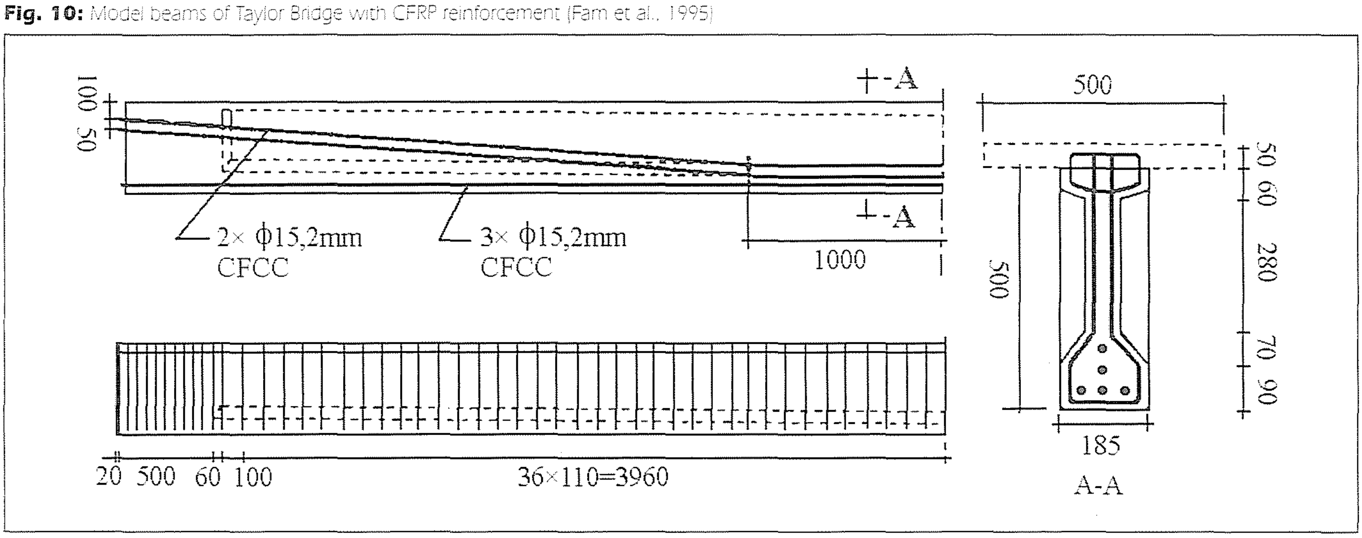
The second Canadian example is the Taylor Bridge over the Assiniboine River at Headingley, Manitoba, which was opened in October 1997 (Rizkalla et al., 1998). The total length of the bridge is 165 m, divided into equal simple spans by four piers. The bridge girders are 8 pretensioned I-beams at each span. Height of the girders is 1.8 m. The bridge consists of four girders with Carbon Fibre Reinforced Polymer reinforcement. Two beams were cast using ∅15.2 mm CFCC® strands, and another two with Æ10 mm Leadline® tendons. In the deck slab a portion of mesh reinforcement was also replaced by ∅10 mm Leadline® CFRP reinforcement.

The Taylor Bridge is of great importance because of two applications. One is the use of CFRP stirrups as shear reinforcement. It is important to mention these because FRP stirrups can not be fabricated on site. Bent shapes with sharp edges can be produced by the manufacturer only, before setting the resin matrix. The other application is the use of draped CFRP prestressing tendons in the bridge girder, the first of its kind in the world. CFCC® prestressed elements consist of 32 straight and 14 draped strands, while the Leadline® prestressed units consist of 38 straight and 18 draped tendons. Draping can be done only by slight angles; e.g. transportation of FRP products is possible by means of rolls of the diameter of 2 m. A further aspect of the superstructure is the shear force transfer between girders and the deck slab, which is allowed by stirrups overhanging the top surfaces of girders. Therefore the shear capacity of CFRP dowel bars can be also be examined in this structure.

Tests were carried out on model beams of ratio 1:3.6 to estimate structural behaviour of Assiniboine River bridge girders (Fam et al., 1995). The effective depth-to-span ratio was the same for test beams and the bridge girders. A total of six I-beams were fabricated with 9.3 m length and 500 mm height. After 7 days a 50 mm deep, 500 mm wide top slab was cast to provide composite action similar to the actual bridge deck slab. Cross sectional as well as longitudinal details of test beams prestressed with CFCC® strands are indicated in *Fig. 10.* Similarly to the bridge girders, 40% of the prestressing tendons were draped with a 4° angle. All the stirrups were overhanging the top surfaces of the beams into the slabs to provide dowel action. The strength of the stirrups was found to be about 45% of the uniaxial tensile strength of the CFRP reinforcement due to the inclination of shear cracks not perpendicular to the direction of the fibres. Other researchers also confirmed this finding. For what concerns the dowel action of CFRP bars it is seems to be adequate to transfer forces between the top slab and the beam.

The Taylor Bridge won the Harry H. Edwards Industry Advancement Award in 1998. The jury comments: "*This project features an unusual design that takes advantage of CFRP in both the prestressing tendons and reinforcement. This technique offers great potential for the future. This approach*

Fig. 10: Model beams of Taylor Bridge with CFRP reinforcement (Fam et al., 1995)



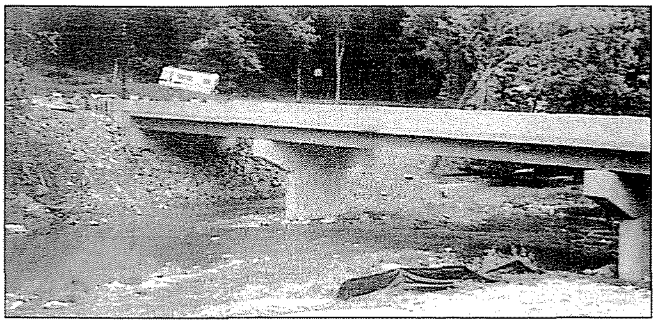


Fig. 11: View of McKinleyville Bridge (Thippeswamy et al., 1998)

should create an excellent ability to resist corrosion and should be able to increase the load carrying capacity as well. That latter element provides a very attractive feature."

The third, briefly introduced example is the McKinleyville Bridge, West Virginia, USA (Thippeswamy et al., 1998). The two-lane bridge was opened to traffic in September 1996 (see *Fig. 11*). The composite superstructure consists of rolled steel I-beams with cast in-situ GFRP reinforced deck slabs. The total length of the three-span bridge is 54 m and the 330×1300 mm steel girders are spaced at 1.5 m. In the bridge deck (thickness of 229 mm) ∅13/152 mm GFRP main reinforcement and ∅10/152 mm GFRP distribution reinforcement was applied. Therefore, clear cover is 38 mm on the top and 25 mm on the bottom of the deck. Two different types of GFRP reinforcement were used: one was C-BAR® (Marshall; introduced before) and the other one was a helical patterned and sand coated GFRP bar (producer: Grating International). Detailing of top and bottom reinforcement with epoxy coated chairs is indicated in *Fig. 12*. It is to be mentioned here that the density of GFRP reinforcement is approximately 16 kN/m^3, which can cause flotation of reinforcement (thus altering concrete cover) during vibrating (applied density of concrete was 24 kN/m^3 in this case). To avoid flotation, the GFRP mesh was tied down to the formwork as well as against the chairs in both directions (see *Fig. 12*). Due to the flexibility of GFRP reinforcement, chairs have to be spaced closely to avoid damage due to the weight of the construction crew and concreting equipment.

Field monitoring of the structure under service was carried out at regular intervals until the summer of 2000.

4.3 Cable stayed bridges with FRP tendons

A critical structural aspect of the design of very long-span bridges, if a suspension or a cable stayed bridge, is the self-weight. The maximum theoretical span of a suspension bridge is about 5000 m, using high strength structural steel. However, using high strength, low-weight Carbon Fibre Reinforced Polymer or Aramid Fibre Reinforced Polymer tendons this theoretical limit can be spread to between 10000...14000 m. This considerable difference throws light on the possibility of an economic application of FRP materials in suspension or cable stayed bridges. Moreover this can be the only possible material in the case of very long spans above (the arguments do not include aerodynamic aspects).

Fig. 12: FRP bars of the deck of McKinleyville Bridge (Thippeswamy et al., 1998)

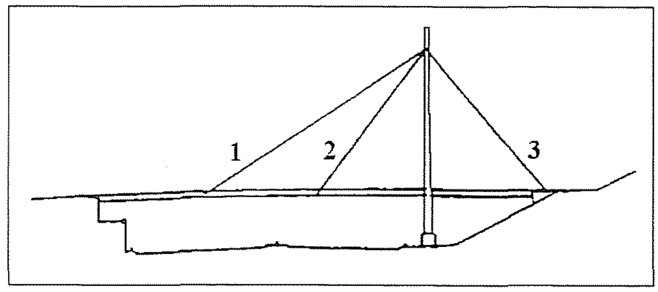


Fig. 13: Sketch of cable lay-out of Box Lane pedestrian bridge (Head, 1996)

For this reason many researchers have begun to deal with this topic, but a lot of questions have yet to be answered, such as (Head, 1996):

- aerodynamic stability under wind load,
- could FRP cables be damaged (due to its low transverse strength) during construction, lighting strikes, accident or vandalism?,
- is it possible for ice to erode the surfaces of FRP cables?,
- could the cyclic and long term strength of FRPs differ from the extrapolations of experiments?,
- how large can the allowable design stress in cables be?, etc.

Only a few applications can be found of non-metallic tendons in cable stayed bridges worldwide. Strong development of this field is to be expected.

The first use of Carbon Fibre Reinforced Polymer tendons as stay cables was in Winterthur, Switzerland in 1996, as part of the bridge at Storchenbrücke (BBR, 1996). The 124 m long, single-pylon cable stayed bridge has steel main girders of spans of 63 m and 61 m. From the total 24 stay cables two were substituted by CFRP cables. As the outer diameter of the original steel cables and the CFRP cables are almost the same, the appearance of the bridge is not disturbed. Strains are continuously measured in these CFRP stay cables.

Two prototype cable stayed bridges can be found in the UK: one with Aramid Fibre Reinforced Polymer stay cables (Aberfeldy Bridge), and the other with Carbon Fibre Reinforced Polymer stay cables (Box Lane Bridge, Staffordshire). A sketch of the cable arrangement of the latter is indicated in *Fig. 13* (Head, 1996). The bridge has an asymmetric cable arrangement, steel main girder and tower, spans are 38.55m and 12.60 m and cable lengths are 19.5, 20.5 and 28.9 m.

5. STRENGTHENING WITH FRP TENDONS

Strengthening of bridges by use of cross section post-tensioned cables has successfully been used for many years. This method is used mainly in those cases whenever excessive crack widths or deflections occur due to increase in traffic loads, overloading the structure or even inaccurate consideration of loads during design. This method allows the increase of both moment and shear capacity.

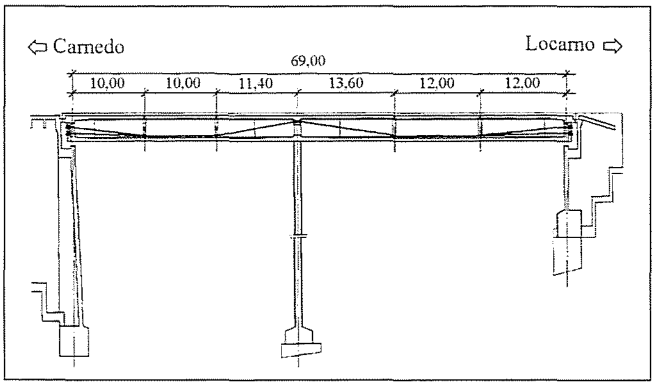


Fig. 14: Side view of the bridge over the Ri di Verdasio (Meier, 2000)

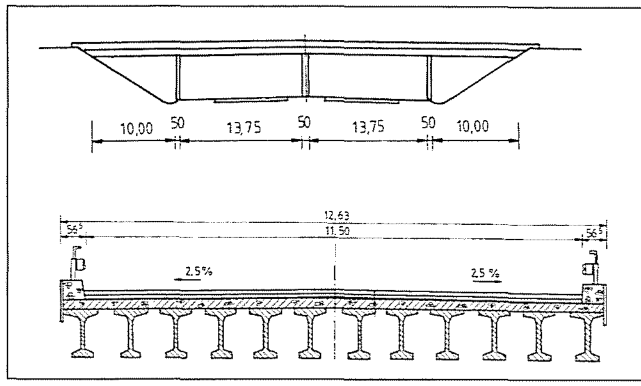


Fig. 15: Sketch of a usual prestressed concrete higway bridge

Strengthening of bridges can be needed due to advanced corrosion as well (e.g. rupture of prestressing tendons). In these cases an aggressive environment can be present also after strengthening of the structure, therefore, the use of corrosion resistant Fibre Reinforced Polymer tendons can be practical.

Such strengthening of a two-span two-lane box section prestressed concrete highway bridge was applied in the autumn 1998, over the Ri di Verdasio, Intragna (Meier, 2000). Spans have 31.4 m and 37.6 m length, the middle support is a 25 m high slim column while the width of the superstructure is 6 m. During a routine diagnostics inspection of the bridge, soaked areas and signs of inner corrosion were found. Later a detailed analysis proved that the chloride-content of the structural concrete (related to the mass of cement) was 2.8% at the level of the reinforcement and 2.0% at level of the prestressing tendons. Let us here remind ourselves of that the allowable maximum chloride-content of structural concrete is 0.4% when related to the mass of cement. During the 14 years of service the reinforcement in the bridge was corroded by almost 100% in some critical localities.

The strengthening of the structure was carried out by applying 4 draped Carbon Fibre Reinforced Polymer cables, each of which contained 19 pieces of Ø5 mm CFRP wires. The initial level of prestress was 1610 N/mm^2, that is 65% of the tensile strength of the material. Some preliminary laboratory tests organised by both the producer (BBR Ltd.) and EMPA at Dübendorf were carried out before the construction. These tests dealt with the proper anchoring devices of the post-tensioning of CFRP cables as well as the maximum draping radius allowable for this material. This latter was 3.0 m due to the relatively low transverse strength of Carbon Fibre Reinforced Polymer wires. An elevation of the bridge with the tendon layout can be seen in *Fig. 14*.

6. CONSIDERATIONS OF ECONOMIC USE

The economical use of a bridge is affected by many parameters such as function and location of the bridge, the construction materials applied, type of the superstructure and substructure, the construction techniques practised, transportation, length of period of construction, the securing of normal traffic during construction, considerations of connecting structures or the maintenance costs as the most important aspects. All these parameters have an effect on overall costs which have to be taken into account during design. Costs basically can be subdivided into two parts: the capital cost of construction and the operating/maintenance cost. It is misleading to examine economical use of a bridge taking into account only the capital cost.

The construction of a bridge functioning with minimal total cost over its whole life cycle with expected serviceability needs different views of design. Use of such methods of engineering economics are needed, which can compare the Present Value of all costs and incomes during the whole life cycle considering an allowable level of rate of interest (Balázs – Almakt – Erdélyi, 1998). In the case of bridge structures incomes usually can not be taken into account (derived from the function), therefore the Life Cycle Costs should be the basis of the decision. With this view of decision-making the economic use of Fibre Reinforced Polymer materials can be proved with much lower maintenance cost besides the higher initial cost.

For example, consider the sketch of a usual prestressed concrete highway bridge with prestressed precast bridge girders indicated in *Fig. 15*. The four-span four-lane superstructure has an overall length of 40 m and effective width of 11.5 m and 12 prestressed precast I-beams as girders in each span. Let us suppose there is no need of pile foundations. The overall initial cost of the construction can be estimated at 3 billion HUF (10 million USD) from which the total cost of girders can be estimated at 30 million HUF (100,000 USD). Considering the use of CFRP reinforcement and prestressing tendons the cost of the girders can increase to 60 million HUF (200,000 USD) which is inclusive of the labour cost increase as well. This means a 1% increase of initial cost of the bridge. Assumed maintenance/repair cost is 20% of the initial cost in every *ten* year period, so the Life Cycle Cost of the original structure with service life of 40 years is 3.82 billion HUF (12.7 million USD) (with 5% rate of interest). Supposing the decrease of maintenance/repair cost to 20% of the initial cost in every *twenty* years, the Life Cycle Cost of the structure with CFRP reinforcement in the girders with a service life of 40 years changes to 3.38 billion HUF (11.3 million USD) (with 5% rate of interest). Therefore the most economical solution is the use of non-metallic reinforcement in the girders.

7. STANDARDISATION

Widespread, well-known construction materials and successfully used structural details make possible quick and safe design in civil engineering practice. The development and use of new structural materials in any case may need new design methods and essentially, publishing of design recommendations, manuals and standards which consider different characteristics of the new materials. Considerable efforts have been made so far in this field, but there has not yet been produced a comprehensive standard. Japanese, American, Canadian and European design and/or construction guidelines are available but none of them with obligatory use. To promote standardising work the ***fib*** (fédération internationale du béton) and the ACI (American Concrete Institute) have regular work-groups which deal with the subject of non-metallic reinforcement for concrete structures.

8. CONCLUSIONS

The deterioration of bridges due to corrosion has turned the attention of structural engineers to the possible use of non-metallic (therefore non-corrosive, FRP) reinforcement in concrete structures.

Examples in this paper from Japan, Canada, the USA, Switzerland and Germany have shown that non-metallic (FRP) reinforcement can not only be successfully used in bridges but that their design and construction need special considerations.

Non-metallic reinforcement is made of Fibre Reinforced Polymers. Fibres can be glass, aramid or carbon fibres. The resin matrix is usually epoxy. Mechanical characteristics of the fibres (e.g. strength in the longitudinal direction, fatigue strength, long-term strength) are better than that of steel prestressing materials. Young's moduli of FRPs can be higher or lower than that of steel.

FRPs are linearly elastic and brittle materials. Therefore, the risk of brittle failure has to be taken into account during design. Difficulties can also arise in the case of anchoring of prestressed members.

Non-metallic (FRP) reinforcement can be used in bridges as:

1. reinforcement or prestressing tendons,
2. stay cables in cable stayed bridges, or
3. post-tensioned strengthening elements.

The application of non-metallic (FRP) prestressing tendons in bridge girders for durability purposes can be a promising economical alternative when considering the Life Cycle Cost of structures.

The authors hope that in Hungary the first prototype bridge with non-metallic (FRP) reinforcement will be constructed in the near future and that through this construction prove advantages of these materials with regard to the durability aspects of bridges.

9. ACKNOWLEDGEMENT

The authors greatly acknowledge the financial support of the (ÁKMI), of the project "Non-metallic reinforcement for bridge applications", project No. 3810.3.3/2001.

Also thanks to Dr. Erdélyi, L. (e&h kft.) and Mr. Almakt, M. (BUTE, Dept. of Bridges and Structures) for their help in some details of this paper.

10. REFERENCES

Balázs Gy. editor (1991), "Corrosion protection of highway bridges", *Research Report, BUTE, Department of Construction Materials*, Budapest, 1991.

Balázs, L. Gy. – Almakt, M. – Erdélyi, L. (1998), "Non-metallic (FRP) reinforcements for bridge girders", *5. Research Report to OTKA No. T 016996*, Budapest University of Technology, 1998.

Balázs, L. Gy. – Farkas, Gy. – Erdélyi, L. – Borosnyói, A. – Almakt, M. (1999), "Non-metallic prestressing systems", *Summary Report to OTKA No. T 016996*, Budapest University of Technology, 1999.

BBR (1996), "BBR Review – Introducing BBR CARBON STAY Technology", *Bureau BBR Ltd.*, Zurich, Switzerland, November 1996.

Crivelli Visconti, I., (editor) (1998), "ECCM-8 European Conference on Composite Materials – Science, Technologies and Applications", *Proceedings*, Naples, Italy, June 1998.

El-Badry, M.M., (editor) (1996), "Advanced Composite Materials in Bridges and Structures", *Proceedings*, 2nd International Conference. Canadian Society for Civil Engineering, Montreal, Quebec 1996.

Fam, A. Z. – Abdelrahman, A. A. – Rizkalla, S. H. – Saltzberg, W. (1995): FRP Flexural and Shear Reinforcements for Highway Bridges in Manitoba, Canada. *Proceedings of the Second International RILEM Symposium (FRPRCS-2)*, Ghent 1995., L.Taerwe, Editor, E FN Spon, London. pp. 395…402.

FRP International – *Quarterly Technical Paper*, Editor S.H. Rizkalla (*ACI, ASCE, CSCE, Composite Institute, JCI, ACMBS Network of Canada, ISIS Canada*). Volume V., Issue 2, Spring 1997.

Head, P. R. (1996), "Advanced Composites in Civil Engineering – A Critical Overview at This High Interest, Low Use Stage of Development", *Advanced Composite Materials in Bridges and Structures*, M.M. El - Badry, Editor, Canadian Society for Civil Engineering, Montreal, Quebec 1996. pp. 3-15.

Japan Concrete Institute (1997), "Non-Metallic (FRP) Reinforcement for Concrete Structures", *Proceedings of the Third International RILEM Symposium (FRPRCS-3)*, October 1997, Sapporo, Japan.

Japan Prestressed Concrete Engineering Association (JPCEA) (1998), "Prestressed Concrete in Japan", XIII. FIP Congress, *National Report*, Amsterdam, Holland, 1998

Kollár L. – Kiss R. (1998), "SzálerQsítéses mqanyagok (kompozitok) az építQiparban – I. Kompozitok anyagai", *Közúti és Mélyépítési Szemle*, XLVII. évf. 9. szám, pp. 331-338.

Marshall Industries Composites, Inc., (1995), "C-Bar® Reinforcing Rods. The Future of Concrete Reinforcement", *Technical Data*.

Meier, U. (2000), „Spannglieder aus CFK", *Proceedings*, Massivbau 2000 – Forschung, Entwicklungen und Anwendungen, 4. Münchener Massivbau-Seminar 2000, Technische Universität München, Springer-VDI-Verlag, 2000. pp. 205-216.

Rizkalla, S. H. – Shehata, E. – Abdelrahman, A. A. – Tadros, G. (1998): The New Generation – Design and construction of a highway bridge with CFRP. *Concrete International*, June 1998, pp. 35…38.

Rizkalla, S. H. – Tadros, G. (1994), „A Smart Highway Bridge in Canada", *Concrete International*, Vol. 16., No. 6., June 1994, pp. 42-44.

Rubinsky, A. – Rubinsky, I. A. (1959), „A Preliminary Investigation of the Use of Fiber Glass for Prestressed Concrete", *Magazine of Concrete Research*, Sept. 1959. pp. 71-78.

Schöck (1997), „Schöck ComBAR®, Betonbewehrungselemente aus Glasfaserverstärktem Kunststoff", *Schöck Bauteile GmbH Manuel*

Taerwe, L., Editor (1995), "Non-Metallic (FRP) Reinforcement for Concrete Structures", *Proceedings of the Second International RILEM Symposium (FRPRCS-2)*, Ghent 1995, E FN Spon, London.

Thippeswamy, H. K. et al. (1998), "FRP Reinforcement in Bridge Deck", *Concrete International*, Vol. 20., No. 6., June 1998, pp. 47-50.

Tokyo Rope (1993), "Technical Data on CFCC®", Tokyo Rope Mfg. Co., Ltd. *Manual*, Tokyo, October 1993.

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REPAIR OF A 100 METRE HIGH VENTILATION CHIMNEY GROUP

Dr. József Almási – Prof. Árpád Orosz

The condition of four 100 metre high ventilation chimneys of the Paks Nuclear Power Station deteriorated to an extent that repairs could no longer be postponed. The deficiencies of the construction technology originally used, the lack of compaction at the time of the concrete pour, the uneven material strength and several decades of weather related damages resulted in a condition endangering structural integrity.

According to structural analysis, resistance of the structure to wind load is adequate, but the structures did not have a proper margin of safety for earthquake loads. An evaluation of the available alternatives revealed that the strengthening of the existing structure would be more cost effective than building a new chimney. The repair, or rather the strengthening, of the chimneys was accomplished by shotcreting of the inner and outer surface of the chimneys using wiremesh layers of reinforcement. The repair of the loose, high void ratio structural concrete, the increase of the material strength and improved consistency in strength was achieved by injection of microcement paste. The flexible method of repair suitable to the circumstances in conjunction with the highly organised and well executed quality assurance program necessitated by the significance of the structure greatly contributed to the success of the strengthening.

Keywords: repair, strengthening, shotcrete, injection, fiber reinforced concrete, quality assurance, repair methods

1. INTRODUCTION – THE CONDITION OF THE CHIMNEYS PRIOR TO STRENGHTENING

The ventilation chimneys were erected between 1980 and 1982 applying slip form technology. Over time the concrete failed in many locations. The failures were manifested by falling loose concrete chunks, the appearance of holes in the wall of the structure and the onset of corrosion of the reinforcement.

Fig. 1: The middle portions of the third and fourth chimneys prior to strengthening

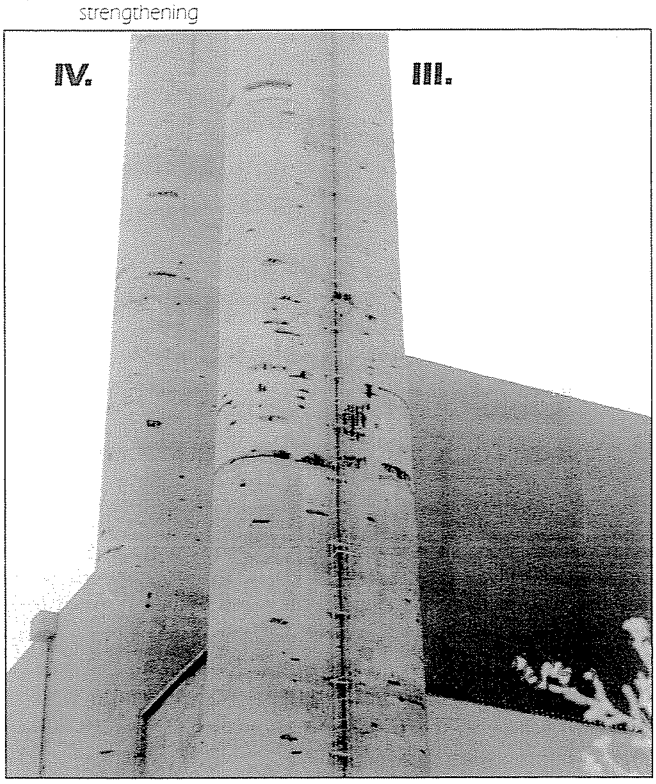


Fig. 2: A detail of a deteriorated area of the structure

Based on an on-site inspection and a detailed examination of the structure it was concluded that:

- in many places the concrete was loose in structure,
- density of the concrete was not uniform,
- strength of the concrete was low and easily chiseled away,
- along the height of the chimney at 0.50 to 0.70 m intervals horizontal cracks were present,
- rainwater seeped through the wall of the structure resulting in frost damage,
- strength of the concrete was uneven,
- thickness of the cover over the reinforcement varied greatly,
- corrosion was evident at the locations of exposed reinforcement,
- joints of the circular reinforcements were exposed (*Figs. 1 and 2*).

2. THE 'AS DESIGNED' CONDITION OF THE CHIMNEYS

The design and general arrangements of the chimneys is briefly described in the following. The cross-sections shown in *Fig. 3*

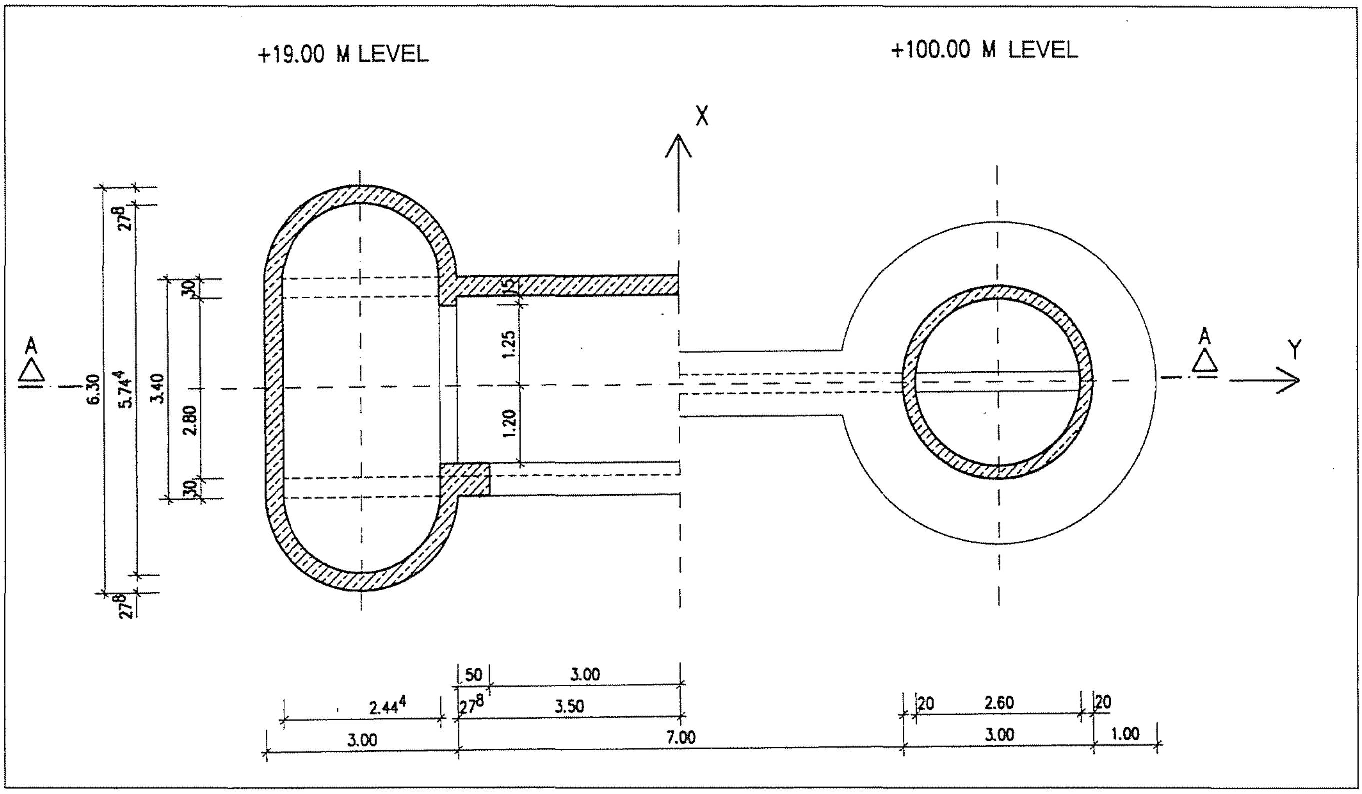
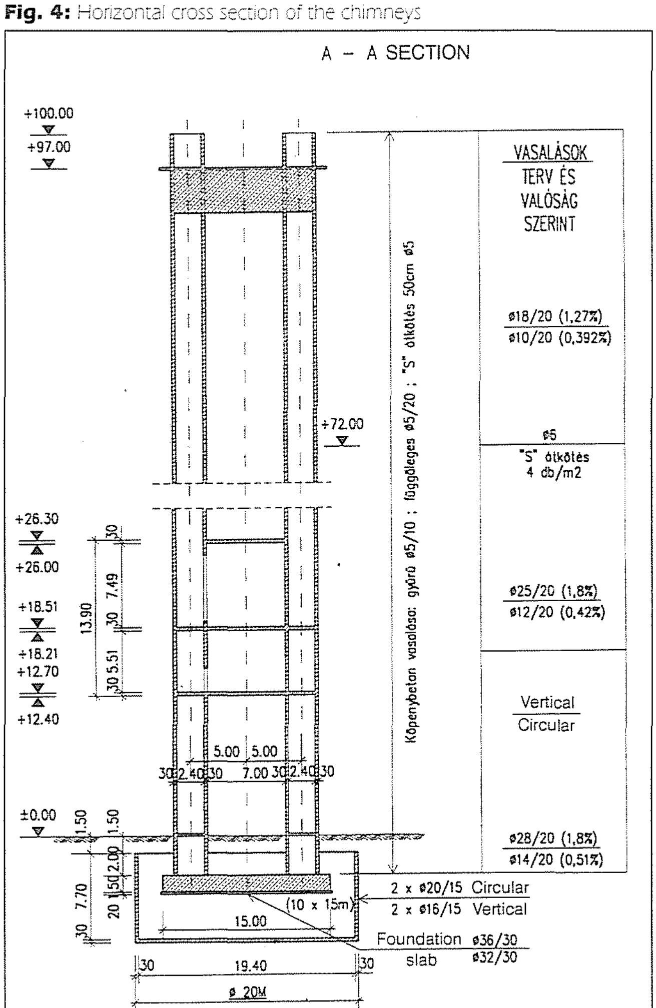


Fig. 3: Horizontal cross section of the chimneys

Fig. 4: Horizontal cross section of the chimneys



illustrate the twin arrangement of the chimneys. At the level of the foundation, 3-meter diameter semicircles connected with a straight section results in a cross sectional measure of 7 m. At the highest elevation a 3 m diameter circular cross section is achieved by the gradual reduction of the length of the straight section connecting the semicircles. The wall thickness at the bottom is 30 cm and 20 cm at the top.

The twin towers are at an axial distance of 10 meters and they are connected at certain elevations (*Fig. 4*).

The structure was built with a complicated geometry. It can be considered as a frame structure across the smaller cross sections whereas in the direction of the larger cross section it can be considered as a cantilevered beam.

The design documents specified us of concrete B280 and steel reinforcement B.50.36. The vertical reinforcement at the lower elevations was ∅20/20 cm both at the outside and inside corresponding to a reinforcement ratio of 1.8%, whereas at the higher elevations it was ∅18/20 cm again both outside and inside, resulting in a reinforcement ratio of 1.27%. The hoop reinforcement was ∅14/20 cm both at the outside and inside and the reinforcement ratio attained was 0.39%.

The inner and outer reinforcement grids were connected at 50 cm × 50 cm intervals with 6 mm bars as detailed in the design.

3. CONCLUSIONS OF THE STRUCTURAL ANALYSIS CONDUCTED PRIOR TO STRENGTHENING

The on-site observations of the structure were followed up with a structural analysis by Almási & Orosz in 1977.

The following loads were considered in the structural analysis: the weight of the structure, the wind load increased by a factor to take into account dynamic effects, an earthquake load with a 0.1 g acceleration factor, thermal loads due to temperature changes of the chimney walls.

The stresses in the Y directions resulting from the wind loads are presented in *Tables 1 and 2*.

Table 1: Stresses due to wind loads, Y direction

Forces		Stresses [N/mm²]			
M and N	[kNm kN]	Concrete stresses	C10	C5	C0 (steel only)
			Concrete grade		
M_{ymax}	10 632	σ_c	–3,7	–2,8	–220
N_{min}	4 618	σ_{ct}	1,8	1,3	105
M_{ymin}	9 099	σ_c	–3,6	–2,70	–213
M_{max}	5 644	σ_{ct}	1,25	0,93	73

Table 2: Stresses due to wind loads, X direction

Forces [kNm kN]		Stresses [N/mm²]		
M and N	Concrete stresses	C10	C5	C0 (steel only)
		Concrete grade		
M_{max} = 17 901	σ_c	–3,65	–2,8	–239
N_{max} = 8 113	σ_{ct}	–0,67	–0,5	43

The stresses were calculated assuming concrete strength classes C10, C5 and C0 (i.e. assuming that the integrity of the reinforcing steel is maintained only). Due to the danger of buckling the allowed stresses in the reinforcement were limited to a value of 210 N/mm2.

The results show that the cross section in the Y direction is not free of cracks and that in case of a uniform concrete cross section the compression stresses are relatively low.

For wind loads the maximum load limit was calculated using combined loads as prescribed in the Hungarian Standard 15022/1.

The calculated values are presented in *Tables 3 and 4*.

Table 3: Y direction at elevation + 26.3 m

Forces [kNm, kN]	"K" value with σ_c [N/mm²]		
M and N	5,0	2,0	0,0
$M_{y.max}$ = 10 632 $M_{x.max}$ = 2 281 N_{min} = 4 618	0,402 < 1	0,465 < 1	0,61 < 1

Table 4: X directions at elevation –3.15 m

Forces [kNm, kN]	"K" value with σ_c [N/mm²]		
M and N	5,0	2,0	0,0
$M_{y.max}$ = 4 464 $M_{x.max}$ = 17 901 N_{max} = 8 113	0,434 < 1	0,589 < 1	0,635 < 1

Therefore, in spite of the low strength of the concrete, the structure can adequately resist wind-loads due to the abundant reinforcement. It can be shown that the reinforcement alone is sufficient to carry the loads, even when considering the limits imposed by the possibility of buckling. The primary role of the concrete in this case is the prevention of buckling.

The seismic forces resulting from a 0.1g ground acceleration creates the limiting stresses presented in Table 5. *Therefore, in areas of lesser strength the load-bearing capacity is insufficient, reinforcement is required and can be attained by the strengthening of the concrete. Stability was considered and the foundation was examined to verify the limiting stresses: the resulting values are shown in Table 6.*

Table 5: Analysis of seismic loads at elevation –3.15 m

Forces [kNm, kN]	"K" value with σ_{cd} [N/mm²]		
M and N	5,0	2,0	0,0
M_x = 33 366 M_y = 810 N = 8 113	0,744 < 1	1,041 > 1	1,736 >> 1

Table 6: Earth pressure under the base

Forces M and n [kNm, kN]	Earth pressure
M_y = 44 031 N_y = 24 026	σ_{min} = –307 kN/m² < 400 kN/m² σ_{max} = –92 kN/m²
M_x =36 699 N_x = 20 002	σ_{min} = –314 kN/m² < 400 kN/m² σ_{max} = –21 kN/m²

Therefore the soil pressure does not exceed the 400 kN/m2 value.

The factor of safety for stability is:

$$k_{sy} = \frac{M_{sy}}{M_{sby}} = 5{,}50 \quad \text{and} \quad k_{sx} = \frac{M_{sx}}{M_{sbx}} = 3{,}4 > 1{,}8$$

There is an adequate factor of safety against overturning moments.

Based on structural analysis and on site inspection it was determined that:

- the chimneys are adequate for the usual loads (wind, dead loads and loads resulting from temperature differentials),
- integrity of the structure for a 0.1 g ground acceleration seismic load can not be assured,
- it is necessary to strengthen the Chimneys due to the significant deterioration of the concrete and the onset of corrosion in the reinforcement.

Considering the results of the analysis a recommendations are made to select a strengthening technology.

4. SEISMIC ANALYSIS OF THE CHIMNEYS

Subsequent seismic analysis of the chimneys was conducted using a 0.25 g ground acceleration prescribed by the code in effect (cca. 10th grade on MKS scale) and was conducted by Almási and Orosz in 1998. The determination of the loads was achieved with the cooperation of J. Györgyi and A. Lovas of the Technical University of Budapest. The analysis was made in consultation with Dr. B. Kovács of EMI Inc. We are grateful for their contributions. The purpose of the analysis was to determine:

- the stresses due to extreme events, considering the given geometry of the chimneys,
- the load resisting capacity of the cross sections taking into account the reinforcement in place and the additional reinforcements embedded after the strengthening is complete,

Elevation considered m	$N_{min;0.25}$ (kN)	$N_{max;0.25}$ (kN)	$M_{y;0.25}$ (kNm)	$M_{x;0.25}$ (kNm)	$V_{x;0.25}$ (kN)	$V_{y;0.25}$ (kN)	$T_{c;0.25}$ (kNm)
-3,16	-3496,50	-22356,90	24416,50	50988,10	2374,80	1748,90	294,90
26,30	-3353,80	-11546,30	17432,60	22785,70	1333,50	1010,30	1083,50
55,00	-944,30	-6945,10	10659,90	16780,50	698,90	577,40	787,50
80,00	771,90	-4093,90	10659,90	7976,70	698,90	535,70	343,40

Table 7: The resulting forces in the truss model for a 0.25g ground acceleration

Elevation considered m	$N_{min;0.1}$ (kN)	$N_{max;0.1}$ (kN)	$M_{y;0.1}$ (kNm)	$M_{x;0.1}$ (kNm)	$V_{x;0.1}$ (kN)	$V_{y;0.1}$ (kN)	$T_{c,0;25}$ (kNm)
-3,16	-9154,60	-16698,80	9766,60	20395,20	950,00	699,90	118,00
26,30	-5811,50	-9088,50	6973,00	9114,30	533,40	404,10	433,40
55,00	-2744,50	-5144,90	4264,00	6712,20	279,60	231,00	315,00
80,00	-687,80	-2634,20	4264,00	3190,70	279,60	214,30	137,40

Table 8: The resulting forces in the truss model for a 0.1g ground acceleration

- strength of concrete required to meet the design criteria given the calculated stresses and the amount of reinforcement used,
- strength of concrete that can be attained using the technology employed in strengthening.

The base values of the loads were determined by a model analysis where the discretization of the structure resulted in a model consisting of truss elements. The results are presented in *Tables 7 and 8.*

The values for a 0.1g ground acceleration were determined by interpolating the values determined for the 0.25g ground acceleration. The design loads were arrived at by considering the recommendations of EC8. The values are shown in *Tables 9 and 10.* Note that in the calculation of the design moments an over load factor (ℓ_0,) was used, which is determined from the stresses due to reinforcement stiffening and form the limiting stress. The values of ℓ_0, is usually between 1.3 and 1.5.

In the calculation of the design shear the q_0 value that is characteristic for this structure was used. This value reflects the structure's ability to absorb energy, the ductility of the structure, the change of the structure along the elevation points and the structure's failure modes. The values of the q_0 factor are usually between 2.0 and 5.0.

The loads determined on the truss element were further divided and apportioned to the shell elements used in a refined model.

The goal of refining the model was to permit the analysis of the local behaviour of the structure when a general cross section of the structure is under consideration. This way the relationship of global and local failure modes can be examined. The details of the analysis are presented elsewhere. In conclusion the local failures generally occur at higher loads compared to loads causing the failure of the cross section of the structure. Therefore greater attention was focused on determining the load-bearing capacity of each representative cross section. Combined loads were used in determining the load-bearing capacity of a cross section of the chimney. Therefore a given load was used in determining the ratio of the design moment to the limiting moment (see Hungarian Standard 15022/1).

Table 9: Design loads for the case of a 0.25 g ground acceleration

Elevation considered	$N_{MIN.Sd}$ (kN)	$N_{MAX.Sd}$ (kN)	$M_{Y.Sd}$ (kNm)	$M_{X.Sd}$ (kNm)	$V_{X.Sd}$ (kN)	$V_{Y.Sd}$ (kN)	T_{sd} (kNm)
-3,16	-384,50	-25468,90	32473,90	67814,10	7124,40	5246,80	884,00
26,30	-956,00	-17090,60	23185,40	35130,00	4000,50	3030,90	3250,00
55,00	45,80	-7935,20	14177,70	24751,00	2096,70	1732,20	2362,00
80,00	1574,70	-4896,70	14177,70	16683,50	2096,70	1607,10	1030,00

Table 10: Design loads for the case of a .1g ground acceleration

Elevation considered	$N_{MIN.Sd}$ (kN)	$N_{MAX.Sd}$ (kN)	$M_{Y.Sd}$ (kNm)	$M_{X.Sd}$ (kNm)	$V_{X.Sd}$ (kN)	$V_{Y.Sd}$ (kN)	T_{sd} (kNm)
-3,16	-7909,20	-17942,80	12989,30	27125,60	2848,80	2097,60	353,90
26,30	-5271,00	-9629,10	9273,80	14052,00	1600,20	1212,40	1300,20
55,00	-2348,50	-5540,90	5671,08	9900,40	838,70	692,90	945,00
80,00	-366,70	-2955,30	5671,08	6673,40	838,70	642,80	412,10

The shear resistance available at a cross section was determined for two different modes of failure. A diagonal shear and a horizontal shear failure along access points. For diagonal shear the stresses resulting from shear and bending moments were combined using the guidelines in Hungarian Standard 15022/1.

For horizontal shear the combination of V_{dd} and the V_{fd} resulting from granular friction were summed to arrive at the design shear.

The mentioned calculations were completed for a given reinforcement configuration for various grades of concrete. The concrete grade was deemed acceptable only when the cross section was adequate for the combined loads.

One result of the calculations was the determination of the required strength of concrete along the height of the structure. These were regarded as the values that will have to be attained by the strengthening. The result of the calculations are presented in diagram 5, for a ground acceleration of 0.18g. The first three columns of the diagram contains values of the original concrete strength, values for concrete strength that can be attained after shotcrete application and values for concrete strength that can be attained after shotcrete application and injection. "Attained" in this context means that the correct application of the prescribed technology makes it possible for the originally weak concrete with uneven strength to be repaired, resulting in a reconstituted material integrity. The subsequent columns of *diagram 5* also present those concrete grade values that are needed to resist the indicated forces. The last two columns indicate the actual strength attained with the completion of the repairs. Comparisons of the required and attained (by the strengthening) values indicates how the refurbished will chimney measure up to various loads. It should be mentioned that as originally anticipated the chimney is adequate for loads resulting form a ground acceleration of 0.1g, while unfortunately the attained strength is inadequate in the case of a 0.25g ground acceleration.

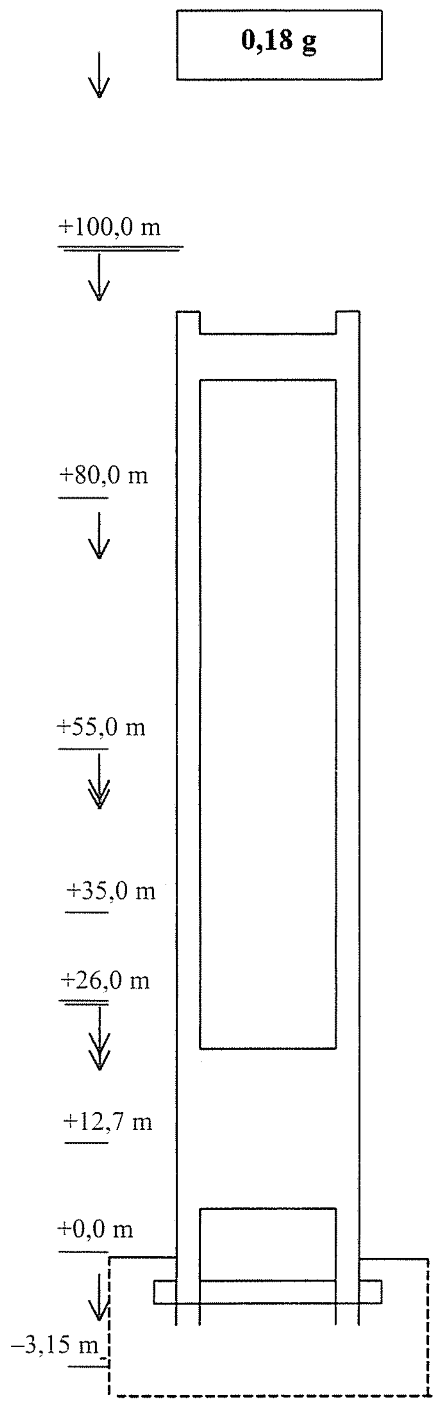
5. THE FIRST STAGE OF THE STRENTHTHENING AND QUALITY CONTROL

5.1 Initial criteria used to select the repair technology

The condition of the chimneys was determined by rappelling. Verticor Inc. was engaged to remove the loose concrete by chiseling it away and documenting the structural damage with photographs.

Based on this it was determined that:

- while building the chimneys there were construction inadequacies, concrete placement problems an improper formation of access points,
- the concrete is not uniform in consistency and strength, porous and easily chiseled away,
- Onset of the corrosion of the reinforcement was evident,
- due to weather changes and freezing the separation and falling of concrete chunks constitutes a work hazard,



THE ORIGINAL DESIGNED CONCRETE CLASS C16	The strength of concrete before repairs	Designed strength by repairing		The necessary concrete strength for 0,18 g load	The strength after repair with shotcrete and injection	
		Repair with shotcreate	Shotcreate and injection		III. chimney	IV. chimney
	C8	C9	C12	C12	C13	C16,8
	C8	C9	C12	C14	C22,7	C17,2
	C11	C12	C18	C18	C26,2	C21,9
	C14	C16	C20	C20	C24,5	C24,0
				Loads M + N and V		

Fig. 5: Strength of concrete prior to the repairs, the strength necessary based on structural calculations and the strength attained with the strengthening.

– the integrity of the structure is affected by these conditions,

A detailed analysis of the chimneys revealed that the refurbishing and strengthening of the structure could no longer be postponed. The deteriorated ability of the structure to resist extreme loads and maintain stability made this course of action necessary. Considering the extent and characteristics of the existing faults of the structure and reviewing the results of the structural analysis the consultants working for CEAC Inc. (the firm engaged to conduct the study) concluded that the repair and strengthening of the existing chimneys was a viable alternative to the dismantling of the existing structure and building new chimneys.

To select a method of strengthening, the following criteria were considered:

- the 'as designed' condition of the structure must be restored,
- uniform strength of the concrete must be assured,
- a reliable method must be found to protect the reinforcement against corrosion,
- the structure must be capable to resist loads due to extreme events with an adequate margin of safety,
- the strengthening of the foundation is to be avoided,
- the continuos operation of the Power Plant during repair must be assured.

5.2 The technological steps of the strengthening

The first variation of the repair, or rather strengthening method, was developed by CAEC Inc. with the contributions of VERTICOR Inc. and TechnoWato Inc. (László Csányi).

The primary technological processes prescribed at the outer surface were:

- cleaning of the concrete surface using high pressure wet sand blasting,
- removal of loose concrete by hand chiseling,
- treatment of the exposed reinforcement with a corrosion resistant substance,
- concreting of the large voids using suitable formwork,
- creation of an average 4 cm thick shotcrete layer applied over a wire mesh at the outer surface assuring the adhesion between the new and existing concrete,
- application of an elastic, reflective, moisture permeable surface protection compound,
- injection of the loose concrete, as necessary, to assure the homogeneous material consistency and uniform strength of the concrete.

The interior surface of the chimney could not be inspected satisfactorily due to the continuos operation of the power plant. The simple visual inspection did not permit the true state of the structure to be ascertained, and the assumptions made concerning the corrosion protection of the reinforcement were not born out.

Based on this, the following technological steps were prescribed for the interior of the chimney:

- The cleaning of the surface by high pressure sandblasting.
- The concreting of any large voids.
- The formation of an adhesive layer on the interior surface.
- The application of a 2 cm cement based stucco layer.

It was stipulated as a general rule that during the repair only compatible technologies and materials will be used. In addition it was an important consideration that the technologies or material used could be flexibly altered to suit the conditions without adverse effects. The final stipulation was that good quality materials were to be procured the repairs.

5.3 Quality control

The designers of the strengthening techniques paid detailed attention to the development of an organised quality control process of high intensity. The relevant regulations were summarized in a quality control document (VERTICOR 1997, Almási – Orosz 1997 October, 1999 June). The details of this are presented in Section 7.

The customer engaged The company Construction Quality and Innovation Inc. (ÉMI) to co-ordinate the overall quality control process. They provided helpful recommendations and insisted throughout out the repair work that only materials of proven and documented origin could be used and that materials applied were truly cement based and of high quality.

6. IMPROVEMENTS OF THE STRENGHTHENING TECHNOLGOGY

6.1 Results of an in-depth study and the necessity of improvements

Being familiar with the above directions, VERTICOR Inc. commenced the work on the structure. Using the hanging scaffolds that was made available, the examination of the concrete strength was the first task to be completed.

The destructive and non-destructive test of the core samples obtained revealed that the structure was in a far worst condition than previously thought. The concrete consisted of horizontal 30-60 cm deep layers of alternating strength and consistency. The amount of material that could be removed by hand chiseling was significant and the safe removal of this material presented a challenge. To assure structural integrity the amount of material removed at any time had to be limited. The erection of forms for the areas worked and the organization of the concreting of these areas was complex. The control of the quantity of material removed required constant engineering supervision. The on-site instrumental control of concrete quality was in many instances not possible. Furthermore, it was difficult to determine to a high degree of certainty whether the old and new concrete adhered properly.

During the erection of the chimney and due to raising of the formwork in stages, a tooth-type surface impression created on the internal surface of the chimney was observed. These surface imperfections made the application of the 2 cm cement-based mortar nearly impossible. Based on a two-month detailed investigation and analysis of the structure, the collaborators concluded that the planned technology needed to be improved and further enhanced.

6.2 Analysis of the operational requirements of the ventilation chimneys

In this situation the owner of the structure decided to slow the pace of the on-going repair work while a detailed analysis was conducted on the long-range operational requirements of the chimneys.

The following scenarios were considered:

- demolition of the existing chimneys and erection a new reinforced concrete or steel chimneys,
- a partial demolition and the erection of a steel upper portion,
- construction of a new load-bearing chimney in the interior of the existing chimneys,

The analysis of the above cases indicated that:

- continuos operation of the chimney could not be assured,
- alteration and strengthening of the foundation became necessary,
- costs increased to become exceedingly high.

6.3 Improvement of the strengthening technology

Prior to the above described analysis, and being aware of the results of the structural analysis, CAEC Inc. recommended improvements to the strengthening technology employed. The recommendations were seconded by EMI, who supervised the overall quality control. The most important characteristics of the improved strengthening technology were:

- instead of the removal of the loose, weak concrete, a reinforced concrete jacket would be created applied both on the outside and inside, to hold the structure together,
- the two layers (inside and outside) would be connected with reinforcement,
- the loose and low quality concrete would be improved by injection to create a homogenous concrete structure,
- the outside layer would be covered with a protection which would retard the formation of cracks,
- fibre reinforced shotcrete would be used on the outer layer.

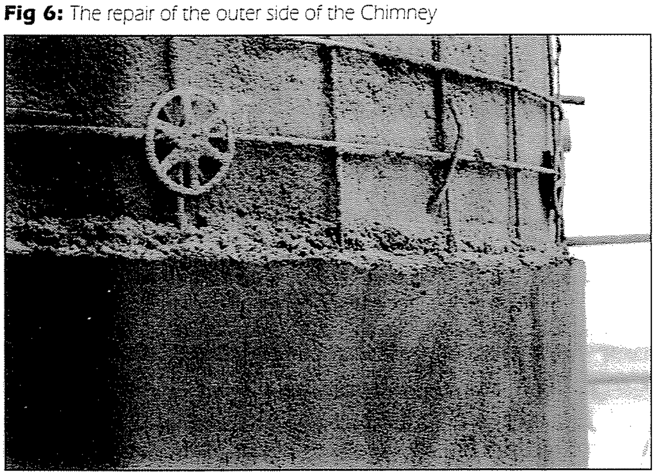
Fig. 6 illustrates the arrangement of the structure. The containment effect of the layers on both side can be illustrated by an analogy considering the load bearing ability of a sand bag: As long as the bag is not torn at the sides it can bear loads.

The advantages of employing inside and outside layers connected by reinforcement are illustrated in *Fig. 7*.

Specifically, if in the 'a' vertical cross-section the buckling in 'b' direction causes an outward displacement due to the supporting effect of the inner core, then in the 'c' horizontal direction on the inner side, the shortening of the jacket would result in a compressive force. The resultant of that force would try to restore the 'jacket' to its original shape. The situation would be similar on the outside, only here tensional forces would be present in the jacket. This effect is significant in arched sections, and a structure with a 'jacket' on both side would also respond favorably to dynamic loads.

The shotcrete jacket applied to both the inner and outer surfaces would result in a structure with an improved ability to resist earthquake loads. Compressing the weaker sections of concrete would increase the ability of the walls to undergo ductile deformation and therefore the overall ductility of the walls would be increased. The bi-directional cracks resulting from an earthquake would be resisted by the vertical and horizontal reinforcement. The compressed walls would protect the reinforcement against buckling and bending outward.

Fig 6: The repair of the outer side of the Chimney



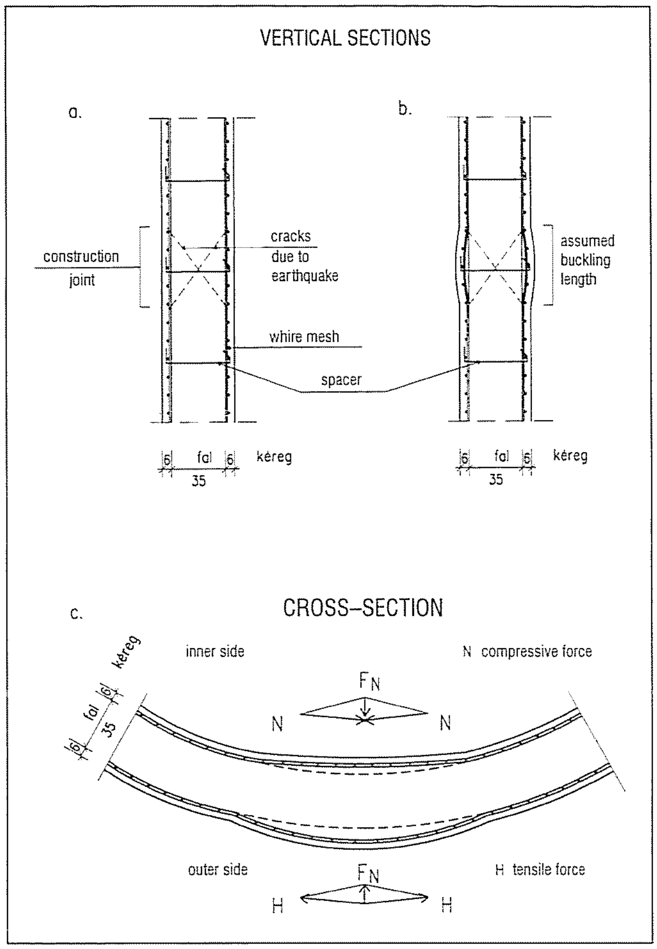


Fig 7: Illustration of the structural role of the "jacket"

The advantages of this modified strengthening technology can be summarized as follows:

- continuos operation of the power plant is assured,
- additional dead load represents only a 20% increase so there is no need to strengthen the foundation,
- the original design conditions can be attained,
- in addition to static loads, the resistance to earthquake loads is also improved,
- the costs incurred to increase adhesion of the concrete are nominal and there is no need to remove and transport material.

After some deliberation the owner decided to implement the modified version of the strengthening technology and subsequent work was carried out accordingly.

7. RESULTS OF THE QUALITY CONTROL

Continuous operation of the Power Plant requires a functioning ventilation chimney, therefore a custom-made monitoring and evaluation program was designed and detailed in a quality control document. The conditions elaborated in this document exceed those laid out in the standards and a more frequent and stricter inspection program was prescribed. The inspection plan was included, detailing the steps of the work procedures, the purpose of the work activities, the timing and the subject of the inspections, the tasks to be performed by the inspector, the testing methods and the methods to evaluate test results and also the daily, weekly and monthly reporting requirements.

In addition to engaging their own quality control inspector, an independent quality control group working under engineering supervision was organised to complete the works.

The most important quality control measures prescribed were:

- materials testing of:– the existing concrete, – the shotcrete, – the injected material, – the surface protection material which was subjected to both destructive and non-destructive testing.
- continuous supervision of the following work phases: – the cleaning of the surfaces, – assembly and placement of the welded mesh reinforcement, – application of the shotcrete, forming of work areas, – post placement treatment of the concrete.

The concrete samples obtained from the chimney were subjected to both destructive and non-destructive testing.

With regard to the destructive testing of concrete:

- 90 mm diameter cylindrical core samples were drilled at 5 different elevations (approximately 20 meters apart) at 4 locations at each elevation. The samples were obtained both from the original and repaired structure.
- 50 mm diameter drilled concrete cores were tear-tested, to measure the adhesion of the new and old concrete and to examined the tensile strengths. These were completed along the height in 5 m increments, at 4 locations at each elevation, to test the existing concrete and the shotcrete layer.

Non-destructive testing

- Schmidt hammer examination, at heights of 5 meters at 4 locations at each elevation. The testing was carried out on the old concrete, on the shotcrete layer applied and on the core samples. Consolidation of the results of the destructive and non-destructive testing allowed the creation of a diagram that is representative of the entire structure,
- strength of the injected material was tested by using 7 x7x7 cm cubes and Häðgermann prisms. The measurements obtained exceeded the C30 class.
- adhesion of the surface protection was tested using disks in a tear-off test, the results obtained exceeded the expectations.

Results of the concrete testing of the strengthened chimney are summarized in Section 9.

7.1 Characteristics of shotcrete

The shotcrete applied at the inner and outer surface of the structure was prepared using a factory premixed and packaged aggregate with 4-8 mm maximum grain size. Due to the considerable height of the structure the dry aggregate and the water were mixed in the spray gun nozzle that was used to apply the shotcrete.

Care was taken to ensure that the surfaces were properly wetted. First a layer consisting of a coarser aggregate mixture was applied. The shotcrete covering the welded wire mesh was made up of somewhat finer aggregate that was easier to work with to create a smoother surface.

The shotcrete mixture applied at the exterior surface of the chimney contained 1% fibre-reinforcement.

The quality control testing revealed that the shotcrete that was used could be classified as C30 strength grade, meeting the prescribed and guarantee conditions for concrete strength.

7.2 Deformation measurements

The weight of the chimneys increased about 20% as the result of the strengthening. According to structural analysis this additional dead load did not necessitate the reinforcement of the foundation.

Surveying techniques were used to measure the deformation of the structure and to establish the effects of seasonal temperature changes. The measured deformations were barely within the limits of accuracy of the instruments used. The vertical deformations of the original chimney wall due to compression was measured with a 250 mm base length deformations measuring-device and there was no measurable deformation.

The results of the quality control of the technological processes were reassuring. The application of the shotcrete was organized to employ two skilled workmen working simultaneously in opposite directions without a vertical gap while a level was completed. Special attention was afforded to the after placement treatment of the concrete which was necessitated by the great height and the relative thinness of the shotcrete layer. This special care and the addition of fibre-reinforcement resulted in a shotcrete layer practically free of cracks. The cracks that appeared were concentrated in gaps created in the course of work. These indicated that a surface treatment must be applied to protect the reinforcement against corrosion.

8. THE INJECTION OF THE CHIMNEYS, SURFACE PROTECTION

8.1 Method of injection

Both the initially proposed and the improved versions of the strengthening technology included provisions to deal with the lack of homogeneous material structure by injecting the loose and low-grade concrete of the chimney.

The selection of the injection method was done with the collaboration of Prof. Gy. Iványi, who recommended a microcement based procedure implemented by TechnoConsult 2000 Inc.

To determine the proper amount of injected material to be used and to determine the extent which this structure could be subjected to injection, a 10 m2 test area was injected. After detailed analysis it was decided that injection should take place after the shotcrete was applied to both the internal and external surface. About 3000 injection receptors were placed in a50x50 cm raster in each chimney prior to the application of the outer shotcrete layer. The test injection was considered a success and the injected material occupied about 4% of the volume of the cross section injected. First the receptors were filled with water. The injected material was a compound consisting of several components, the mixing was automated and it was transported and injected with a suitable device. The characteristic penetrating ability of the microcement was noticed when injected material appeared several meters from the location of the injection.

To verify the effectiveness of the injections, 90 mm diameter cylindrical cores were drilled out and subjected to material testing. These tests were suitable to gauge the uniformity of the material strength of the repaired structure.

8.2 Selection of surface protection

The covering of the outside surface of the chimney with a protective layer of paint was undertaken for the following reasons:

- the appearance of hairline cracks in the relatively thin (5 cm) shotcrete layer was anticipated. These would be due to shrinkage, weathering and the unavoidable openings left in the surface during construction. These cracks can result in the corrosion of steel reinforcement,
- aviation regulations require that tall chimneys be painted with coloured stripes,
- to create and aesthetically pleasing appearance.

For surface protection the Keston Flex brand material was selected for its ability to bridge cracks, its moisture permeability and light-reflective properties.

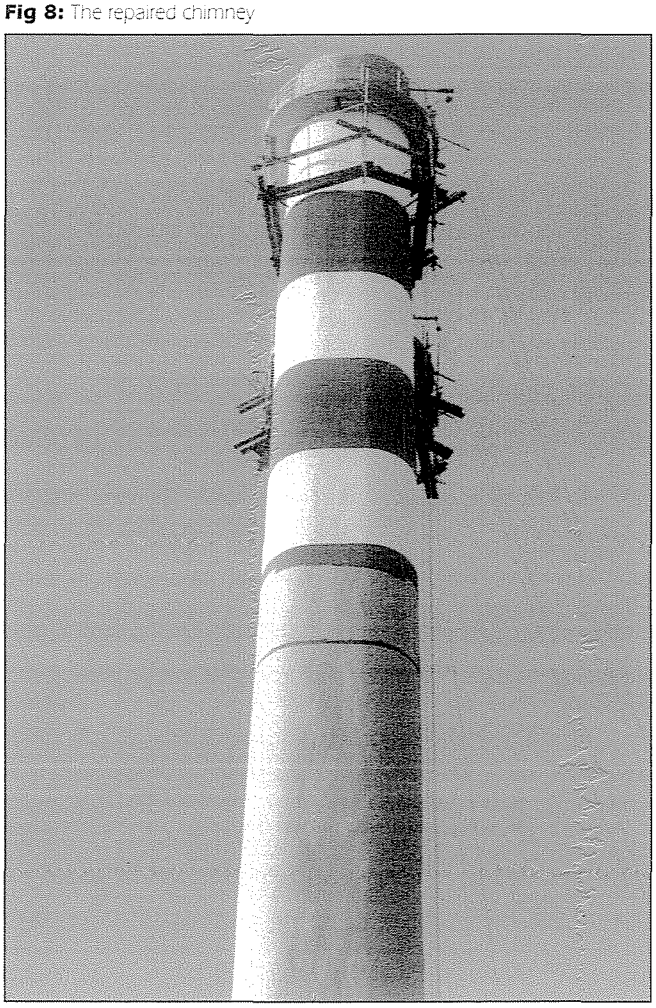
The material was applied in 2 layers. Prior to that the outside surface of the chimney was cleaned with high pressure sand blasting to clear away debris stuck to the surface of the structure at the time the shotcrete layer was created.

The thickness of the surface protection layer is about 1 mm, the adhesion to the surface exceeded a value of 1 N/mm2. The surface of the chimneys as it appeared after repairs are shown in *Fig 8*.

The shotcrete 'jackets' improved the ability of the structure to resist seismic loads. Keeping the weaker parts of the wall compressed results in an increase of the wall's ability to undergo ductile deformation and the overall ductility was increased.

The vertical and horizontal reinforcements were placed to prevent the appearance of diagonal cracks in both directions during a seismic event. The compressed walls increase the protection against the bending and buckling of the reinforcement.

Fig 8: The repaired chimney



9. EVALUATION OF THE CONDUCTED QUALITY ASSURANCE TESTS

The quality assurance tests prescribed in the quality control document and conducted during the repair and strengthening of the chimney yielded satisfactory results.

The regular testing of the materials and the accumulation, recording and evaluation of several thousand data elements proved the excellent quality of the materials used and that good construction practices were followed.

The timely and regular inspection of the construction technology and the placement of the material verified that the specifications regarding the thickness of the shotcrete layer and for the fastening of the reinforcement mesh were followed.

The presented data illustrate the effectiveness of the measures taken to improve the stability of the structure and the ability to resist seismic forces. This was achieved by creating a more homogeneous and uniform material of the structure.

The table permits the comparison of the strength values along the height of the structure, obtained from core samples prior to the repair and after the repair was completed.

It must be noted that prior to the repairs core samples could only be extracted from areas off sufficient material cohesion, the areas to be tested were directed away form areas where the loose material would not permit core sampling. After the repairs were completed, the core samples were obtained from randomly selected areas of the structure. The data derived from test cylinders taken from the selected areas presents the condition of the structure in a more favorable light, whereas the test data obtained from randomly selected areas is more reliable.

The results have shown that when the repairs were completed there was a significant increase in the strength of the concrete. The distribution of the strength of the concrete along the height becomes more uniform and the design strength was achieved and distributed evenly throughout the structure. The improvements in the concrete and the attainment of a homogeneous material state was due to the injection. The core samples have provided proof that the injected material penetrated the loose concrete and filled out voids, generally verifying the effectiveness of the injection.

10. CONCLUSIONS

The condition of the four 100 m height ventilation chimneys of the Paks Nuclear Power Station deteriorated to an extent that repairs and strengthening could no longer be postponed. The chimneys were built applying a slip-forming method. During construction there were deficiencies in the technology employed, the concrete was not properly compacted and vibrated, there were tears due to the movement of the slip form assembly and the concrete quality was not uniform. The passing of the time and the effects of the weather also contributed to the deterioration of the structure.

A structural analysis has revealed that the strength of the structure is adequate for resisting wind loads but does not have and adequate safety of margin to resist seismic loads. The analysis has also shown that the most cost effective solution was to strengthen the structure using a combination of shotcrete application and injection with a microcement based material. The repair, or rather, strengthening was achieved by applying shotcrete to both the inside and outside surface of the chimneys after a welded wire mesh reinforcement was placed in position, forming a protective 'jacket' that is also connected to the reinforcement.

To improve the material of the concrete walls and to increase the strength of the concrete a micro-cement based material was injected.

The general rule that during construction or repairs unforeseen and frequently unfavorable conditions are encountered also held true for the repair of these chimneys. This necessitated the development of a strengthening technology that is flexible and can be used under varying circumstances.

The use of compatible cement-based materials and the employment of construction technologies that were suitable to the structural conditions present achieved excellent results. The use of a flexible construction technology and a rationally organised and precisely executed quality control process, taking into consideration the great significance of the structure contributed to the success of the strengthening.

As the result of the strengthening the usability of the chimneys and the operation of the power plant is assured for decades to come.

11. ACKNOWLEDGEMENTS

The authors would like to acknowledge the contributions of all who helped to resolve problems that surfaced, provided technical support and assistance, and have demonstrated their loyalty and readiness to co-operate in this venture:

They are listed here:

The Construction Department of the Paks Power Station and the chief resident inspector,

CAEC Inc., who completed the structural analysis, developed the strengthening technology and was responsible for quality control,

VERTICOR Inc., who was the primary contractor,

Techno Consult Inc., subcontractor,

Techno Wato Inc., suppliers of the materials.

REFERENCES

Kovács, K. (1999), "History of repair of concrete and reinforced concrete structures", *Vabetonépités*. 1999/2., pp. 50-53. (in Hungarian)

Orosz, Á. –Csató, Gy.- Tamáska, J. (2000), "Strengthening of a 2000 wagon capacity reinforced concrete grain silo located at Marcali in Hungary." *Concrete Structures (2000) Vol.1. pp. 44-51.*

Orosz, Á. (1999), "Strengthening of reinforced concrete structures with fibre reinforced shotcrete. Fibre reinforced concrete – from research to practice", (ed. Balázs, L. G.) pp. 259-267 (in Hungarian)

"Rehabilitation of Concrete Structures" *RILEM Proceedings* (1992) Melbourne

Documents of quality control and repair. Almási J. – Orosz Á. (in Hungarian)

Structural analysis of ventilation chimneys of Paks Power Plant, 1997. January, CAEC Ltd., (in Hungarian).

Quality plan for reconstruction of 100 m high ventilation chimneys of Paks Power Plant, April1997, VERTIKOR Inc. (in Hungarian).

Quality control document of the reconstruction of the ventilation chimneys of Paks Power Plant I., October 1997, CAEC Ltd. (in Hungarian).

Quality control document of the reconstruction ventilation chimneys of Paks Power Plant II., December 1997, CAEC Ltd. (in Hungarian)

Opinions regarding questions arising in analysis of ventilation chimneys of Paks Power Plant made by Mélyépterv Civil Engineering Ltd., 1998. January, CAEC Ltd. (in Hungarian)

Structural analysis of ventilation chimney no. III. at Paks Power Plant for earthquake loading, April 1998, CAEC Ltd. (in Hungarian)

Analysis and results of experimental injection at ventilation chimney no. III. at Paks Power Plant, May 1998, CAEC Ltd. (in Hungarian)

Quality control document of reconstruction of the ventilation chimneys of Paks Power Plant III., December 1998, CAEC Ltd. (in Hungarian)

Quality control document of reconstruction of ventilation chimneys of Paks Power Plant IV., June 1999, CAEC Ltd. (in Hungarian)

Quality control document of reconstruction of ventilation chimneys of Paks Power Plant V., 1999. June, CAEC Ltd. (in Hungarian)

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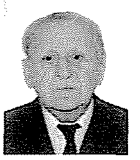
COMPATIBILITY AND INHOMOGENEITY AT THE JOINTS OF PREFABRICATED ELEMENTS



Dr. Jenő Gilyén

There is basic incompatibility between high strength prefabricated elements and the cast in-situ poured joint-filling concrete. The latter is never properly compacted; full of voids and its elasticity modulus differs too much from the same parameter of the precast element. The concrete used for the jointing starts shrinkage only at the moment of the compilation of the complete structure, and the measure of this shrinkage is rather big. Consequently, definite cracks appear around the contours of the elements. These cracks and the great differences between the elasticity modulus heavily influence the structural behaviour and the statical model. These issues, being the differences between the physical parameters of the different concrete types involved are consequences of the very differing conditions at production phase including the significant variances in the dimensions. The resulting phenomena cause effects modifying even the structural model - mostly with structures erected using high strength precast elements.

The results gained by experimental modelling of structures may be verified only in the cases where the experiment has properly repeated also the technological conditions of the construction.

The joints with their small dimensions can be filled only by pourable concrete, and its lower strength and rigidity should be properly modelled.

Keywords: concrete, water-cement ratio, prefabrication, joints, compatibility, inhomogeneity, structures

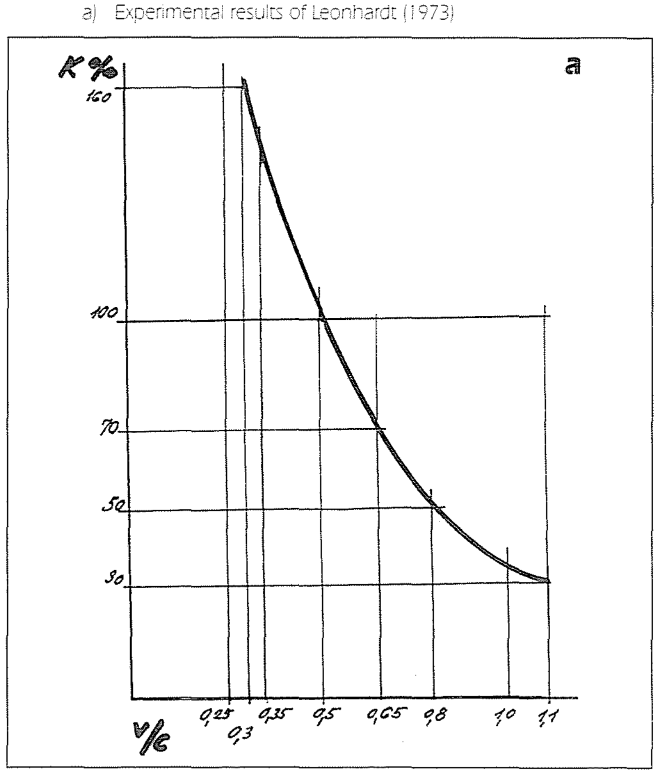
1. PHYSICAL AND MECHANICAL PROPERTIES OF THE CONCRETE WITH REFERENCE TO THE TOPICS OF THE ARTICLE

It is known that solid stone-like materials behave elastically (or quasi elastically) under compression but at their strength limit they break explosively. Although concrete is a manmade heterogeneous mixture, high strength modern concrete behaves similarly to the natural stone – showing considerably fewer characteristics of inhomogeneity. On the other hand, weaker types of concrete (having a higher voids-ratio) have significant non-linear portion in their compression curve after the linear portion and before break. The cement-clinker binding components (and the boundary surfaces) normally have lower strength than the particles of the aggregate. Under compression, these elements of the concrete break first and furthermore when sufficient void space is "at hand", this debris fills the voids. Since there is great friction at the internal cracks (between the particles) at the end of the linear (harmless) loading section nothing extra happens but the greater deformations – say the plastic (-like) deformation zone starts.

It is clear that the so-called ***plasticity*** of the concrete is nothing but the start of a break. This specific zone in the load-deformation function has a significant *effect* ***only with weaker, non-properly compacted concrete***. Please remember that all concept with regard to plastic behaviour (and its involvement in the static calculations) originated at a time when generally C8-C10 type concrete was available for (mainly cast-in-situ monolithic) reinforced concrete structures.

Author had the opportunity to visit the lessons of Kazinczy in the 1940's. The lecturer was one of the pioneers who introduced the effects of plasticity in static calculations. He underlined that the plastic zone is followed by a harder zone at the steel, but in the concrete this "plastic zone" is immediately followed by the collapse of the material. Based on this concept he proposed to use the plasticity of the concrete (as a possible increment in load-bearing capacity) with the structural calculations only with load-bearing structures which are highly statically undetermined and so capable to redistribute the loads during some moderately extended deformation. At that time, great monolithic frames and slabs were common with multiple static undetermined structures. He also warned that in case of repeated loading and unloading special investigation is required.

Fig. 1: The influences of the water-cement ratio ("w/c") on the strength of concrete
a) Experimental results of Leonhardt (1973)



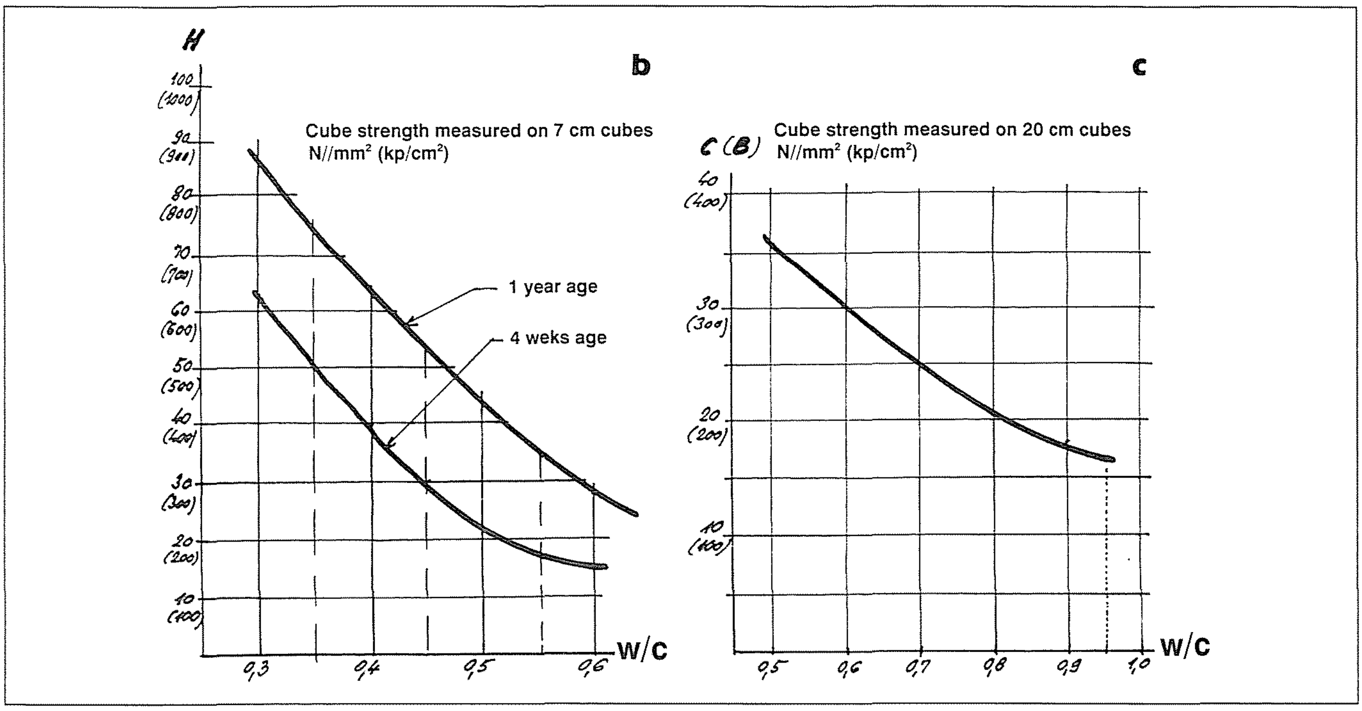


Fig. 1: The influences of the water-cement ratio ("w/c") on the strength of concrete
b) Results on test cubes (7×7×7 cm) made with portland cement - Mihailich-Schwertner-Gyengő (1946). Lower curve: 4 weeks age Upper curve: 1 year age
c) Results of test-cubes (20×20×20 cm) made with portland cement - Mihailich-Schwertner-Gyengő (1946)

Common medium-strength concrete behaves in a fully linear-elastic way up to 30-40% of its ultimate strength. When stresses increase above this level a continuing deformation occurs, so that up to about 70% of the ultimate strength the actual deformation is non-elastic – or say contains significant remnant component. At the end of this section of behaviour the degradation of the concrete structure commences. During the degradation process the previously existing incontinuous shrinkage-cracks start to grow and meet each other (Balázs, 1994)

Before the beginning of the fatal zone of degradation, the deformation of the concrete (in joints) is about 1/1000. At this point its 1 to 0.16 Poisson number does not allow the closure of the shrinkage-cracks ranking 0.6 to 1 mm in linear meter perpendicular to the axis of load. As a consequence, the pourable-type filling concrete in joints will not be "horizontally" supported all around (by the "stronger precast elements") while already "vertically" it is catastrophically overloaded.

On the other hand, more recently used common medium-strength reinforcement steel bars have 1.5/1000 deformations during average load conditions. Around these, the concrete cover starts developing definite cracks over 0.3/1000 deformations. So undisturbed contact-co-operation does not exist between them and this condition promotes seriously the atmospheric corrosion of the steel.

The reinforced concrete structures constructed until the 1940's in the past century still supported the belief that a long life-span could be achieved without extra measures against corrosion. When searching for reasons for the spontaneous durability we may state that those types of concrete had much less shrinkage because of lower cement portion in mixture and accurate compacting even with low water-cement ratios. Simultaneously the steel bars had much less tension (lower ultimate strength) and so their deformations were much less than the steel bars of recent structures. Therefore, the existing deformations of the steel reinforcement and the body of the concrete were compatible. In many cases, the accurate stress calculation may prove that the RC remained in stress referring to the "first" status, and so the stresses and load in the steel bars remained still more moderate. The actual load was normally much less than the ultimate load defined in the Standard Calculation. An additional protection against corrosion occurred with properly maintained lime-mortar plastering over the concrete and reinforced concrete structures.

Later the industry of the construction made efforts to avoid plastering and this movement ran parallel with the introduction of precast reinforced concrete elements. These factory-made elements had fine strength, attractive smooth surfaces and proper compaction rate and consequently allowed for higher allowed stresses to be applied. With this new range of allowable stresses (mostly used with modern high-stress steel bars) a set of cracks definitely occurs – producing the free entrance of corrosive effects into the structure.

There was a dangerous period in prefabrication when convenient and safe compaction was ensured by the incrementation of the water-cement ratio. Later on specific organic adhesives solved this problem. Famous researchers investigated the catastrophic reduction of the strength of the concrete mix caused by the higher water-cement ratio. *Fig. 1.a* shows the experimental results of Leonhardt (1973), *Fig. 1.b and 1.c* show results from Mihailich, Schwertner and Gyengő (1946). In the latter the post-strengthening of the test cubes was measured as well. Although there is a certain post-strengthening at a higher W/C ratio it may not compensate for the general loss in strength. (Most probably, the longer dry-out period allows some additional, more accurate hydration of the not so finely ground portland cement particles.)

When pouring the joint concrete between the prefabricated elements it is common practice unfortunately to apply some additional water. This is said to avoid water admission at the dry precast surfaces and to promote the entry of fresh concrete into complicated small profile holes, where proper compaction is hardly or not possible. These facts of practice should be taken into consideration during the time of structural design. The usual execution of the technology influences the obtainable results of material strength sometimes even more effectively than the mixture design. These conclusions are proved also by (Balázs-Tóth, 1998).

Using proper number of stirrups can effectively increase the ultimate safety of compressed zones in reinforced concrete. It is important to apply secondary (longitudinal) bars to moderate stirrups when the dimensions require. In several cases, the dimensions of a joint will not allow the application of reinforcement due to corresponding regulations. In these cases the body of the joint must be considered not as reinforced concrete but as plain concrete.

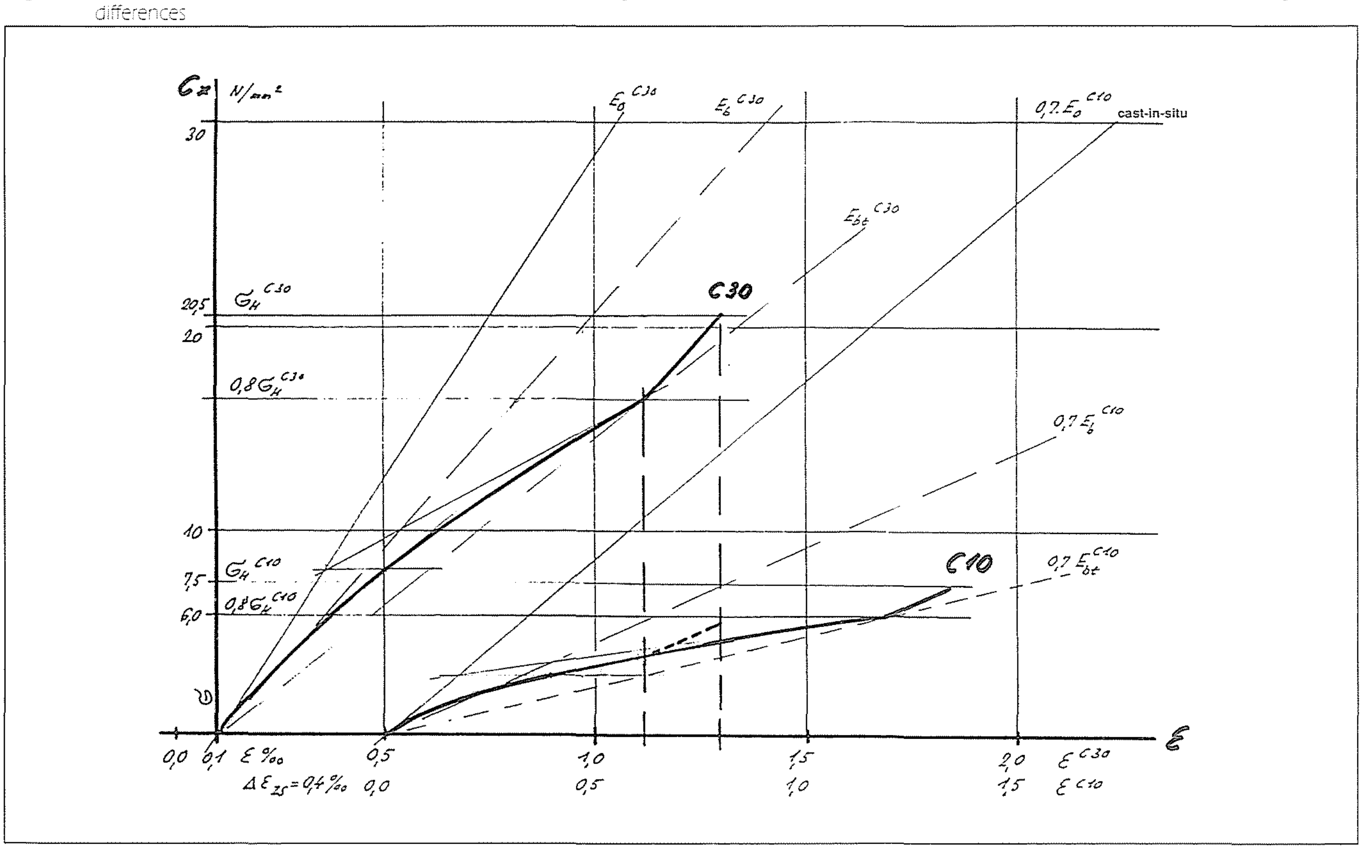
The overdosing of water and cement both increase shrinkage. The author has witnessed in many cases 1mm/1m shrinkage, so the 0.6 mm/m Eurocode value may be taken as the proposed minimal value of shrinkage. Plasticity additives reduce this problem but do not fully solve it since the proper compaction of the joint concrete is still a remaining issue.

2. THE INFLUENCE ON THE STRUCTURAL MODEL CAUSED BY THE METHODS OF COMPILATION OF STRUCTURES CONSISTING OF PRECAST ELEMENTS

The precast, prestressed-precast elements of the engineering structures have usually joints between them filled with in-site pored concrete. This method is inexpensive and insensitive to inaccuracies in the size and positioning of precast elements. Normally the positioning has a standard allowed inaccuracy of in the cm-range. It might be reduced only with expensive and time-consuming operations. Therefore, it would be useless to force a smaller range of standard variation in size which would cost also more. On the other hand, its aim would be to reduce the dimensions of the joint. The joint concrete must be pourable; its maximum aggregate size must be between 8 and 12 mm. Its ultimate strength is not ideal and has a tendency for large amounts of shrinkage and creep. Furthermore, full post-pouring deformation causes cracks between the precast element and the body of joint. When any load transfer is required, first deformations and dislocations of the precast components occur while the joint-concrete remains unloaded – unless these deformations and dislocations cause closing of cracks around the joint-concrete.

Taking into consideration the comments above, the deformation curve is shown in *Fig. 2.* The two different types of concrete (reinforced concrete element and joint) have different stress-deformation curves. To this fact should be added the impact that the joint-body "enters into load bearing" only later, when the cracks became closed. Based on these facts the distribution of the load is significantly different from the expected case neglecting the preliminary process of crack closing. In some cases (e.g. at the joint of high strength-concrete piers with bedding mortar/concrete in-between) the rather rigid high strength concrete may be compressed nearly until its ultimate capacity under the areas where they touch directly each other. At the same time, at the larger part of contacting surface - where the fitting mortar interacts - stresses remain rather low. The purely theoretical approach may be misleading as the real data occurring in great numbers at the sites show much greater inaccuracy in dimensions, Young-modulus and ultimate stress, than it is expected based on data of concrete samples made accurately under convenient conditions in testing laboratories. Some detailed numerical explanations and analyses may be read in Refs. Mihailich-Schwertner-Gyengő (1946) and Balázs (1998). As the final consequence, it must be said that the simple addition of the ultimate load-bearing capacities – SUM ($A_i \times r_i$) - is totally misleading and dangerous!

Fig. 2: Stress-deformation curves of prefabricated elements (C30) and joint-concrete (C10) including the influence of creep, shown with the shrinkage-differences



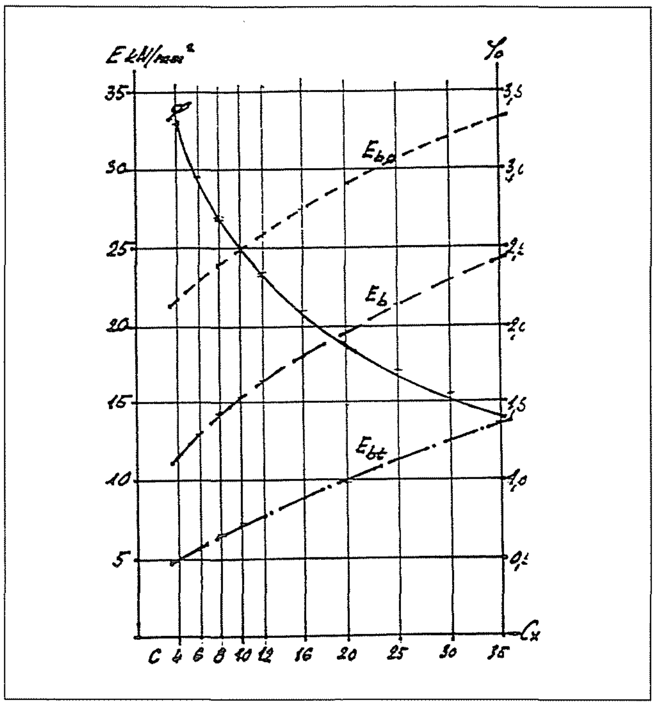


Fig. 3: Young's-modulus (E - lowest curve: permanent load; middle curve: normal case; Upper curve: immediate load) and creep-coefficient (%) for different types of concrete (Strength: from C4 to C36)

Fig. 3 shows the Young's-modulus and creep parameters at different qualities of concrete based on Eurocode EC2. While the ultimate stress becomes ten-times greater through the steps of quality-classes, the Young-modulus becomes three-times greater - at smaller loads. When increasing the loads/stresses the effect of creep arises. In the cases when the quality of the concrete is lower, both the smaller Young-modulus and (even with relatively smaller stresses) the more effective creep should be considered. If the concrete had too much surplus water at the time of pouring then the volume occupied formerly by water turns into a void as the concrete dries out. This reduction of the effective solids in the cross-section will increase the said effects making the lower quality concrete "soft".

The above discussed "non proportional" distribution of the load in the joints (with all of its consequence) can be detected at all composite load-learing structures. It is more dangerous then the rigidity of the total structure is rather high, and it is more sensitive for the modified deformations, as the structure does not have significant capacity for the redistribution/modification of joint-loads through secondary effects. At "soft" classic reinforced concrete frames these problems of joint-overloading do not occur typically. At a reinforced concrete wall, in the zone above an opening the cracks starting from the corner of the door may be so great that the elastic deformation generated by the shears of the usual horizontal loads will not close them at all.

The presence of low-quality concrete may occur not only at he discussed cases when secondary concrete-pouring takes place between precast elements but also with complicated parts of monolithic structures where the reinforcement bars are too close to each-other and this condition hinders proper compaction. At these zones (just at the cross-sections where the loads are maximum) locally increment of voids inaccurate compaction causes defects in physical properties of the concrete. Typically the sections of beams, which are close to the pier with their upper reinforcement against negative bending moments, may be weakened by these problems. Consequently, the positive moment will increase in the middle of the field of the

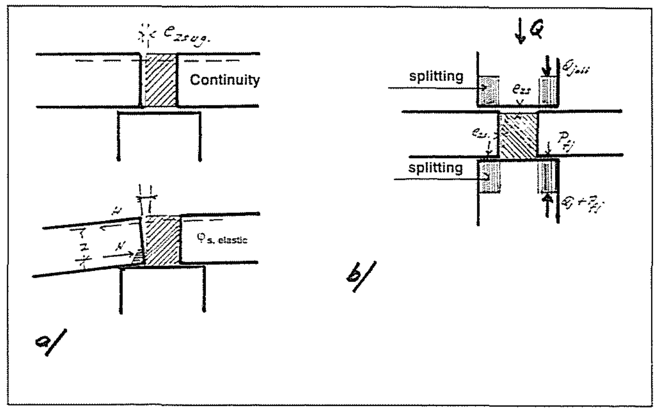


Fig. 4: Illustration: how the "monolithic behaviour" is destroyed in shrinkage and the low Young-modulus of joint-concrete.
Result: Inhomogeneity!
a) Concentrated rotation occurs reducing the bending moment at the support
b) Inhomogeneous flow of loads in the vertical: Joint-concrete in the centre remains loose, unloaded.
(Edges of slabs: overloaded!)

beams and the load-bearing capacity (reserve on elastic load-redistribution) of the whole structure will significantly decrease. *Fig. 4* shows example explaining the discussed effects.

The concept, experiences and the elaborated explanation referring to the above-discussed specific effects are more detailed in Ref. Gilyén (1982). It is important to underline that the structural behaviour differs quite a lot from the widely expected linear elastic theory. As the bending moments increase, the load-bearing concrete cross-sections become cracked. This local change in rigidity parameters destroys the linear elastic character of the structure forever. The deformations/deflections suddenly and locally increase; the structure loses it linear response. This effect is also very characteristic at the sheared sections as mentioned above.

At the supports the bending moment decreases since there is a concentrated rotation as the crack develops. This resulting bending moment can be calculated therefore on the difference between the equivalent (moment-making) rotation at the structure without cracking and the rotation caused by cracking.

$$M_{crack} = K \times tg\varphi_{crack} \times I/L$$

where at a beam with uniform load: $K = 2 \times E$

The formula shows that the greater the inertia (I) the greater the effect from the cracking. That means that a deeper beam or smaller span increases the development of cracks – and all its structural consequences.

If the load is characterised by shearing effects then the deformation of the beam is inflexion-like (see *Fig. 5*). In this case, the cracking next to the supports causes fast degradation in bending moments and so we may calculate only taking into consideration the cracks at the corners between the beam and wall/pier. So using basic mechanical theory the moment:

$$M_{support} = T \times L/2$$

and the deflection (vertical dislocation) for the half-length of the beam:

$$d_M = T \times L^3/(24 \times E \times I).$$

The other part of deflection is:

$$d_{crack} = tg\varphi_{crack} \times L/2.$$

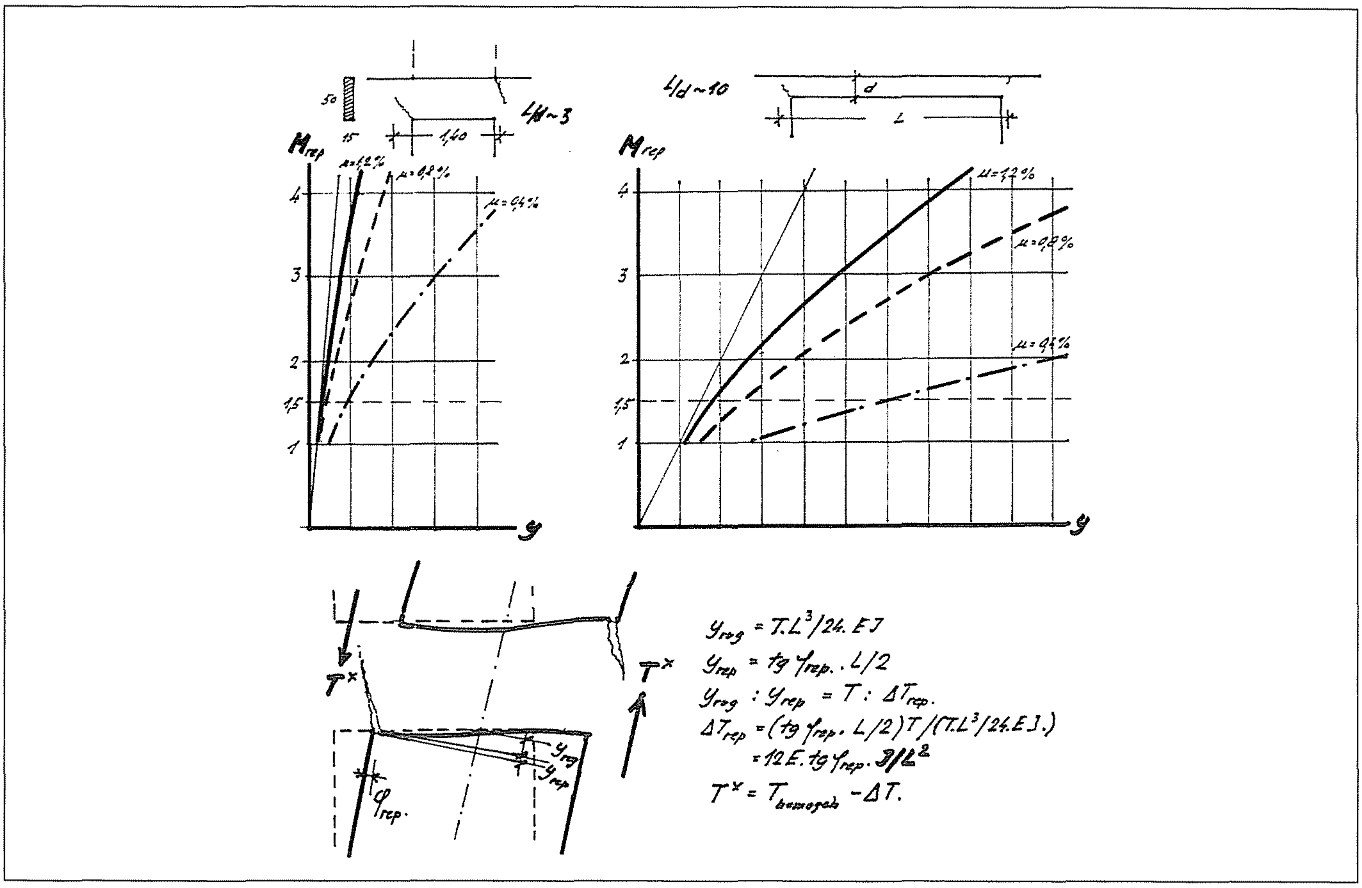


Fig. 5: Deformations in composite walls in zones above openings - Including the impact of cracks and reduced Young's-modulus at higher stresses. Curves show the cases with different reinforcement ratio and inertia.

The ratio of those two components will determine the moment at the support. In a given case of real primary deformations (caused by settlement of fundaments, inclination of wall-zones, etc.) the decrement of the ultimate response (limit shearing force) of the structure is described by the ratio as follows:

$$T = 12 \times E \times tg\varphi_{crack} \times I/L^2 .$$

That means: the effects caused by the crack increase as the inertia of the beams (connecting the wall-pier zones) grows or as the second power of the span decreases. Because of these facts, it is a great failure to neglect the presence of cracks in the structural analysis of a wall having rows of windows or doors! If a homogeneous, non-cracked wall-disc-model is used then the role of "connector-beams" in horizontal load bearing (through bending moments) is seriously over-estimated! This failure in load-bearing brings a secondary failure: the false information about the loads born by the wall-zones. Actually, they will be greater than the homogeneous model suggests. Although remembering the physical properties of the recently used steel and concrete materials it is clear that their calculated elastic deformations in a normally loaded cross-section are seriously incompatible, i.e. *the cracks in the concrete definitely appear!*

Recently in Hungary 20% of the housing volume consists of houses constructed with large-panelling technology. The majority of this was built between 1966 and 1986. This mass of flats was constructed using strongly uniformed architectural standards, referring to the political dictate of the age. A great number of these flats have come to an age for renewal of the installations while their aesthetic appearance also requires upgrading. Unfortunately, the big construction companies which designed and built them disappeared in the changes of the 90's economic modernisation. So the specific knowledge of this technology has been dissipated although some books [for instance Refs. Levicki (1965), Gilyén (1982), Gilyén (1998)] deal accurately with the structural extremities existing in these domiciles. The structures with large wall panels use basically plain prefabricated reinforced concrete elements bound into final position in the house to form the box-like cells of rooms. The factory-made wall panels were accurately cast with controlled dimensions and high quality concrete. The reinforcement bars were running out of the edges (rims) to accommodate final binding for load-bearing joints in the completed appearance. In many cases, the connecting bars had loop-like endings. The connecting process of the elements applied sets of U shaped bent reinforcement bars contact welded into final position. These connections in the reinforcement system allowed for tension-bearing junctions between the strong prefabricated wall components. These wall elements were designed to be rather strong mostly to account for the additional technological requirements such as the early striking of formwork and transportation. The compressive and shearing loads in the composite walls consisting of panels above and beside each other were transferred from one panel to another through the joints. The "vertical joints" (between the sides of panels) are chimney-like holes with a 15×15 cm cross-section filled by soft concrete in one step for their full depth of 2.6 to 2.8 meters. This was performed after the panels were finally positioned. The edge-pattern of the panels are designed "saw teeth-like". Although the edge profile fits to the requirements, the soft joint-concrete with its greater shrinkage creates incompatibility with the high strength concrete panel. This incompatibility is impossible to avoid because of the construction technology. The resulting structural inhomogeneity occurs also at the "horizontal joints" (i.e. at the joint between the panels above each-other). The proper design of the profiles (taking into consideration the construction technology) may widely reduce these problems but anyhow there is never uniform load-sharing between the different age and quality concrete components acting together in and around a joint. Failures in structural analysis may occur as discussed in the article on the IABSE Colloquium Berlin 1998 (Gilyén, 1998) – (Progressive collapse of a tall building caused by blasting while uniform load-bearing was expected in inhomogeneous horizontal joints).

Due to its box-like "3D" character a panelling house normally has a considerable reserve in structural strength to cater for extreme loads. This "excess of load-bearing capacity" can be considered as additional insurance against catastrophic events. This type of domicile normally has a great number of flats, so it is very reasonable to incorporate better safeguards against progressive collapse than with smaller buildings. It must be taken into consideration that the mentioned "excess safety" is not "at hand" automatically for repeated loading cases, which require the lack of remnant deformations. The limitation of crack-size is unavoidable for the variation of service loads/usual horizontal loads. The realistic calculation of the crack-size must return to the inhomogeneity and in consequence the problems of compatibility. The inhomogeneous structural zones at the joints define the weakest parts of the building structure. The unrealistic and optimistic homogeneous wall-model gives significantly less stresses at these critical zones. Using this "easy to calculate" model just results in unexpectedly larger cracks which would seriously shorten the safe life-span of the structure. The importance of the problem has been realised in France where a large number of panelling house have also been built. Since it was impossible to use a purely theoretical approach, the CEBTP research institute in Paris executed more than 120 tests, loading the modelled joints to break point. Measuring the correlation between the shearing forces and the deformations/crack-sizes it has been proved that there is no linear behaviour, although the amount of connecting reinforcement, the strength category of the concrete and the shaping of boundary surfaces were taken into consideration! The results with the experimental formulae developed have been reported by Simurda (1984).

The weak point of the experimental series was that the joints were cast horizontally which is contradictory to the actual case where the narrow joint was filled in the vertical position. Consequently the manipulation/compaction conditions were unrealistically easy in the laboratory pavement. Nevertheless, the experiment demonstrated the tendencies but still did not comprehensively illustrate the problems of hard on-site conditions.

The author first encountered these technological issues in 1951 during the construction of the Budapest Sport Stadium. This structure incorporated saw-teeth shaped internal surfaces which were later joined by pouring concrete into cavities with 12 × 13 cm cross-section. Although the test cubes of the joint concrete showed excellent strength, in the real structure it transpired that there was much less capacity associated with the connecting/shearing forces at the joints. The filling concrete formed nothing but a separately filled body contoured all around by shrinkage cracks so that before significant dislocations there was no mechanical (load-bearing) contact between the elements to join. These early experiences led the author to introduce his own way of dealing with joints cast with concrete in a later phase.

Designers often do not have enough additional field experience with regard to recent construction technology and actual site circumstances. To reduce this gap in knowledge and related theoretical disputes, the Hungarian Institution of Construction Science (ÉTI) executed a series of 1 to 1 scale model experiments with wall-panels in the period 1980-81. These were conducted at the Laboratories in Szentendre. Although there were strict financial limitations, which influenced the planned dimensions of the tests, the results proved the characteristic inhomogeneous behaviour of panel-structures. See the evaluation article from Kaliszky, Györgyi and Lovas (1983). A committee led by the author had already elaborated, in 1970-72, the Hungarian Standardised Technical Prescriptions regulating the calculation and design of inhomogeneous structural behaviour in document ME 95-72/74 (Technical specification, 1976). The statements and the approaches introduced and laid down in the mentioned Standardised Regulation were eventually proved by the laboratory test-series - 10 years later!

To go into the details, it has been demonstrated that a wall composed of panels must not be considered as a homogeneous vertical "beam" or disc with a simply calculated bending moment generated by the horizontal loads. The wall works rather as a Vierendeel-type "beam". The "binders" formed by the so-called slab-zones (above the openings) and their neighbouring zones regulate the horizontal dislocations making them quasi homogeneous for all parts of the wall. This effect causes such great forces that they cause concentrated discontinuity at the joints. Neither the pier-like wall-panels above each other nor the connections between them (=beams above openings) worked as parts of a homogeneous monolithic structure.

During the time of the above mentioned French experiments, similar tests were also conducted in England. They applied a different system to ensure the co-operation of the neighbouring elements. Binding reinforcement bars were positioned mostly at the height-zones of slabs above openings instead of the saw-teeth like contours of the elements (which cause difficulties also in technology). The English results/tendencies were similar to French ones but in their case the concentrated dislocations experienced at the same range of shearing force became two or three times greater than with the French shaping concept.

As, later, the joint concrete is poured - it starts developing its own shrinkage. In case of load/deformation of the composite structure, the full length of neighbouring element-contours does not start acting as load bearing - since at many sections this shrinkage gap "contours" the prefabricated element. The limit shear force is only a fraction of the that expected. As the shear increases some sections of the loaded (actually load-bearing) concrete gets destroyed (*rigidly!*) while at other zones cracks only start closing and so a new zone enters into load bearing. (Note that a condition never develops wherein full-size "acting-together" occurs at the boundary-surface of the joint!). Meanwhile dislocations significantly increase and as load (and deformation) is increased, finally almost all of the direct concrete-to concrete shearing capacity becomes damaged. By this end-phase, nothing but the connecting bars transfer the shears anchoring the concrete surfaces to each other. It should be underlined that the shearing capacity of the bars can never be added to the calculated load-bearing capacity of the normal-state concrete, even in extraordinary load cases! Although the tests mentioned and the related discussions were all dealing with structures of panelled houses, the illustrated phenomena is the same at any other composite structure – anywhere with co-working elements having different mechanical parameters. Therefore, these other similar structures can definitely be described as having differences in ultimate stress, ultimate elastic deformation and cracks at connections. The structural analysis proved also the key-importance of the appertaining technological conditions.

3. CONSEQUENCES AND RECOMMENDATIONS

The advantages of the use of precast reinforced concrete elements is widely known and accepted. We should not forget that with these particular structures we definitely loose the ability to use material continuity (homogeneity)! With the development and construction of the joints, new effects appear, new risks appear and the possible impact of workers' failures significantly increases.

Under construction these monolithic reinforced concrete structures also experience some "work-gaps", where pouring of concrete stops for a while (cold joints). At these cold joints the reinforcement runs through without being cut and since the time interval before restarting the pouring is much shorter than with the prefabrication, the difference in the shrinkage cracks are much less. Furthermore, neighbouring concrete is made of the same quality mixture and the compacting effectiveness may be expected to be very similar. So there are significant differences which really proves the necessity of discussing prefabricated composite structures separately, and where the site-works and the corresponding technology details have greater importance in the development of actual and final structural behaviour.

Great designers and researchers never failed to check the introduction of their results in industrial applications and utilisation. At the beginning of 20th century there was wide co-operation and communication between the universities and institutions of Europe. The results of experimental research were universally available. This aided the composition of common sets of rules and practical guidelines regarding RC-design. Abrams defined his aggregate-size distribution curves using the results of more than 40,000 tests. The famous Hungarian professors of that age: Zielinski, Czakó, Mihailich and Csonka were all leading engineers of important projects and they used the experience gathered to elaborate National Standards in construction.

Practical simplifications were based on actual experience covering the quality and production range of that age. They knew very well that if the methodological calculations involved were simpler, then the executed control calculations could be viewed with more confidence. For example, the introduction of the uniformed linear stress-deformation curve allowed for safe and fast design and the taking into consideration of the real range of the effects of shrinkage, creep and non-linearity in simple load-deformation tests at the upper zone. In the past three decades new possibilities have become common in the daily calculation practice. The "bilinear" (say plastic elastic) load-deformation curve, having a better approximation to reality, became popular. But this mathematical description is still too simplistic in several cases where the load-deformation curve of a composite structural element contains further breaks, etc. - and possibly other irregularities in the description-function of the material's behaviour. This irregularity in the functions is very hard to handle with a simple apparatus. Nevertheless, it is sure that such problematic zones of the curves exist at the appearance and closing of significant cracks/crack-zones. Cracks definitely appear, even promoted by periodical temperature-changes.

The design and calculation of load-bearing structures is never a simple theoretical issue belonging to the field of mathematics. The used/elaborated structural model is a realistic compromise between various possibilities of the safe and effective calculation methods and acceptable simplifications of the actual structural behaviour. It must not be forgotten that a structural model is at any time a transformation - having its limitations in applicability. There is a certain risk in application mostly based in neglecting some new effects which cause differences from past experiences, or even of known phenomena where their impact increases by some modifications in material or construction technology.

In the field of modern electronics there is wide-ranging design activity in the production of new devices/products. However, there is an excellent "tool" to control and moderate unexpected, negative effects which uses "feedback" circuits. We structural engineers unfortunately do not have such possibilities. If a joint fails we may not "develop a circuit" to strengthen the field-moment resistance capacity. The common statically undetermined structures have a certain load redistribution capacity, but serious local increments definitely occur in loads while other zones experience a reduction in their primary loads. These load-increments (sometimes in unexpected zones) act against the complex safety of the structure. The most often occurs when some plasticity is evident. The only replacement for "feedback" is the knowledge and experience of the designer.

The problems presented in this article are often very difficult to be correctly interpreted by mathematical methods - predominantly because of the serious irregular zones of the basic description-functions of the materials or composite sub-components (see *Fig. 5*). With "sensitive" structures where an extended impact exists (caused by smaller local deformations) the accuracy of the construction (joints) has extreme importance. This is the case with the composite wall-discs where rather "rigid" behaviour is expected but with the local "softening" at secondary finished joints the distribution of the loads and stresses will significantly modify. The real situation is still more complicated because of the impact from floor-slab zones, which ensure the uniformity of deformations of the walls once for every floor (see *Fig. 6.*).

The cited book (Levicki, 1965) has a chapter introducing the method from Prof. Rico Rosman (Zagreb) for the calculation of homogeneous wall-discs having a single row of openings. The method is based on the concept of energy-minimisation of deformations and is very complicated and time-consuming. Nevertheless, it is out of the confidence range just at the 2nd phase reinforced concrete conditions (when cracks start developing). It is still more inaccurate at walls composed of precast panel elements with joints finished at the building. However, this method showed correctly the tendencies and so helped the author to develop his practical approaches beside the significant inaccuracies contained therein.

A great number of panelled dwellings were built with between 10 and 16 storeys in Hungary. It was necessary to develop a practical method for the calculation of their load-bearing walls taking into consideration the previously explained effects. This method (using the available computing facilities of Hungary in the 1970's was based on a Quasi-finite-element-method) considered the composite walls as multi-storey frames with properly adjusted resistance and deformation parameters. Also taken into consideration was the regulating effect in horizontal dislocations which the co-working floor-slabs allow. The author must mention two excellent colleagues, Mr. György Prepeliczay and Mr. László Szabó who worked in 1971 on the numerical modelling which solved a series of walls specified with different patterns of doors or windows (Prepeliczay-Szabó, 1971). Practically the earlier mentioned Rosman-model is also based on the frame-approach but the contours being

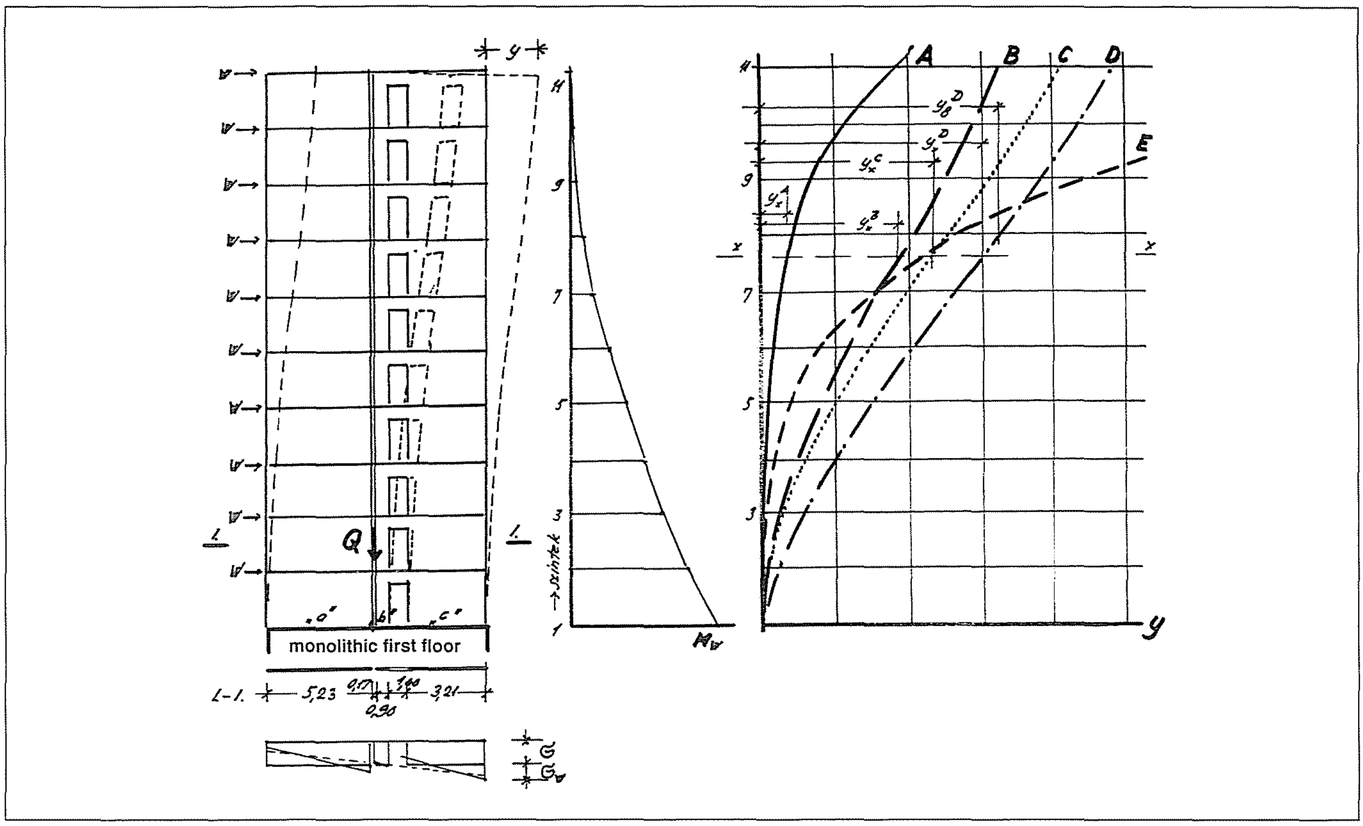


Fig. 6: Deformations of an 11-storey high wall-disc. Uniform horizontal load is given at each floor-slab level. Different cases and modelling:
A) Monolithic wall without openings (traditional "cantilever-structure")
B) Monolithic wall with one row of openings (doors). Zones above doors are expected to remain non-cracked.
C) Monolithic wall with one row of openings (doors). Zones above doors are expected to be cracked.
D) Inhomogeneous wall with one row of openings (doors). Zones above doors are expected to be cracked at corners. One row of ("softening") joint is involved.
E) Homogeneous (monolithic) wall-model including non moment-bearing zone above doors.

significantly different from real frames as the energy absorbed by the shearing-deformation is significant. At some zones, like the ones above the openings and the pier-zones on lower levels, even this part of the absorbed energy is dominant. Within the frame-approach, the zones above the openings were modelled as connected small discs to promote smooth and realistic mathematical description.

The calculation model based on the frame concept gave a convenient tool to deal with the compatibility of the deformations in the vertical joints using "replacement-beams". Furthermore, it was possible to take into consideration the cracked zones of the structure above the openings (reduced rigidity) by a moderated inertia-parameter. The results obtained were significantly different to the simple homogeneous model. The greatest differences occurred with the distribution of stresses at the middle-zone (see *Fig. 6*). So the impact of the cracked zones and the inhomogeneity (locally increased capacities in the deformation) are thereby demonstrated. This sophisticated model still has a serious simplification in allocation of zero-moment points (in beam-like parts above openings), allowing them either at the end or in the middle. The calculation capacities of the computer program (developed in 1971 by the colleagues mentioned) allowed a maximum of 1000 nods. Based on the results the designer had to decide to modify iteratively the replacement-inertia and the location of zero-moment points for the critical zones above openings.

This article intended to illustrate the fact that the use of a theoretical plastic deformation "reserve" together with the simplification of modelling which neglects the impact of inhomogeneity/local cracking zones, entails serious risk – that of indicating any given structure's resistance to be much higher that it really is. This risk increases when dealing with composite prefabricated structures while neglecting the impact given by the technology applied at the site. As it is required to ensure that economical structures are built, it is an on-going process and new efforts and theories to reduce the over-dimensioning of load-bearing structures plays its part. This concept permanently decreases the theoretical safety of the structures and the reduction in a structure's dimensions is expected to be balanced by the development of calculation methods. Nevertheless, if we forget about effects which can unexpectedly reduce the load-bearing capacity of a building, then safety is violated. In most cases this does not mean that the building will soon collapse, rather it causes "only" a dramatic shortening of expected life span. This initiates the concept wherein an increase in a structure's economy can have inverted effects – a 2 - 5 % saving in structural construction costs may reduce the life span by 10 to 40 %. This economic effect is even more serious when taking into consideration that 2-5 % of ("saved") structural cost is equal only to 1 - 2 % of the costs of the whole building – accordingly the defect of the future utilisation should be measured by using the whole value/cost of the building including all infrastructure value.

At the time of the introduction of the new (prefabrication) technologies, nobody could be aware of these particular impacts based on their specific characteristics. Nevertheless the leading scientists immediately started investigating these issues and, even 80 years after its first publication (Reinforces concrete structures - 1921), the words that Professor Mihailich wrote remain perfectly valid:

"The accurate observation of the behaviour in the experiments will save us from the failure to consider simply the results as mathematical constants."

4. CONSEQUENCES AND RECOMMENDATIONS

When contemplating a structure consisting of prefabricated elements the structural modelling must take into consideration that the joint concrete has unavoidable and significantly different physical parameters (i.e. Young's modulus, etc.) to the material used globally. This is based on the different conditions (dimensions and technology) representing the joint itself. Its ultimate strength and shearing resistance is much less while the shrinkage is great. The connection between the larger elements is definitely "loose" since there are cracks and gaps all around the joint and its material is "softer" than the normal concrete. Although the size of the gaps will practically not influence the structure's geometry, the load-bearing behaviour will be significantly modified by them. The more rigid the prefabricated element is (i.e. wall discs) the greater is impact (reduction) which occurs in structural behaviour. With composite walls of panelled dwellings, the structural modelling is dramatically different from the homogeneous model. *The ultimate load-bearing capacity will be determined mostly by the capacities of the joints.*

Furthermore, the horizontal and the vertical joints have different features. If beam-like substructures are connected to these walls other special measures should be taken with the modelling and calculation.

Recently, beams using high resistant steel reinforcement and modern concrete mix types have had serious incompatibility even at the simplest bending moment zone around the steel bars. The concrete becomes cracked in wide zones even in daily load cases. This reduces the inertia and increases the elastic deformations. The reaction of a common statically undetermined structures will cause a greater increment in moments at the supports than would be expected at first. This local overloading reduces the safety mostly in the at frame - in the lower floors - where the moments are even more significant and their influence on the overall structure is greatest at the columns.

The theory and the regulation of reinforced concrete design was for long time referenced mostly to classical monolithic frames and beams related to the middle of 20th century. It so happens that the issues of compatibility and inhomogeneity came to be researched under structural theory only later, supported by new concept experiments in the early 1980's in England, France, etc. The development was influenced also by the condition that the required tests were complicated and expensive.

It must emphasised that even the most splendid mathematical modelling will have serious inaccuracies if the input data (stress-deformation curves and ultimate strength, etc., inertia of the structure - sometimes varying in time!) have themselves more than 10% inaccuracy. The description of the construction material must include the specific technological site conditions (e.g. of compacting, impact of water/cement ratio, sizes and dimensions, etc.).

With respect to the so-called plasticity of the concrete used for joints it is advised to use only for small size local effects like edge-stress irregularities or unexpected deviation in physical parameters.

It is dangerous to consider the structures monolithic without specific evaluation - mostly with regard to the recent low-percentage reinforcement cases. In many cases it may be necessary to modify the calculation model or the basic data - based on the first calculation outputs. The author has used examples and experiences collected during his nearly 60 years in the field of structural engineering.

5. REFERENCES

Balázs, Gy (1994), "Concrete and reinforced concrete", Vol.I., *Akadémai Kiadó*, Budapest

Leonhardt, F. (1973), „Vorlesungen über Massivbau" Bd.I. *Springer Verlag*, Berlin

Mihailich, Gy.- Schwertner, A. – Gyengő, T. (1946), „Concrete structures", *Németh József Mqszaki Kiadó*, Budapest

Balázs, Gy.- Tóth, E. (1998), "Diagnostic of concrete structures" Vol.II, *Mqegyetemi Kiadó*, Budapest

Gilyén, J. (1982), "Influence of non-linear deformations on statically undefined structures", Budapest, Journal *Magyar Építőipar*, 1982/3, pp.152-162

Levicki, B. (1965), "Construction of houses with great-panelling technology" (translation from Polish origin), *Mqszaki Könyvkiadó*, Budapest

Gilyén, J. (1982), "Structures of panel buildings. Design. Calculation", *Műszaki Könyvkiadó*, Budapest

Gilyén, J. (1998), "Retrofitting of residential prefabricated building in Hungary", *Proceedings, IABSE Colloquium*, CD-ROM Reference Numbers, 2154. Berlin

Simurda L. (1984), "Shaping and load-capacity of vertical joints of panel buildings", Budapest, *Journal Magyar Építőipar*, 1984/1, pp.663-671.

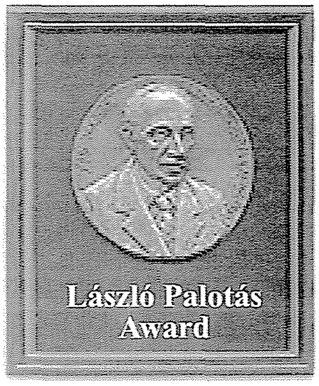
Kaliszky, S.- Györgyi, J.- Lovas, A. (1983), "Evaluation of 1 to 1 scale model experiments with wall-panels", Budapest, *Journal Magyar Építőipar*, 1983/6, pp. 328-334.

Prepeliczay, Gy.-Szabó, L. (1971), "Applying computers for calculating panelling buildings", Budapest, *Journal Műszaki tervezés*, 1971/1, pp. 51-54.

Technical Specification ME 95-72(74), „Design and construction of panelling buildings, 1st part. General requirements of design, calculation and quality" Budapest, *Informating Centre of Constructing Affairs*

Dr. Jenő GILYÉN (1918) licensed architect (1943), titular professor. He taught at the Technical University of Budapest between 1943 and 1947, after which he was the controlling engineer for various constructions. Starting in 1950 he worked as the leading structural designer of Budapest Stadium. For this work he received Kossuth Díj (=National Award) in 1954. Between 1955 and 1960: Standardisation of public buildings; 1960-1980: Chief Structural Engineer of Hungarian panelling construction, elaboration of National Dimensioning Prescriptions in Panelling, etc. Recently he has led courses in post-graduate training for engineers and architects on old building structures.

PRESENTATION OF LÁSZLÓ PALOTÁS AWARDS
– FIRST TIME IN 2000 –



Even if our world is full of engineering structures, people rarely think of the civil and structural engineers who erected them. Engineering structures like houses, halls, bridges, dams, etc., are important parts of our environment. We can not only use them but we can admire them or hate them.

A structure starts with its design and the production of the constituent parts of its materials. Quality control during construction is an essential element of a satisfactory result. Universities and colleges contribute to the learning process and assist in the material dreams of future construction. Codes of practice for buildings and structures are developed to create a common language amongst engineers. On the other hand extraordinary constructions that differ from the conventional lay in wait to surprise the engineer.

Civil and structural engineers collaborate throughout this process with architects, chemical and mechanical engineers.

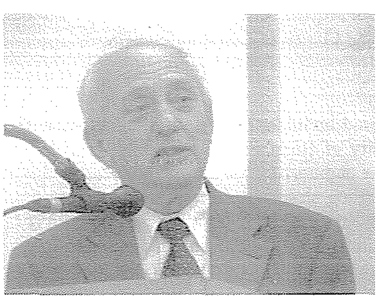
Let us therefore start to appreciate the efforts of civil and structural engineers. The Hungarian Group of *fib* (*fib* = fédération internationale du béton = International Federation for Structural Concrete) has inaugurated an award to recognise Hungarian engineers for their outstanding achievements in concrete engineering. The award is named after Professor László Palotás who was a well-known teacher and researcher both in Hungary and abroad. Generations of engineers studied from his books. He was also one of the presidents of our Hungarian association.

The Palotás Award is given once a year to a single Hungarian engineer living in Hungary and another one living outside the Hungarian borders. The Award winners may represent any phase of concrete engineering and may belong to design, execution, material production, prefabrication, research, development or the teaching profession.

Winners are chosen from the proposals of the members of the Hungarian Group of *fib* by a jury of 7 members elected internally (Dr. Tamás Balogh, Dr. Zsuzsanna Józsa, Dr. Miklós Loykó (chairman), Dr. Gábor Madaras, Lajos Szígyártó and Sándor Zsömböly). They represent the various fields of concrete engineering.

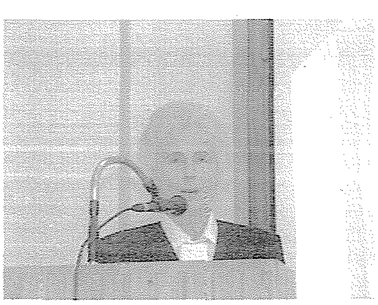
The first Palotás Awards was given on 11 December 2000 at the Budapest University of Technology and Economics, to:

**Péter Wellner, Hídépítő Co., Hungary and
Dr. Gábor Köllő, Transilvanian Technical-Scientific
Society, Cluj-Napoca, Rumania**



Péter Wellner
Palotás Award
winner in 2000
Hídépítő Co, Hungary

Dr. Gábor Köllő
Scientific vice-president
Transilvanian Technical-
-Scientific Society
Cluj-Napoca, Rumania



The decision of the jury was detailed by Dr. Miklós Loykó, chairman of jury. The Awards were handed over by Piroska Palotás, daughter of Professor Palotás. Finally, the Award winners presented their curriculum and achievements. Projects of Award winners were on display during the Award ceremony.

Prof. György L. Balázs
President of the Hungarian Group of *fib*

Piroska Palotás

PÉTER WELLNER
PALOTÁS AWARD WINNER IN 2000

I was very honoured to receive the Palotás László Award at the Ceremony held on 11th December 2000, at the Budapest University of Technology and Economics. How shall I describe the situation? My former bosses, who taught me so much, were present as well as my present and past colleagues: together we achieved success and went through hard times. It was touching.

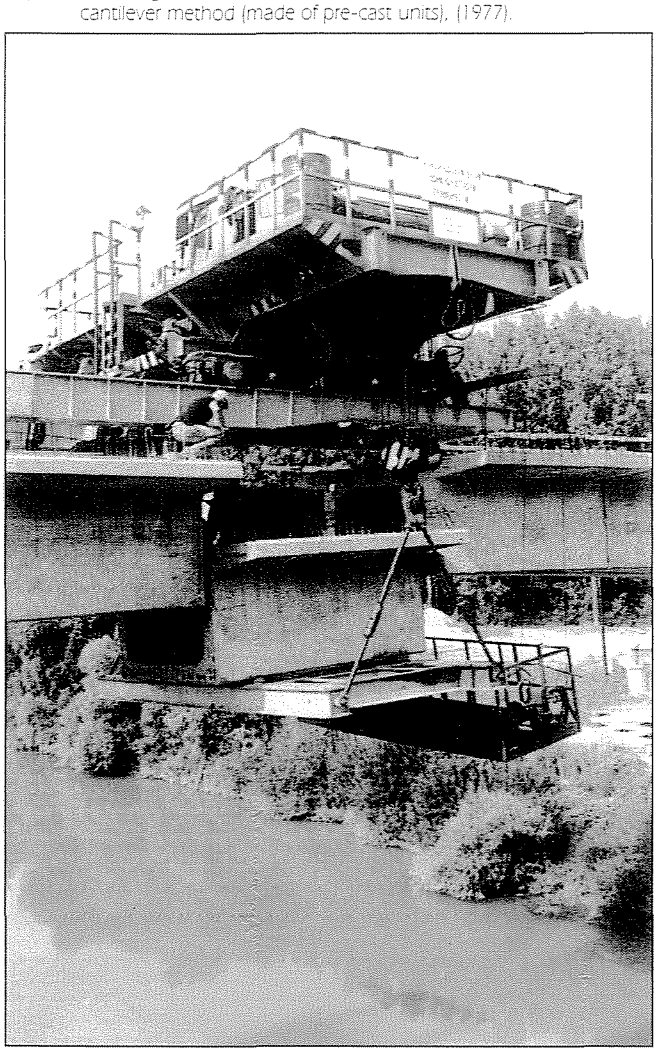
As the award is a tribute to the work of Professor Palotás it is fitting to recall an old memory from his lecturing and teaching which is more than the profession, he turned his palm outwards and addressed his audience: "Kids..."

Good heavens! I wonder what the Professor thinks of my award up there from where there is no way back. Maybe he says to the rest of the experts there: "Kids, he was my student!" I hope it happens so.

Now there is only one thing left: to evaluate - myself. And so I set to the task of looking myself in the face. Mirror, mirror on the wall .... Have I done so much in the field of reinforced concrete and prestressed concrete that I deserve this award? It is a pertinent question, indeed. What have I done anyway?

My first step in the profession was a fortunate one without doubt. I started work at the UVATERV Bridge Engineering Office in the company of excellent experts. I had the chance to participate in great and interesting tasks. It was a time for learning and gaining experience. I couldn't suggest a better choice for any fresh engineer, even now.

Fig. 1: The bridge over Hármas-Körös erected with the balanced cantilever method (made of pre-cast units), (1977).



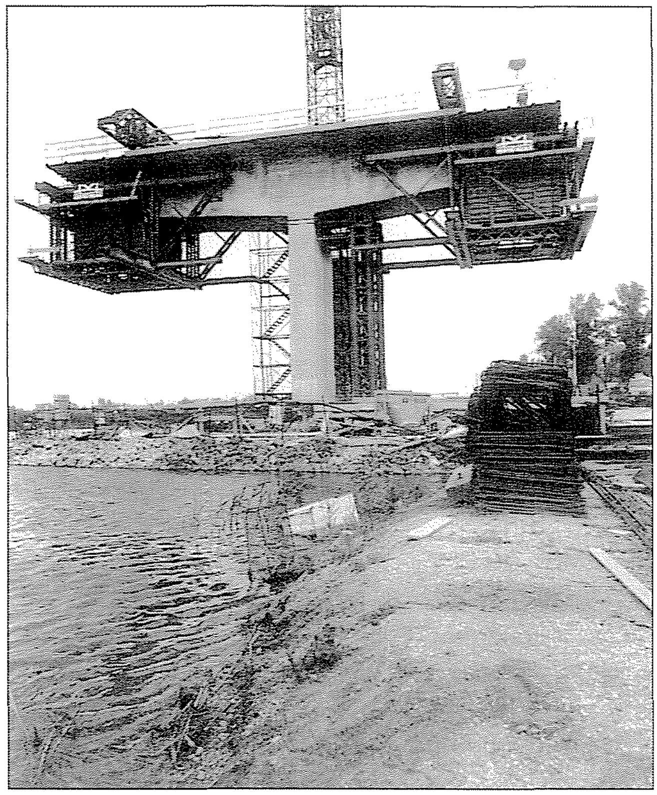
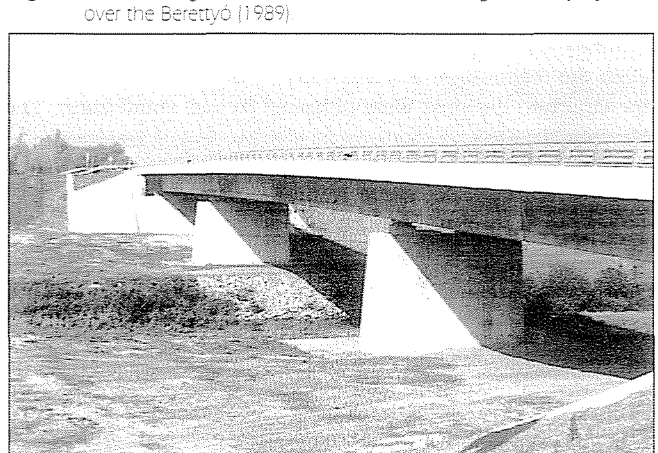


Fig. 2: Balanced cantilever method, M0 motorway, bridge over the Danube at Soroksár (1980).

Only by the time I became a more or less experienced engineer (12 years later) did I get closer to designing prestressed structures. My boss was János Reviczky, who – until then – was almost alone in his fight for using pre-tensioned reinforced concrete in Hungary. From that on, it was the two of us. Soon we were treated with indulgence, later with acceptance and support.

That's how it started. The result: I could participate in the design work of three structures being built with a different construction technology for each.

Fig. 3: The first bridge built with incremental launching at Berettyóújfalu, over the Berettyó (1989).



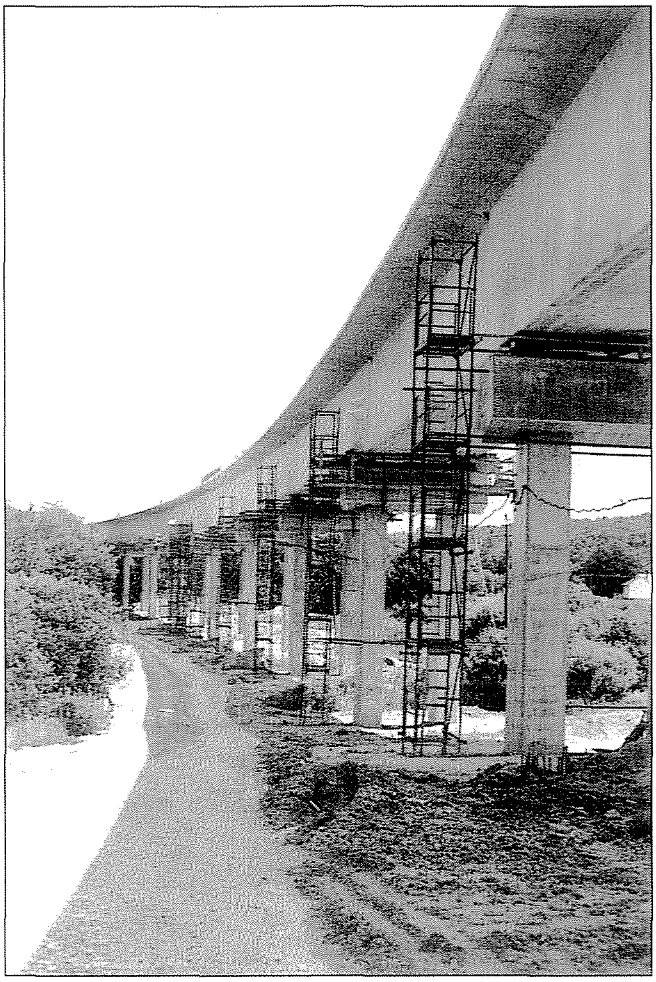


Fig. 4: Viaducts of the Hungarian-Slovenian railway (2000).

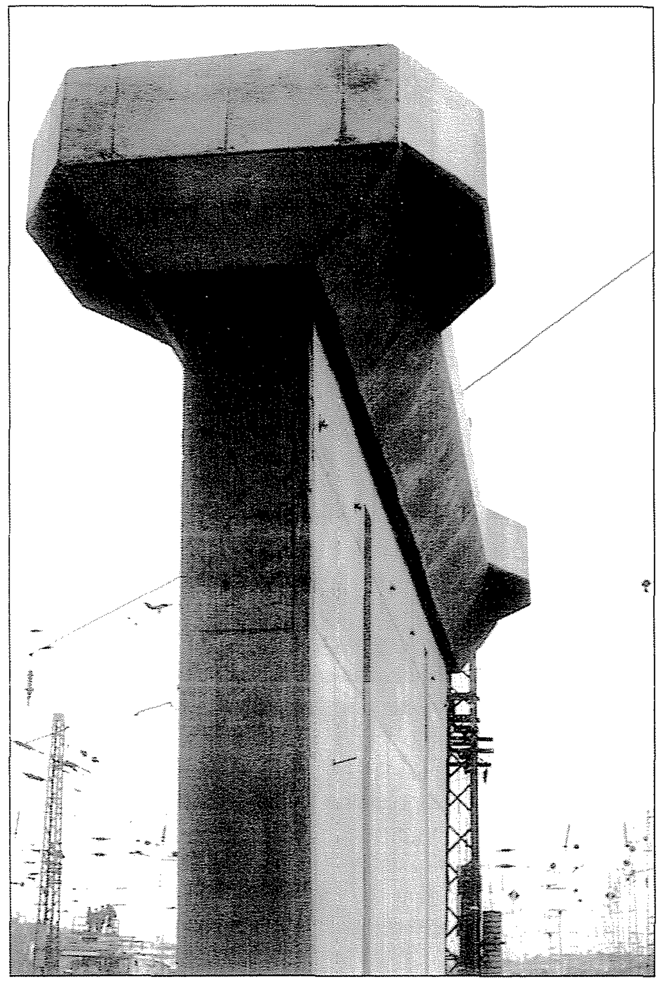


Fig. 5: Pier and cap-beam, M5 motorway Budapest access section (1996).

The first structure erected with the segment construction method (made of pre-cast units) was the bridge over Hármas-Körös at Kunszentmárton (*Fig. 1*). During the execution I worked for the Hídépítő Company. Here we designed the technology for accelerating the construction. In connection with this bridge, our designing and construction teams were given a "State Award". We used the same technology in the design of the ill-famed "naughty" Marx-square fly-over.

My next task was to partake in the introduction of the balanced cantilever method. The first bridge constructed with this method brought from France was the Kis-Duna bridge in Győr (*Fig. 2*).

The third method, incremental launching, was introduced in Hungary under my guidance. The first bridge built with this method was the bridge over the Berettyó at Berettyóújfalu (*Fig. 3*). It was followed by 11 more structures of the same kind. The last of these are the two viaducts of the Hungarian-Slovenian railway (*Fig. 4*). One of them is 1400 m long and possibly the longest bridge in Central Europe. Each design, even the technology and auxiliary structure designs (except the structure designs) were prepared by our small team. The 1400 m long viaduct was built in 14 months, as per our designs. It was a good lesson to learn: successful design work of a major construction means that the phases of the designing are done in parallel, preferably under the same leadership. Construction should be executed with the designer and the Site Manager thinking together, co-operating in those phases of the execution non-separable from the design work. Even more, we are supposed to strive to meet higher and higher aesthetic demands (*Fig. 5*).

This short overview shows respectable work. The question is, to what extent is it my personal achievement. I think it should not be overrated. I always made an effort to work with good colleagues, excellent professionals. I succeeded to connect designers and constructors in our mutual work, and achieved co-operation with the investors and builders. This is what I consider an important achievement.

Now I am only waiting for the answer. Mirror, mirror on the wall....

Péter Wellner

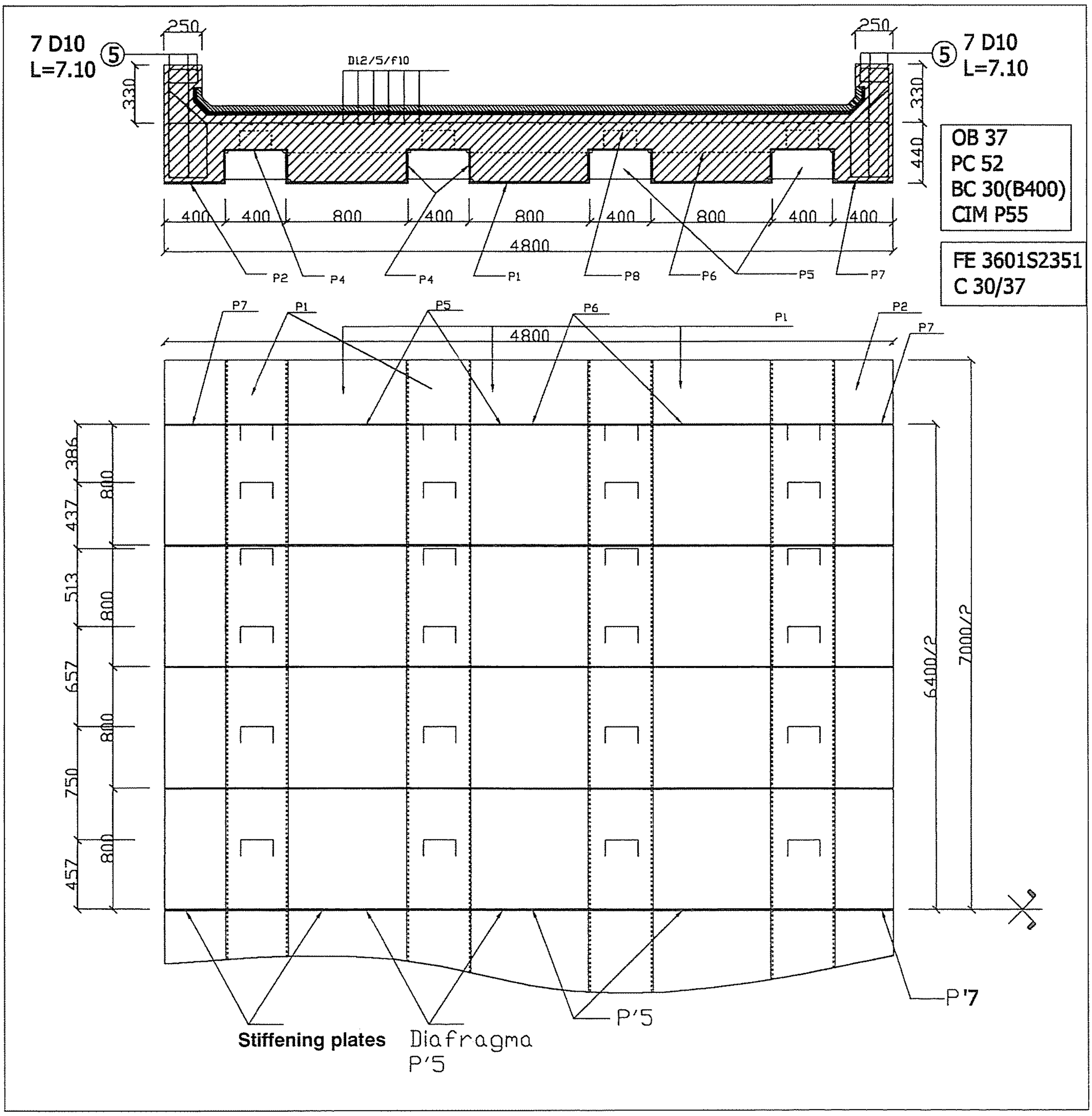
DR. GÁBOR KÖLLŐ
PALOTÁS AWARD WINNER IN 2000

Results concerning the *composite railway-bridges* based on research and design experiences of the last two decades are summarised shortly in this essay. Plate bridges and structures made of closed cross-sectional elements developed at the Technical University of Cluj-Napoca (Rumania) will be presented. Some (10-12 pieces) of these plate bridges running through the railway bedding were put into practice experimentally on the main lines of the Railway Management of Cluj-Napoca.

The increasing speed of trains requires to lead through the super-structural bedding at short and middle span bridges. This needs bridges satisfying the new requirements. Two types of composite plate bridges will be presented, the first one is of compact and the second one is of a hollow cross-sectional arrangement (*Fig. 1*). The advantages of these types of structures relating to the traditional trough bridges are the smaller structural height, higher load-bearing capacity and the better dynamic behaviour due to better stiffness characteristics.

The structure of compact cross sectional arrangement consists of a continuous horizontal steel plate of 8-12 mm thickness and 12 mm thick stiffening steel plates of the same length welded on the steel plate at 400-600 mm distances. Connecting elements assuring the bond between the horizontal plate and the concrete filled on it are welded on the sides of the

Fig. 1: Hollow plate composite bridge.



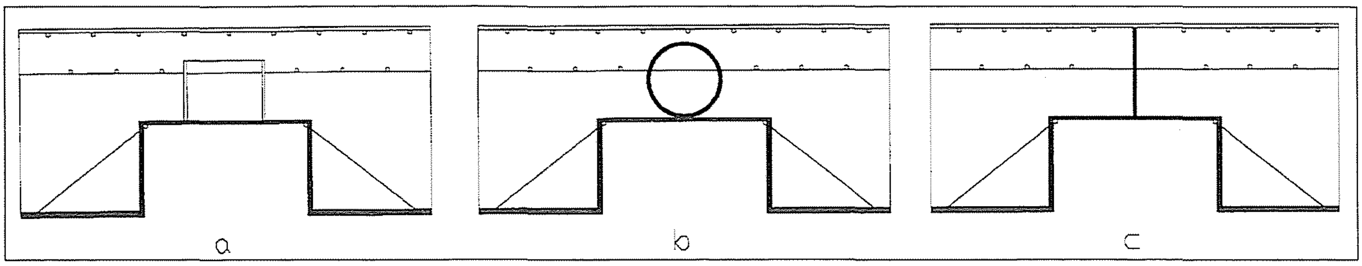


Fig. 2: Hollow composite slabs.

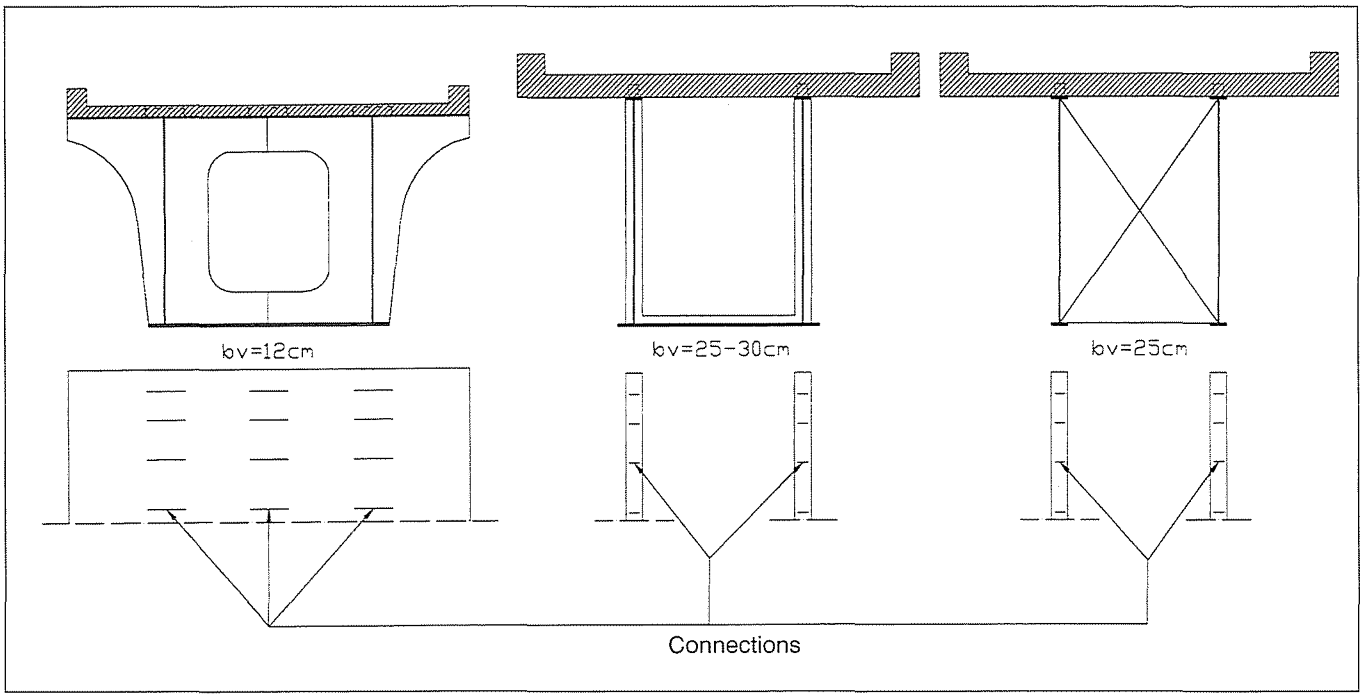


Fig. 3: Traditional composite structures.

longitudinal stiffening plates. With this type of bridges steel profiles have been used for bond elements.

As concrete tension has not been considered during the structural analysis creating holes in the tension zone thereby decreasing the weight of the structure seems to be advantageous. The so-called *hollow composite plates* (*Fig. 2*) have been created upon this principle. The steel structure is made of welded steel plates. The lower horizontal and vertical elements are thicker (12 mm), while the upper horizontal parts are thinner (6-8 mm). The main role of the latter is to hold the stiff or elastic connecting elements. These plate structures are developed for short spans (to 12 m). The ultimate stresses are due to cyclic loading, thus the structural analyses have been made in the elastic range. For these structures using high strength concrete (C 35/45 ... C 50/60) is suggested.

There were in-situ measurements carried out under train loads in order to determine the dynamic factors at the bridges put in operation. The results are favourable so far.

With regard to the other new developments – which are unlike the traditionally known and used steel and concrete composite bridges (*Fig. 3*) - the steel part is a jointed steel girder of closed cross-sectional arrangement. The upper flange is wider and thinner (6-10 mm) than the lower (25-30 mm), which gives a good opportunity to reach a proper bond through the whole width of the structure with the concrete deck of lower thickness (120-160 mm). The weight of such structures is much lower than that of the classical composite structures.

Cross section of a composite bridge structure for a single-rail track created by using one steel girder is shown in *Fig. 4.* The structural height (h) in such cases can be evaluated by h = L / (12-13), where L denotes the actual span of the bridge. In order to achieve a smaller structural height two or more joined steel girders can also be used. The structural height for cases of using two girders (*Fig. 5*) is h = L / (15-16). A further decrease can be achieved by prestressing the steel girder. This way the structural height can go down to h = L / (18-19) for two girder composites. These composite bridge arrangements have good applications for 30 to 50 m spans.

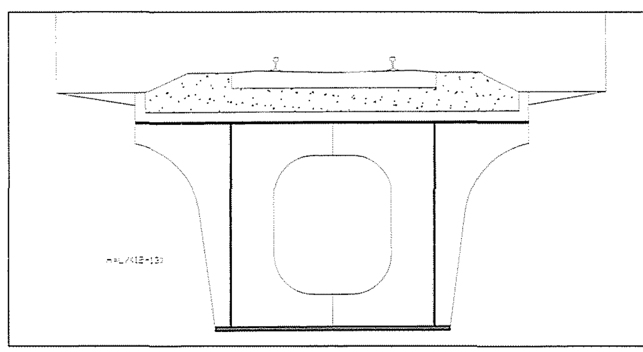


Fig. 4: One girder composite section for single-rail track.

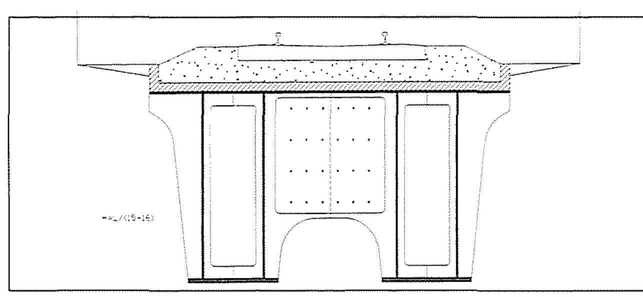


Fig. 5: Two girder composite section for single-rail track

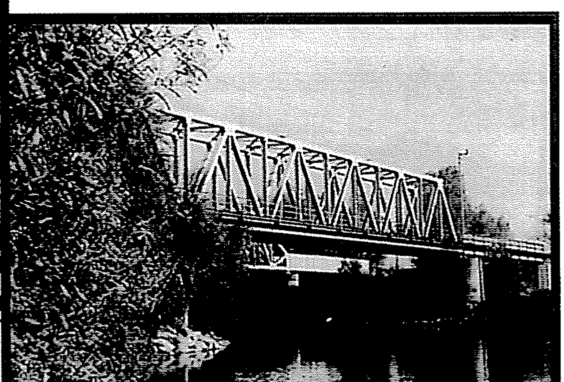
MÁV BRIDGE CONSTRUCTION CO. LTD

Address: 1142 Bp. Mexikói út 71. **Mail address:** 1378 Bp. 64. POB. 13 **Tel.:** 251-0444 **Fax.:** 363-7453, 251-3487 **e-mail:** mav-hidepito-kft @ mav.hu

The MÁV Bridge Construction Co. Ltd. was established on the 1st of August 1992, as the legal successor of the former Bridge Construction Directorate of MÁV (Hungarian State Railways).
The Company consists of a staff of specialists having been experienced in structural engineering and construction engineering for several decades, ensuring short execution periods and good quality of production and construction works applying strict quality control measures.

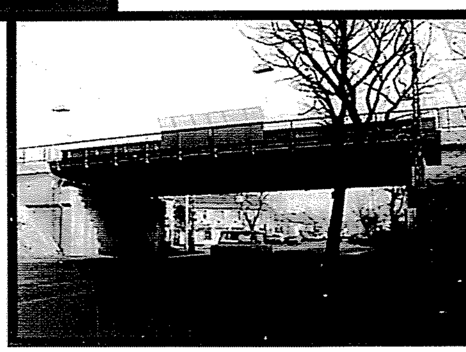
Scope of activity

- Production and assembly of welded and riveted bridge structures;
- Production and assembly of industrial steel structures (crane tracks, hall constructions, etc.);



- Production and assembly of structures of architectural/construction engineering;
- Production of reinforced concrete structures (frame bridges, plates and beams);
- Construction of cast-in-place reinforced concrete bridges;

HÍDÉPÍTŐ KFT



- Construction of concrete and reinforced concrete structures of civil/construction engineering
- Protection of steel and concrete structures against corrosion;
- Shotcreting;
- Sewerage systems;
- Lowering of ground water and injection of ground;
- Testing of steel and reinforced concrete structures with diagnostic instruments;
- Planning and design of bridges and structures;
- Expert survey

TÜV CERT DIN EN ISO 14001

ÉPÍTŐ IPARI NÍVÓ DÍJ 2000

HÍDÉPÍTŐ

ÉPÍTŐIPARI MESTERDÍJ 1996

TÜV CERT DIN EN ISO 9001

HÍDÉPÍTŐ RÉSZVÉNYTÁRSASÁG

H-1138 Budapest, Karikás Frigyes u. 20. Mailing address: H-1371 Budapest 5 P. O. B.: 458
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Extension of the South-Pest Wastewater Treatment Plant

The State-owned Hídépítő Company, the professional forerunner of Hídépítő Részvénytársaság was established in year 1949 by nationalising and merging private firms with long professional past. Among the professional predecessors has to be mentioned the distinguished Zsigmondy Rt., that participated, inter alia, in the construction of the Ferenc József (Francis Joseph) bridge which started in year 1894.

The initial purpose of establishing Hídépítő Company was to reconstruct the bridges over the rivers Danube and Tisza, destroyed during the Second World War, and this was almost completely achieved.

The next important epoch of the bridge builders was to introduce and to make general the new construction technologies.

Even among these can be judged to outstanding the bridge construction by balanced cantilever method, the experts having participated in it were awarded the State Prize. By this technology were constructed five bridges in the region of rivers Körösök and this was applied at the fly-over of Marx square (today Nyugati square) in Budapest, still the most up-to-date two-level crossing in the capital requiring the minimum maintenance works.

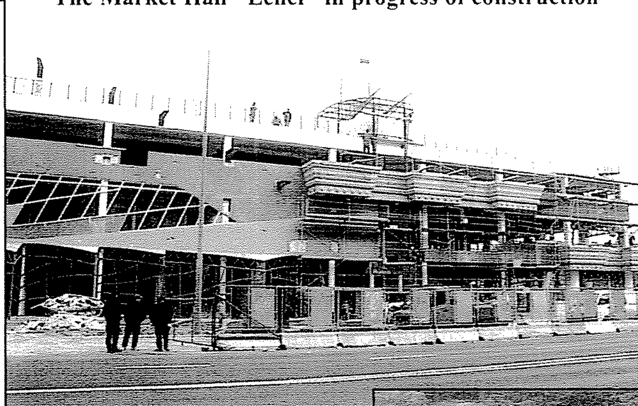
The next big step was the introduction of the so-called cast-in-situ cantilever bridge construction method. This was applied for the bridges constructed over the Mosoni branch of the Danube, for the road bridge over river Tisza at Csongrád and for the bridge of the motorway M0 over the Soroksári branch of the Danube.

Because the strong quantitative growth in the field of bridge construction, the use of prefabricated bridge elements at that time seemed to be a very great development and the "Hídépítők (Bridge Builders)" undertook a significant piece of the work in its development and introduction.

A number of up-to-date monolithic bridges were constructed on the section by-passing the city of Győr of the motorway M1.

An important result of the technologic development in the bridge construction was the introduction of the so-called incremental launching method.

The Market Hall "Lehel" in progress of construction



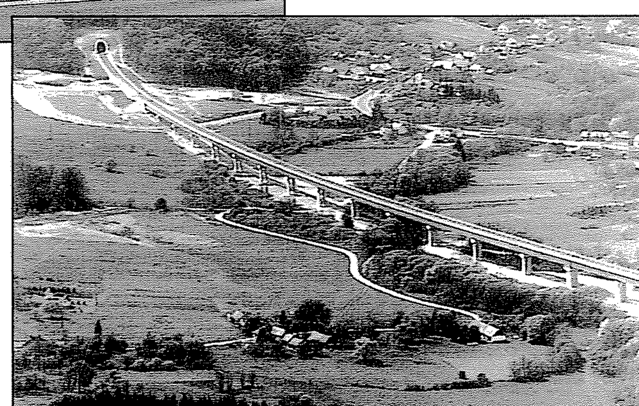
In the period from year 1989 up to now yet 22 bridges were constructed by this method, mainly on the base of the designs prepared by the Company's own Technical Department.

Beside the bridge construction important results were achieved by the "Hídépítők (Bridge Builders)" in the field of foundation's technological development as well. Among others they have introduced and made general the use of the bored piles with large diameter, of jet grouting and of CFA (Continuous Flight Auger) pile preparation, further also a new method, subject of patent protection, was developed for very quick and economic constructing bridge piers in living water.

In the recent two years a great number of important professional recognitions were awarded the high level activity in the fields of bridge construction and bridge designing.

- High standard Prize of Building Industry for designing and constructing a bridge in length of twice 187 m on the section accessing Budapest of the motorway M5 (2000),
- Innovation Grand Prix for designing and constructing in record-time (one year) viaducts in length of 1400 m and 200 m on the Hungarian-Slovenian railway line at Nagyrákos (2001),
- Prize of Concrete Architecture for designing the viaducts at Nagyrákos (2001).

Nowadays, beside the high level activity in the field of bridge construction, the Company has extended its scope of activity by taking part in winding up the backwardness in infrastructure (construction of the Drinking Water Treatment Plant at Csepel, extension of the South-Pest Wastewater Treatment Plant, construction of the Solid Waste Spoiling Area of Salgótarján, sewerage development in the towns and villages Vecsés, Szeged, Dabas, Abony, Tiszaföldvár, etc.) as well as by the introduction of the architectural engineering profile, because of being squeezed out of the construction of motorway bridges.



The 1400 m long viaduct on the Hungarian-Slovenian railway line at Nagyrákos

By working in good quality the Company makes efforts to inspire the confidence of the Clients. For this purpose have been introduced and operated the Quality Assurance and Environment Controlling Systems meeting the requirements of the international Standards ISO 9001:1994 and ISO 14001:1997, justified by international certificates.

The Company is hopefully awaiting the new tasks in order to enhance the reputation of the "Hídépítők (Bridge Builders)".