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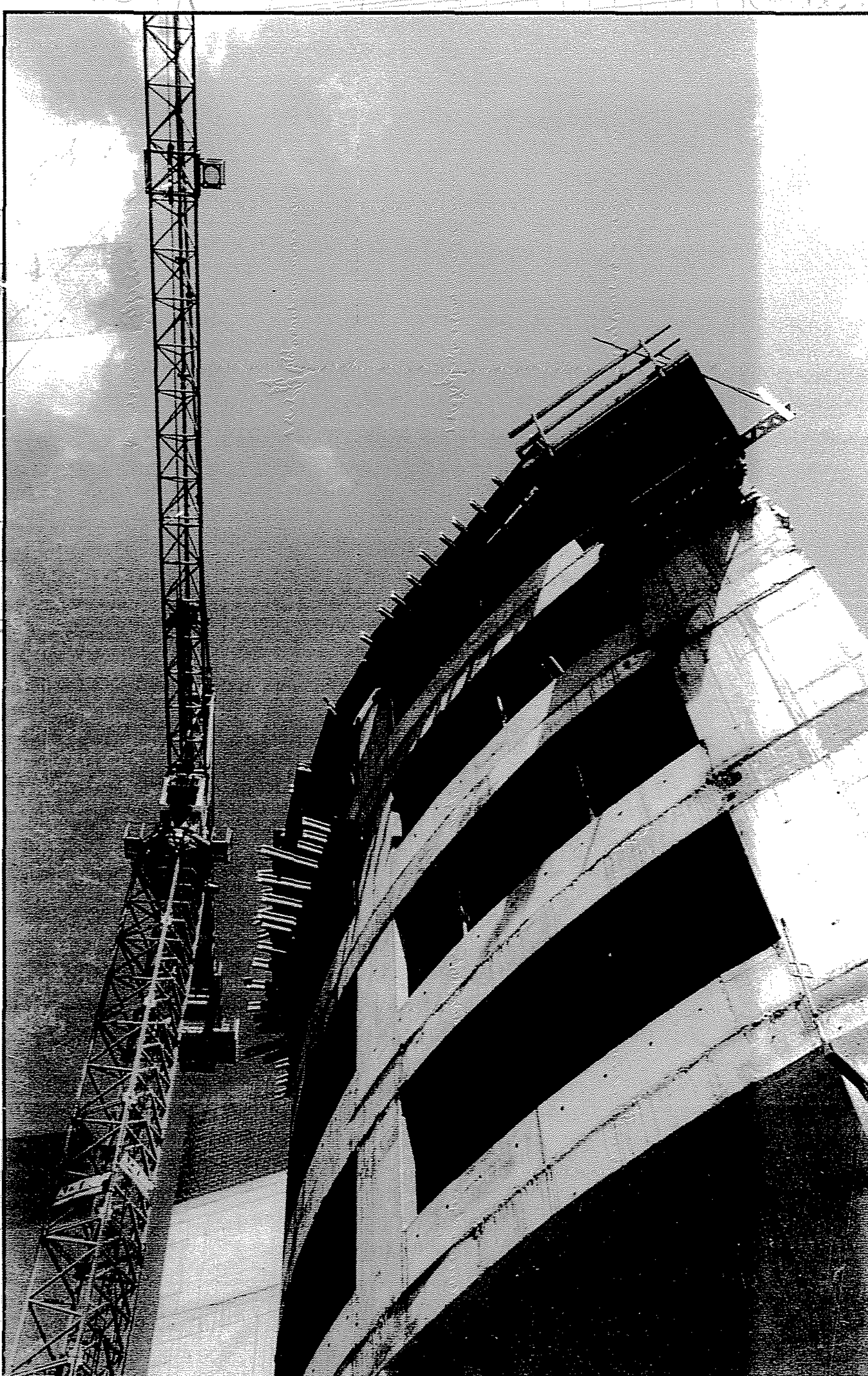
CONCRETE STRUCTURES

ANNUAL TECHNICAL JOURNAL



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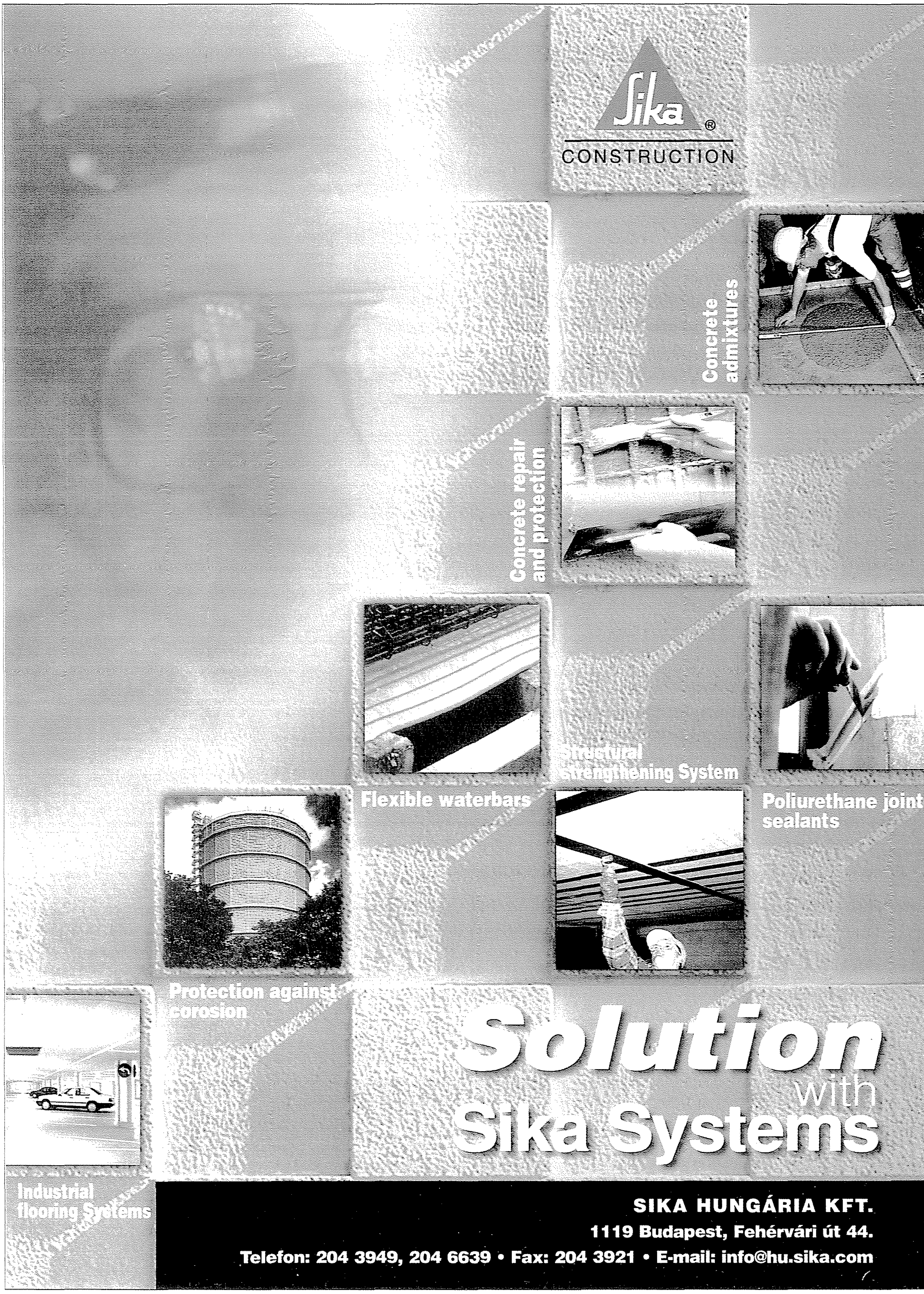
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Dear reader,

You have in hand the latest Hungarian technical journal and it is a pleasure for me to present it as editor-in-chief.

The last century and a half has produced notable developments in concrete technology and it can now lay claim to being the most widely used construction material. Structural concrete offers enormous design flexibility and is an economical solution to almost any structure.

We felt a need for a journal which would concentrate solely on this *extraordinary material.* Nowadays, many countries have a definitive journal for this particular field of interest, while papers on concrete's technological developments in Hungary have so far been published in various, non-specific journals. The Hungarian Group of *fib* (*fib* = fédération international du béton = International Federation for Structural Concrete) intends now to bridge this gap.

Just to mention some important milestones from the past: Concrete strength has increased many-fold and it may even reach the strength of steel. New types of admixtures and concrete technology has made it possible to cast without compacting (with self-compacting concrete). Different types of mould and reinforcement systems enable faster construction. On the other hand, prefabrication has maintained its special position offering short erection times. All of the foregoing have contributed to the higher quality and better durability of modern structural concrete as well to its economic value and popularity. Without reinforced concrete perhaps some contemporary structures would not even exist (we think of special buildings and structures such as bridges, tunnels and dams). Meanwhile, higher and higher aesthetic expectations have been fulfilled.

The *principle objectives* of the journal, entitled "CONCRETE STRUCTURES", is to publicise the most recent technical developments in Hungary in the fields of concrete, reinforced concrete and prestressed concrete structures and research. Our aim is therefore to give a forum to practical and theoretical papers related to concrete as a structural material and the scope of the contributions will range into all disciplines connected to realisation, i.e. design, construction, prefabrication, constituent materials, quality management, research and codification.

In addition to conventional normal-weight aggregate concrete, concrete can be made of light-weight aggregates or fibers can be added to improve its characteristics. Alternatively, new possibilities are offered by non-corrosive, non-metallic reinforcement. The title of the journal, CONCRETE STRUCTURES, intends to reflect all these interconnected fields. Papers can be submitted by anyone who finds concrete structures important and who wishes to share their experiences.

As you may imagine, we do not only encourage our potential readers to read our forthcoming issues but also encourage those colleagues who wish to submit manuscripts to support the objectives of the journal.

The journal CONCRETE STRUCTURES does not only offer technical news within our specialist field but also has some new institutional arrangements. In addition to the *editorial board* there is an established *board of reviewers,* members of which are drawn from a pool of well-know Hungarian engineers with several decades of technical experience. Their responsibility is to maintain the quality of published papers. The technical form of submissions we intend to follow has also been modified. For example, manuscripts will not be accepted without conclusions and references. The purpose is to better serve the interests of our *readers* in order to make more information systematically available. On the other hand, we will raise awareness in Hungary and internationally and improve communications through and across the professions associated with concrete generally. The journal will not only be for the members of our association but will also encourage those who want to have an updated knowledge on any defined subject.

The Hungarian Group of *fib* is glad of the opportunity to create this new medium of communication which would not have been possible without the help of our sponsors who are listed below. We would therefore like to take this opportunity to express our gratitude to the sponsors. CONCRETE STRUCTURES will appear as a quarterly journal in Hungarian and once a year in English. The English language issue is intended to include a selection of the papers translated from the four Hungarian volumes.

Finally, I hope that you will find the contents of this English language version of the journal both informative as well as useful and that it provides interesting insights into the technological developments in Hungary. Whether you are a professional in the subject or just an admirer of concrete structures, we trust that we will fulfil your demands now and in the future.

Prof. György L. Balázs
editor-in-chief
president of the Hungarian Group of *fib*

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Execution of "MOM Park" buildings in Budapest, Hungary

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BUILDING WITHOUT FRONTIERS

László Polgár

Motto: "He who acts too late will be punished by life."
(Gorbachev to Honecker in 1989)

Globalisation progressing in quantum leaps and affects all industries including construction. The European Union is also starting to be a reality for Hungary as well. At a slow but constant rate a uniform system of European standards is taking shape. The construction of the METRO wholesale department store chain allows us to take a glimpse of the future. Planning and designing are in strict accordance with Eurocode and there is international co-operation going on in the design and realisation phases.

Keywords: Globalisation, EUROCODES

1. INTRODUCTION

"Your watch is from Switzerland, your cologne is from Cologne..." this is how an old song goes, saying that consumption tolerates no frontiers. Unlike the construction trade. Building projects are very local, they are subject to national standards and local conditions, the costs of transport leave no room for foreign intruders – at least this is what we used to believe. Actually, this attitude has always been unfounded, hundreds of buildings testify to the international character of the construction industry. Even the compulsory architectural style of Socialist Realism in the fifties, the imitation of the Soviet method of building from prefabricated reinforced concrete elements and the common standards of the Council of Mutual Economic Assistance (CMEA) indicate that the construction industry has never been a purely national business. After the change of the political system and when foreign capital began to flow into the country, we tried hard to hold our lines: the Hungarian National Standards (MSZ) had to be used in Hungary, buildings could only be designed by architects registered in Hungary – "we are the only ones who know how to design and build in Hungary...", etc. Even though the uniform standards of the EU are gathering like dark clouds above, our resistance is stronger than ever: Brussels is dawdling, the EC standards are still in an embryonic state and it will take a long time for the new standards to enter into force anyway. This attitude had nothing to do with sound reasoning – we were trying to be afternoon farmers and wait until somebody else did the job for us. Things will change drastically when the tide turns and we become contractors in foreign projects.

2. METRO HUNGARY

The history of the construction of METRO wholesale department stores in Hungary is an excellent illustration of the changes. The Austrian METRO department stores were still only known to "insiders" when METRO indicated their intention to build a department store in Hungary. The course of the events was typical. The model of the first Hungarian METRO department store was the one built in 1993 in Wiener-Neustadt, Austria. The architect, the structural and the mechanical engineers were the same as in Austria (with partners from Hungary, of course). The Hungarian construction industry had still not recovered from the shock of the political changes and hardly had any references at all. There was a total distrust in the MSZ standards that had their roots in the CMEA-era (there used to be a fair amount of experience with these socialist standards: between 1989–1993 10 METRO department stores were built according to them in the former German Democratic Republic). We had a choice between ÖNORM, DIN or EC2. In Hungary we chose EC2 – at least the foreign partners had no better knowledge of it than we did.

The basic structure is predetermined and it can not be changed, but the concrete sizes can still be reduced. The economical Hungarian cross-sections are incomprehensible for an Austrian structural engineer.

The structure requires a lot of manpower; one of the main issues was to economise on building materials (it took some time for the foreigners to understand that at ten times lower hourly wages the results of the economic analyses are rather different). Prestressed main beam, prestressed ridge purlin – partial prestressing had not yet become common in Austria and Germany (even though in 1969 *Thürliman* described the new principles in detail – see *Deutsche Betontage,* 1969).

After the structural erection of two department stores there is more confidence in the Hungarian construction industry and a more favourable scheme can be introduced. A further seven department stores will be built using the methods developed in Hungary.

3. METRO ROMANIA

The next station of the METRO wholesale department store projects was Romania in 1997. In Bucharest, there being acute danger of earthquake, we therefore started with a steel structure. The resulting cost of the structure of the first Romanian METRO department store was three times more than those in Hungary.

This was partly due to the fire proof paint coat and partly caused by the change of column grid – the new department stores were built with a 14 × 21 m grid instead of the former 10 × 20 m size. The earthquake danger also added to the increase of the price. Prior to the erection of the department store in Timisoara we made another attempt to return to the reinforced concrete structure.

The only option was to design in accordance with EC2 because it was also known in Romania (as a matter of fact it was known better than in Hungary because all EC standards were published in Romanian and English complete with examples in an eight-volume edition). The biggest problem was the fireproofing of the pre-stressed reinforced concrete main beams because there were no relevant Romanian standards. The solution was Eurocode 2-1-2 (fire-proofing of reinforced concrete structures). A certificate issued in accordance with this standard brought the green light for the reinforced concrete structure (the detailed calculations were made by Dr. Deák and were reviewed by Mr. Dumitrescu of the Romanian side).

The earthquake tests were performed at the Bucharest University (Prof. Crainic) in co-operation of ÉMI Hungary and INCERT Romania as an excellent example of international co-operation.

Architectural design: Wels (Austria) – Timisoara (Romania); structural engineering: Budapest – Timisoara – Bucharest; mechanical engineering: Linz (Austria); element manufacturing: Hódmezővásárhely, Dunaújváros (Hungary) and Romania (a country where for 10 years there had been no concrete element manufacturing at all). The reinforced concrete structure of the 10,000 square meter department store was completed in 4 months.

The next stop is Brasow. In this case it was cheaper to manufacture the majority of the elements in Brasow with some technical assistance from Hungary (templates, supervision etc.).

4. METRO BULGARIA

The next target country for a METRO wholesale department store chain was Bulgaria. The Romanian experience made the task somewhat easier. Pre-fabrication is well past its heyday in Bulgaria, too. With great efforts we could restart manufacturing in the factory that had seen better times in the past. Planning, however, was going on smoothly – EC2 is well established in Bulgaria.

It is interesting to see what a uniform – though somewhat underdeveloped – European standard means. Difficulties in communication can be tackled more efficiently if the technical thinking is similar.

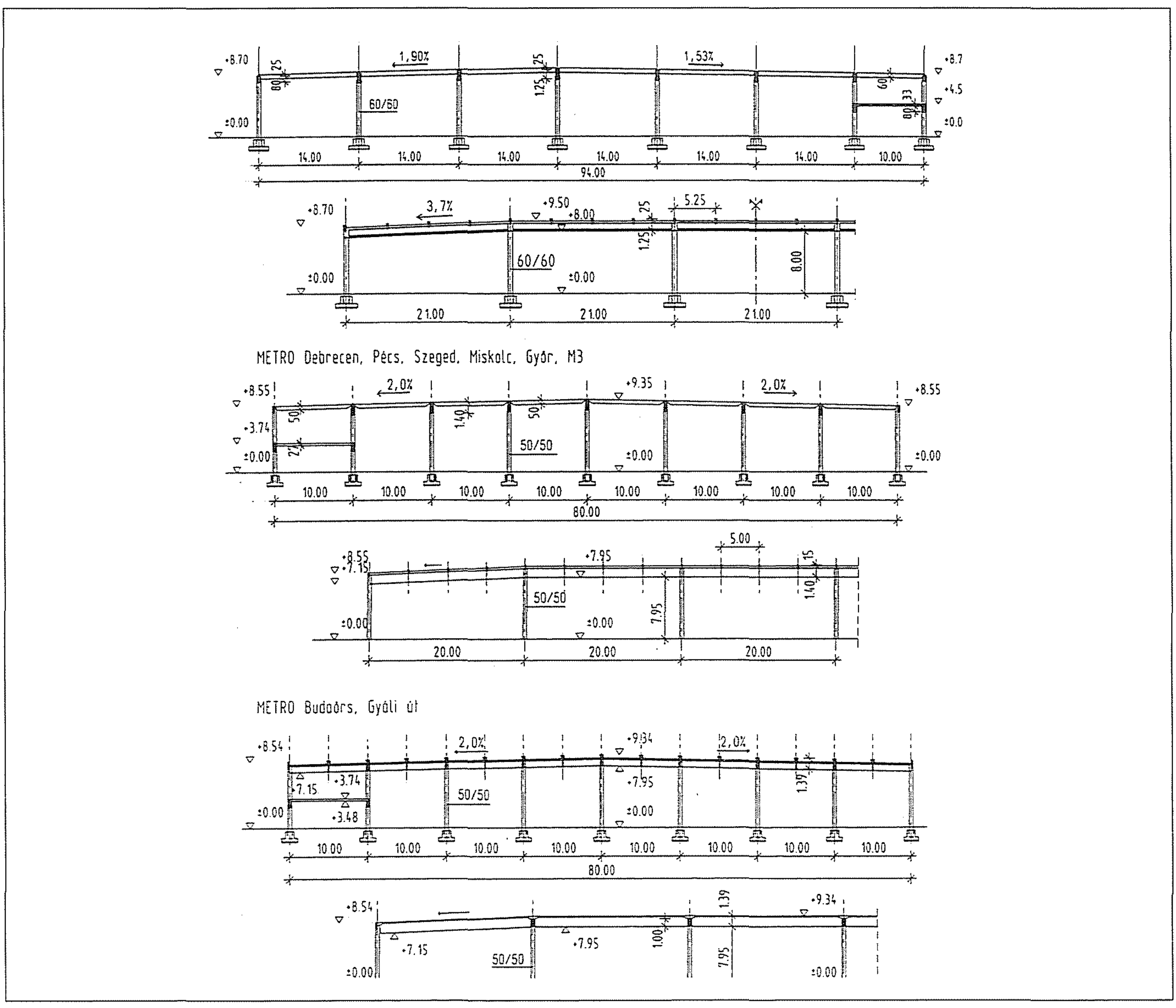
The greatest problem here – not unlike in Romania – is the danger of earthquakes. The "light reinforced concrete frameworks" as they are called in Hungary were unknown both in Romania and in Bulgaria.

High ribbed corrugated steel sheets had never been used there and joints were usually welded.

In Bulgaria it was a generally accepted view that the upper surface of the main beams and the purlins could be brought to the same level so that the steel slab can be fixed to the structure in both directions. The modified structure is the same in Hungary, even though the reason is different. We use this method to reduce structural height. The structural development of the new Romanian and Bulgarian department stores has the same objective.

The METRO wholesale department stores were followed by the RONDO corrugated paper factory in Cluj and the Continental warehouse in Timisoara. The building of the *future without frontiers* is beginning to take shape (*Figs. 1 and 2*).

Fig. 1 METRO department stores in Hungary and Romania, 1994–1999



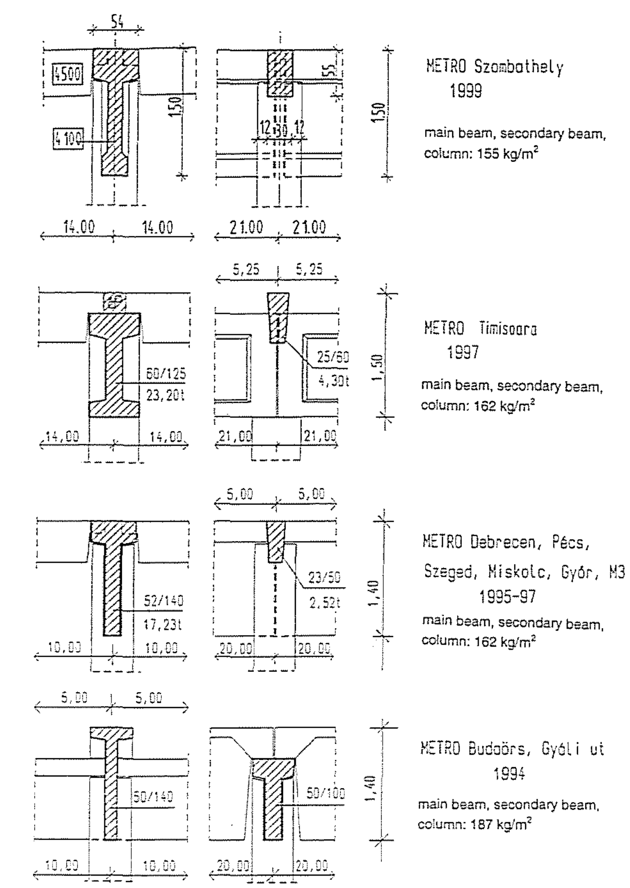
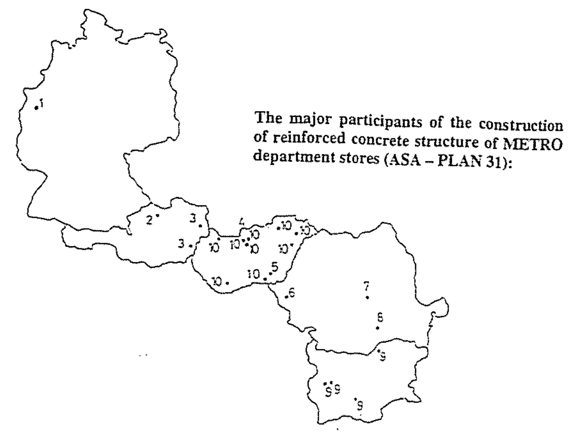


Fig. 2 Details of METRO department stores in Hungary and Romania, 1994–1999



1. METRO Wholesale Trading, central building engineering department
2. Sanitary Engineering: Sulzer Linz (Recesses in the reinforced concrete elements)
3. Roof and Walls: UNGER Oberwart
4. Planning and construction of the reinforced concrete structure: PLAN 31. Mérnök Kft. Budapest ASA Építőipari Kft. Budapest
5. Elements manufacturing: Hódmezővásárhely
6. Timisoara department store: local structural engineering and quality control (INCERT)
7. Brasow department store: some elements manufactured in Brasow (PRESCON)
8. Architectural design, local structural engineering: Bucharest (Prof. Crainic)
9. Sofia, Plovdiv, Ruse (at 4 locations)
10. Hungarian department stores (at 9 locations)

Fig. 3 International co-operation in the design of METRO department stores

5. PLANNNING

In the age of computers the geographical position of the members of the design team has little significance. The main problem arising is caused by lack of compatibility of the various software products. At the beginning the Hungarian Nemetschek–Allplan–Allplot had serious difficulties of communicating with the AutoCad used in Romania and Bulgaria. The implementation of EC2 has not always been easy, either, because the adaptation has not been complete in any of the countries yet. We also had some problems with the different classification of the reinforcing steel used. Stirrups in Romania and Bulgaria are made of a material similar to the Hungarian B38.24, while in Hungary BHB55.50 is used.

The main bars in Bulgaria and Romania are similar to the Hungarian B60.40, but we have long been using mild reinforcement steels of 500 N/mm². Moreover, bar lengths of 18m are not unusual. Thereby, the small cross sections of high strength in our designs resulted in a crowded reinforcement made of the inferior steel bars. On the other hand, the constant danger of earthquakes made it necessary to reduce the weight of the structural elements.

Further design activities in Romania and Bulgaria made it necessary to establish joint design departments. The Budapest – Hódmezővásárhely – Cluj – Sofia network is operating with ever increasing efficiency. Today one of the biggest hurdles to further development is the backlog in Hungary in the implementation of EC2 (both Romania and Bulgaria are ahead of us). Hopefully this will change soon (*Fig. 3.*).

6. CONSTRUCTION

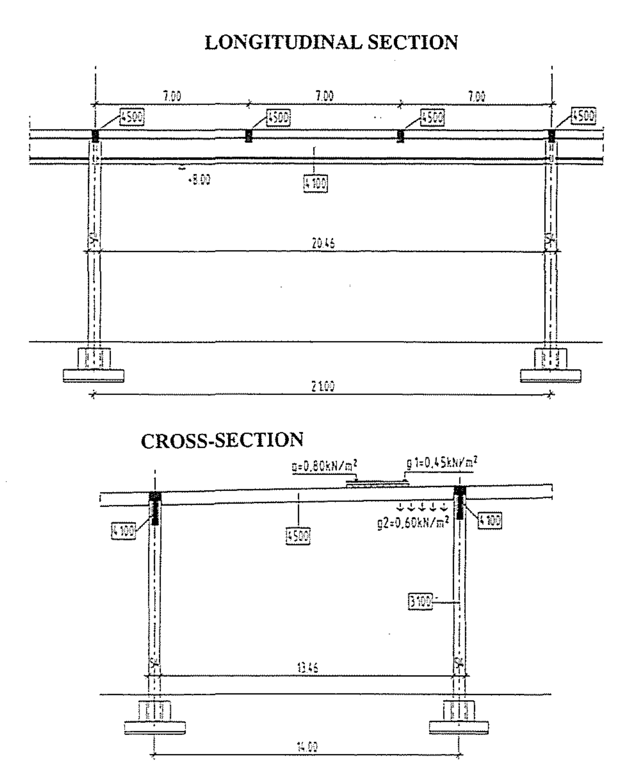
It still holds true of prefabricated concrete structures that the closer they are manufactured to their place of destination, the better. It can make sense, however, to transport moulds and templates even to distant places and if the technology used in the various factories is similar, there are no disadvantageous factors.

Apart from economic considerations, time was another major factor to reckon with in the case of the Sofia and Plovdiv projects. There was simply no time to manufacture new templates.

7. AN EXAMPLE

The example below illustrates the method of calculations in the case of international co-operation. We had to find a method that is comprehensible despite the differences in the language used. The values in the box will be used in the computerised calculations later on. "Manual" calculations are, however, sometimes more important because they give an idea about the processes involved in finding a solution to a problem. As in Hungary, checking according to the MSZ standards is also a requirement: design moments and forces should be calculated according to both the Hungarian and the European Standards.

Structural design is performed by the established international method of "m" "u" and "w", which is easily comprehensible for all foreign structural engineers (alas, in Hungary, young designers are not familiar with this practice even though it is widely used all over Europe, from Hamburg to Sofia).



Reinforced concrete sceleton: 155 kg/m² self load

4500	secondary beam M_{Sd}=581,2 kNm V_{Sd}=166,1 kN	5,6 t/db	C 45/55
4100	main beam M_{Sd}=3111,5 kNm V_{Sd}=631,8 kN	22,2 t/db	C 40/50
3100	column	6,6 t/db	C 30/37

Fig. 4 METRO department store in Hungary, 1999 (as planned)

$g_1+g_2 = 0{,}45 + 0{,}60 = 1{,}05$ kN/m²

		MSZ V_{Sd}; M_{Sd}	EC2 V_{Sd}; M_{Sd}
$V_{G4500} = \boxed{1{,}05 \cdot 7{,}0} \cdot 7{,}0 + 56/2$	= 79,45 kN	107,25	87,40
$V_{Q4500} = \boxed{0{,}80 \cdot 7{,}0} \cdot 7{,}0$	= 39,20 kN	58,80	54,88
		166,05	142,28
$M_{G4500} = 1{,}05 \cdot 7{,}0 \cdot 14{,}0^2/8 + 56 \cdot 14/8$	= 278,08 kNm	375,41	305,89
$M_{Q4500} = 0{,}80 \cdot 7{,}0 \cdot 14{,}0^2/8$	= 137,20 kNm	205,80	192,08
		581,21	497,97
$V_{G4100} = 2 \cdot V_{G4500} + G_{4100}/2 = \boxed{2 \cdot 79{,}45} + 222$	= 380,90 kNm	514,22	418,99
$V_{Q4100} = 2 \cdot V_{Q4500} = \boxed{2 \cdot 39{,}20}$	= 78,40 kNm	117,60	109,76
		631,82	528,75
$M_{G4100} = 2 \cdot V_{G4500} \cdot 7{,}0 + G_{4100} \cdot 21/8$	=1 695,05 kNm	2 288,32	1 864,56
$M_{Q4100} = 2\, V_{Q4500} \cdot 7{,}0$	= 548,80 kNm	823,20	768,32
		3 111,52	2 632,88

Pre-calculation

4500

$$\mu = \frac{581{,}21 \cdot 10^6}{300 \cdot 490^2 \cdot 30} = 0{,}269; \quad \omega = 0{,}3381$$

$A_S = 0{,}3381 \cdot 300 \cdot 490 \cdot 30/435 = 3428$ mm²

$11\phi20 \rightarrow 3454$ mm²

4100

$$\mu = \frac{3111{,}52 \cdot 10^6}{540 \cdot 1420^2 \cdot 26{,}7} = 0{,}107; \quad \omega = 0{,}1150$$

$A_S = 0{,}1150 \cdot 540 \cdot 1420 \cdot 26{,}7/435 = 5413$ mm²

$18\phi20 \rightarrow 5652$ mm²

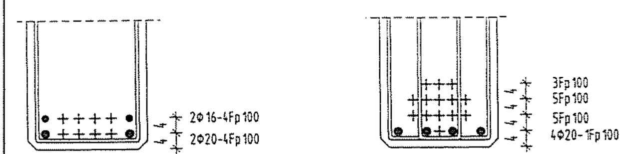


Fig. 5 METRO department store in Hungary, 1999, calculation No. I.

References to the individual elements of the structure are extremely important in the case of a "distant design project". Therefore, all the design sheets, element signs, stresses, location codes and quality certificates use the same codes no matter how they are transmitted from one country to another – by wire, by car or by rail (*Figs. 4 and 5*).

Evaluation of the calculations

In the case of prestressed beams the most difficult problems are:

- to calculate the ideal stress to be used;
- to adjust the ideal proportion of deflection in both directions;
- to guarantee that the structure is fire-proof (by means of increasing the number of steel rods with a simultaneous reduction of span wires or by shifting the span wires closer to the surface), and
- to investigate economic considerations (a span wire is a cheaper way of absorbing stresses than reinforcement rods).

The many pages of computer assisted calculations help to evaluate the *problem (Fig. 6.)*. It is not necessary to print the outcome of the individual runs because an experienced engineer weighs the results of the individual calculations on the screen. However, due to the different price levels, the outcome may be different in the individual countries (i.e. Hungary, Romania and Bulgaria).

In the case of the main beam there is another issue, namely the thickness of the ridge. With a view to the fact that recesses must be made in the ridge for the various pipes and cables, it would not make sense to go for the smallest possible thickness, even though this way the weight of the structure could be reduced significantly. The danger of earthquakes is an important aspect to consider both in Romania and in Bulgaria.

In the view of the Romanian and Bulgarian engineers it is highly recommended to choose a method where the upper surface of the purlins is flush with the upper surface of the main beams of the roof. This is where the concept of purlins set into a "pocket" comes from.

The Hungarian regulations require that the structure must be MSZ-certified:

Purlin No. 4500:

$N_{Sd} = A_s \cdot f_{sd} + A_p \cdot f_{pd} =$

$= 10.28\text{cm}^2 \cdot 42\text{kN/cm}^2 + 8.00\ \text{cm}^2 \cdot 133\ \text{kN/cm}^2 = 1496$ kN

$x_c = 1496/(2.9 \cdot 30) = 17.2$ cm

$z = 0.55 - 0.06 - 0.086 = 0.4$ m

$M_{Rd} = z \cdot Z_d = 0.4 \cdot 1496 = 598.4\ \text{kNm} > M_{Sd} = 498{,}0$ kNm

$(M_{Rd}/M_{Sd} = 1.2)$

Purlin No. 4100:

$N_{Sd} = 12.56\text{cm}^2 \cdot 42\text{kNm/cm}^2 + 14\text{cm}^2 \cdot 133 = 2390\text{kN}$

$x_c = 2390/(2.7 \cdot 54) = 16.4$ cm;

$z = 1.50 - 0.08 - 0.082 = 1.34$ m

$M_{Rd} = z \cdot Z_d = 1.34 \cdot 2390 = 3203\text{kNm} > M_{Sd} =$

$= 2633\text{kNm}\ (M_{Rd}/M_{Sd} = 1.216)$

Obviously, the MSZ-certification has no practical significance but the law must be obeyed. A global world does not care for national standards, but the results do contribute to the tendency that foreign customers refuse to accept MSZ-based structural calculations. (Because it is always proven that the MSZ requires far less than the international standards.)

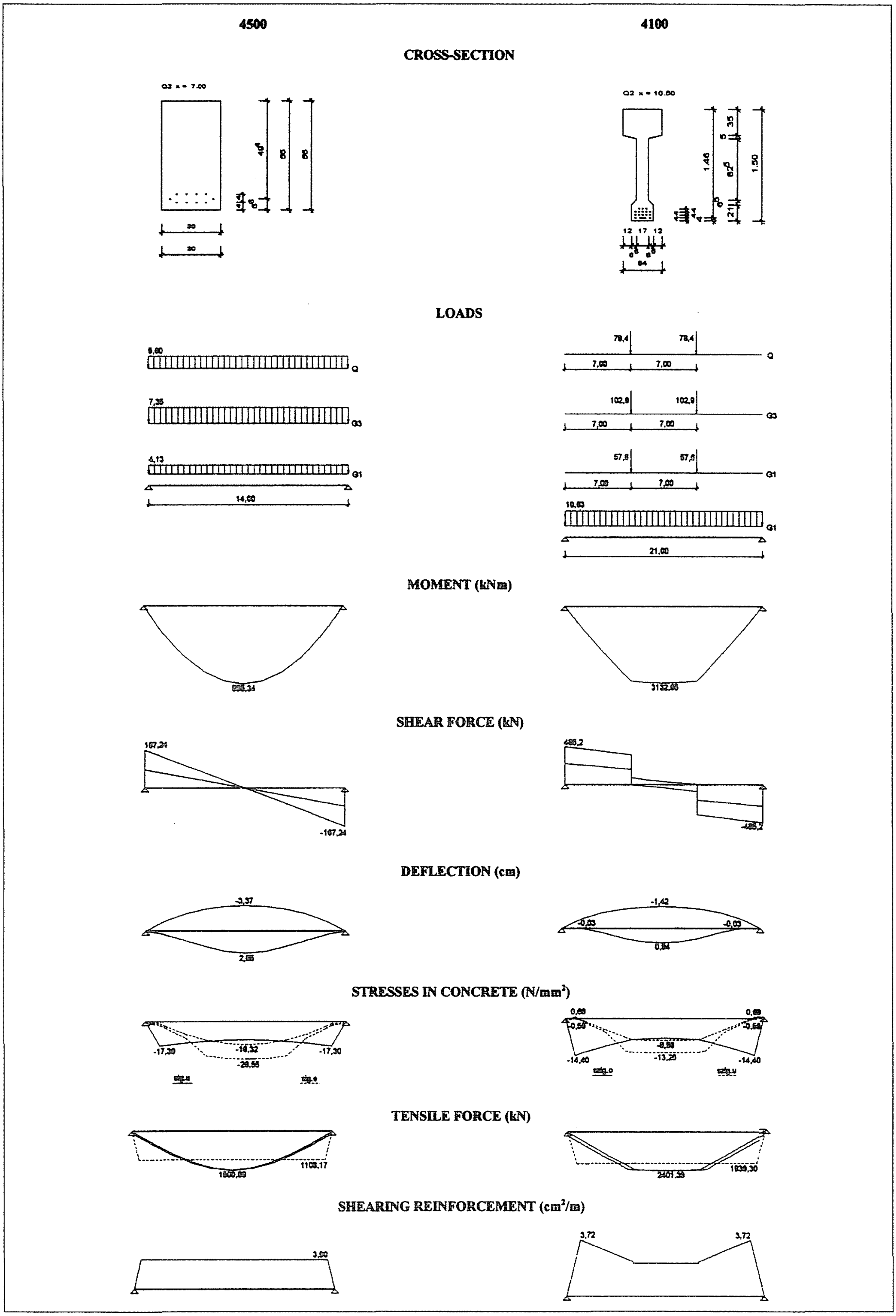


Fig. 6 METRO department store in Hungary, 1999, calculation No. II.

Things are somewhat different in Bulgaria and Romania. These countries are less integrated into the European Union practices. They try to make up for their backlog by feeding a lot of energy into disseminating the use of EC2 standards. In practice this means that they readily accept structural and other designs made in accordance with EC2. Conversely, for instance, a German supervisor insists on the use of the German version of EC2 (e.g. where V_{Rd} has a smaller value for shear forces).

8. CONCLUSIONS

By taking a look at the erection of METRO wholesale department stores in Hungary, Romania and Bulgaria we have tried to illustrate the effects of globalisation on reinforced concrete construction. We are only at the beginning of the process but the example above shows what is to come in the future.

The network of Plan 31 H, Plan 31 Bg and Plan 31 Ro is the first of its kind and proves that it is possible to work without frontiers on the world wide web (*Fig. 7*).

We hope that we have taken the first steps and that time will prove that we were right in doing so.

9. ACKNOWLEDGEMENTS

The METRO department store chain has become global in quantum leaps and there are some 180 units in Europe as proof of their business policy.

We believe it has been a privilege to have their faith in us and to have had an opportunity to work for them and to contribute to the construction of the structure of the Central European METRO department stores.

10. LEGEND

The continuous work in two standard systems (MSZ and EC2) makes life hard for a structural designer. When it comes to international co-operation only the signs used in EC2 can be used.

Fig. 7 METRO department store, Timisoara

11. LITERATURE

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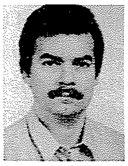
REINFORCED CONCRETE SLUDGE DIGESTERS




Gábor Zoltán Péter – Dr. László Tóth

At the beginning of the article authors give a summary of functional demands and requirements, upon which the concept of reinforced concrete sludge digesters can be created by civil engineers in co-operation with technologist mechanical engineers concerning geotechnical parameters of soil as well. Further the article deals with matters of form and structural problems about the constructions mentioned. It is stated that post-tensioned shell structures constructed from complex shell elements meet more properly static and watertightness requirements. Finally authors present principal trends of structural design and building technology mentioning some foreign examples and describe the post-tensioned digesters of Debrecen, which are the most significant and successful engineering product of home wastewater treatment technology in the past decade.

Key words: form, shell structure, watertightness, gastightness, crack free concrete, environmental integration

1. INTRODUCTION

Elimination of backwardness in wastewater treatment has a key role in the pressing environmental demands of Hungary. Development of wastewater discharge, realisation of mechanical & biological wastewater treatment, reduction, neutralisation and final disposal of wastewater sludge quantity originated in treatment process are of great importance. Consequently, sludge digesting problem comes into the limelight, since secondary energy production and utilisation possibilities are given in addition to the accomplishment of the main objectives of the process. The most important establishments of these rather complex technological systems are sludge digester constructions considering engineering aspect.

This article deals with structural problems of reinforced concrete digesters, and after a short international comparison intends to present the inland situation primarily from design view points.

2. FUNCTION OF DIGESTERS

In these special engineering constructions wastewater sludge digestion is going on at 33 – 35 C° with periodic or continuous mixing. Digested sludge slid down into the sump-like area of the construction is discharged by pump. Biogas originated during digestion is derived on top of the structure. Floating crust originated at liquid level should be systematically crushed for the easier segregation of biogas. As a consequence these technological demands "determine" the suitable geometric form in more aspects, which mainly corresponds to a circular symmetric shell structure.

Besides the technologic demands touched by headwords, digesters should be watertight and also gastight at the upper parts. For technologic reasons the whole exterior surface should be provided by heat-insulation.

Digesters are establishments of great bulk, which are susceptible to greater gross subsidence compared to other type of structures. The size of subsidence depends on the foundation solution and soil parameters. Consequently, the displacements should be harmonised with connected buildings. In some cases this is indispensable for structural, static reasons, and also is demand for operation safety of technologic pipes.

3. STRUCTURE AND FORM

The form of a perpendicularly stretched egg-shaped, doubly curved shell meets the demands previously mentioned in the most properly. Some beautiful solutions are shown in Figures 3–440, 3–442, 3–443 of the Engineer's Manual Volume II (1984) written by Gy. Márkus. It is easy to understand that a solution like the mentioned is the most optimal as far as structural aspects and force analysis are concerned. Due to high water pressure, annular tensile forces are determinant. Doubly curved shell structures are favourable concerning static reasons especially if meridian curve is a flexible line without any vertex points. Bending moments originate at every vertex point even at the joints of different surface parts. Realisation of a bending free shell structure is considered practically impossible, but it has a great advantage if response of the significant part of a complex shell structure approaches to the structural behaviour of membrane. Obviously, even an ideal shaped shell structure should be founded, so bending moments occur unavoidably in the vicinity of the supporting lines or surfaces. Constant and non-uniform changes of temperature cause bending moments as well, which necessarily occur due to technologic demands and climatic relations of Hungary.

General statements of shell theory detailed above refer equally to constructions of 1000–10000 cu.m volume demand, which can be found in practice of Hungary most, not depending on the value of standard annular tensional forces and bending moments. However it can be observed that geometric properties of the structure influence stresses the. Evidently, selection of structural solution depends on stresses. In case of normal reinforced concrete structures, constructions with limited crack width or perhaps crack free structures meet watertightness requirements. The latter can be practically realised only by post-tensioning.

By experiences as far as digesters of low volume are concerned non-prestressed concrete constructions are economic but as for digesters of high volume post-tensioning is inevitable and more economic as well. Especially regarding constructions of more than 4000 cu.m. volume.

Other characteristic type of sludge digesters made of circular symmetric shell structure is the shallow "oil tank like" construction bordered with cylindrical wall. Mixing of liquid bulk, sludge discharge and biogas derivation require fundamentally different technologic machinery in these cases, and operational costs are of more significance. However application of these constructions is reasonable since building of main structural elements is simpler according to building technology.

Building of digesters with doubly curved shell structure need cradling of high standard. It is a problem of occasional examination, how the excess cost of cradling relates to the value of building materials can be saved up. Complex economic assessment should not be ended with the analysis of building costs, for technologic mechanical and operational costs are also significant.

Regarding a special engineering construction, problem of economic efficiency is essentially impacted by the volume of building materials needed. As for sludge digesters of great volume, application of shell structures with optimal geometric shape should be aimed, because stresses of these structures are practicably the less. A shell structure, which is near spherical needs the less specific use of materials, therefore egg-shaped digesters can be considered optimal in addition to technological problems.

Besides professional aspects mentioned above this type of establishment is the best aesthetically as well, representing harmony of function and form. Egg shaped digesters are optimal concerning technology and also structural requirements, therefore aesthetic value is presented by harmony of structure and form as well.

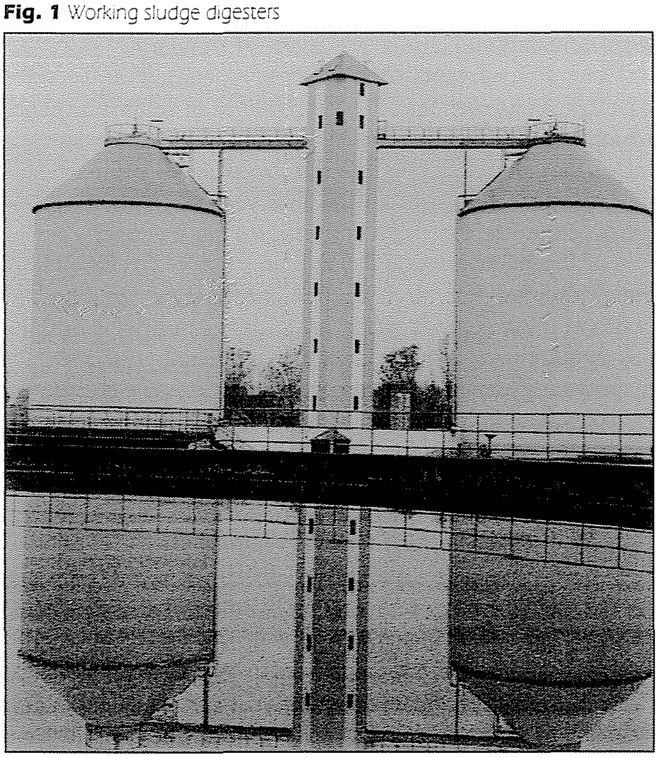
4. LOADS

Liquid storage constructions of great volume are complex spatial structures, force analysis of which should be accomplished by formulation of the bending theory considering the main loads as follows:

- dead load
- load of stored liquid
- constant temperature variation
- non-uniform temperature variation
- wind load
- non-uniform subsidences
- seismic load.

Structural units should be rated for the standard stresses

Fig. 1 Working sludge digesters



derived from the loads detailed above – as a realised situation – in annular and meridian direction as well. Post-tensioning processed in construction period also needs the analysis of temporary stresses with special attention for concrete strength in tensioning period. Introduction of final prestressing forces means linear external loads. Proper sequence of prestressing depends on the inner forces.

Force system of complex shell structure is also influenced by the time history of the prestressing. Definition of effective prestressing force needs analysis of losses originated from friction, type of end anchorage, shrinkage and slow deformation of concrete, creep of tensioning elements. These can not be detailed within the scope of this article.

In the aspects of stresses, in case of digesters constructed from a doubly curved shell structure, foundation method is of essential significance. Foundation method determines the diameter of the circle, along which loads of superstructure are transferred to and the way of impact. The greatest bending moments originate in the region of the supports.

5. WATERTIGHTNESS

Post-tensioned concrete sludge digesters can be prestressed in one or two directions. The solution depends on the geometric properties and the stresses.

In simple cases (e.g.: digesters with cylindrical walls) tensioning only in annular direction is sufficient. Regarding this type of solution, only normal reinforcement works on stresses originating in direction of generatrix. These kind of structural elements are crack clear in annular direction, however in axial direction the structure meets water tightness requirements only according to the conditions of limited crack width.

Evidently watertightness of two directionally prestressed digesters is of essentially higher level, especially in case of full prestressing. According to practice, such stress condition should be created in structure that compressive stress remains in it in case of even the possible greatest tensile forces. This is the designer's decision and responsibility. Analysis of the time history of the effective tensional force loading has essential significance in this problem.

As far as reinforced concrete sludge digesters made of watertight material are concerned, the level of technical solution of some details has special great importance. Solution of permeability prevention at the dilatations proposed needs great care. In addition to designer's work, builder should fulfil completely the requirements of watertight concrete (quality of mixed concrete, conditions of pouring and compacting, avoiding incidental dilatations, appropriate curing of concrete, etc.).

If the conditions outlined previously are completely accomplished on high level, watertightness of tensioned sludge digesters will be generally suitable in practice. Otherwise it can be necessary to apply lining layers in order to develop watertightness, as well.

6. GASTIGHTNESS

Final definition of gastight reinforced concrete structure is not available in the literature so far. Experts according to their responsibility are usually not satisfied with the technical level of the watertight concrete structure, but apply many times residual metal form panels at the upper structural element of the digesters. Application of a properly welded steel structure is correct without doubt and moreover it has some possible advantages considering the crushing of the floating sludge crust and the form-work and scaffold in building period. It can be declared that conditions of gastightness are closer than that of watertightness. In order to prevent gas penetration into the structure, a lining system can be applied in case of crack free surface.

7. FOUNDATION

High specific load effects the subsoil due to the load of the 20 – 40 m high water pressure and the dead load of the structure of sludge digesters. Consequently, designer should take into cautious consideration the justification of the load bearing capacity of soil, in particular because of wastewater treatment plants are usually built in the sunken parts of the settlements, sometimes next to rivers or creeks. In these areas can often be found impermeable soil and high ground water relations. These factors call many times for significant dewatering in building period. Justification of load carrying capacity is not sufficient due to the great specific loads, but significant subsidence should be taken into consideration depending on subsoil parameters. Settlement process drags on frequently for a long time causing many operational problems. In case of shallow foundation methods non-uniform sinking can be taken place because of load intersection or other effects, which can cause overtilting of constructions leading to deterioration of pipe joints.

Due to subsidence or building technology reasons deep foundation can often be found. In certain cases special founding solutions are needed. In particular case the sludge digester as a water-bearing tank is supported by – or set into — a separated foundation structure, which is a properly rigid load translator unit. These kinds of examples can be seen in Figure 3.441 of the Engineer's Manual referred.

8. INTERNATIONAL OVERVIEW

Building of sludge digesters began in the year of 1950 – 60s in the developed countries. Post-tensioned reinforced concrete structures were constructed first of all in order to meet the demand of digesters with large volume. Application of these structures could be realised by developed industrial background (high-tensile steel, modern cradling system, etc.)

Digesters of large volume in Europe are rather shallow tank-like structures in England and France, but in Germany they are perpendicularly stretched. cylindrical, drop and egg shaped. Detailed summary can be seen in the study of H. Bomhard (1979). First the more simple forms have been built in the USA, and recently perpendicularly stretched constructions have appeared perhaps upon European influences. The latter can be found in the Far East countries, as well.

Development of egg shaped digesters has been based on work of German experts. In the respect of realisation of liquid storage basins the DYWIDAG tensioning system has been entirely applied, e.g.: bars and staples set into sleeve pipe. There are numerous references shown by G. ARNOLD (1969). Cradling and scaffold of doubly curved shell structures have been developed continuously. For the first time a digester was built from perpendicular segments by the support of a scaffold, which was spatially stiffened and rotable round a king bolt. Eight sludge digesters with the volume of 6600 cu.m each have been built by the mentioned method in Berlin (U. Finsterwalder and G. Kern in 1963). Later building of annulus units were applied building the units one by one by the help of lighter form work. This means mainly creeping and sometimes sliding form work. These types of structures have been built in Italy, Austria and also Switzerland.

In the Middle-European countries the demand of digesters appeared only in the past decades and mainly in the big cities. Generally it can be stated that the German developments stand out more with their cylindrical – conical solutions. For instance this type of post stressed digesters have been built in Pozsony, as well.

Owing to world-wide trade, well proven technical solutions have entirely spread recently and the trend of development is the building of digesters made of vertically stretched doubly curved shell structure.

9. REINFORCED CONCRETE DIGESTERS IN HUNGARY

Reference of Middle-European countries in the previous chapter is valid for the practice in Hungary, as well. First of all in the end of the 60's and in the beginning of the 70's were sludge digesters built upon the designs and concepts of the experts at MÉLYÉPTERV (Janzó J. and his co-authors, 1972). Significant part of those are of 1000 – 2500 cu.m. volume, consequently the constructions were constructed of normal reinforcement concrete. Having experienced mixed problems, assuring of watertightness caused the difficulties.

In this period there were 4 monolithic reinforced concrete post-tensioned sludge digesters (of 2800 cu.m useful volume each) built in the South-Pest Wastewater Treatment Plant in Budapest by the management of designer Thoma J. In the solution of cone-cylinder-cone the cylindrical walls were prepared by prestressing with MOTALA system. For the exterior surface of the wall there were cement mortar layer applied by cement blower as an anticorrosion coating of the thin high tensile steel wires spooled onto the wall. Constructions are still in operation after a machinery refurbishment, the first structural renovation has begun in 1999.

After an interval of nearle two decades in 1994 a new sludge digester of 2700 cu.m was built, which was constructed from non-prestressed concrete next to the previous one of 2x1500 cu.m in Kecskemét. In 1996 – 1997 a post-tensioned sludge digester was built with the volume of 2x4500 cu.m in Debrecen (see *Fig. 1*).

10. THE MOST UP-TO-DATE POST TENSIONED DIGESTERS

The common section of the post stressed digesters of the largest volume in Hungary, of the interim machine house of sludge treatment process, stair case and maintenance bridges can be seen in *Fig. 2*. About the structures of large bulk, authors as designers wish to describe as follows.

10.1 PROBLEMS OF FOUNDATION

The Wastewater Treatment Plant of Debrecen is located on the lowland next to the creek Tócó. Vertical set up of the structures has been essentially influenced by the high water level.

Soil parameters showed remarkable variety, blocks changed significantly within a little area. After many soil explorations

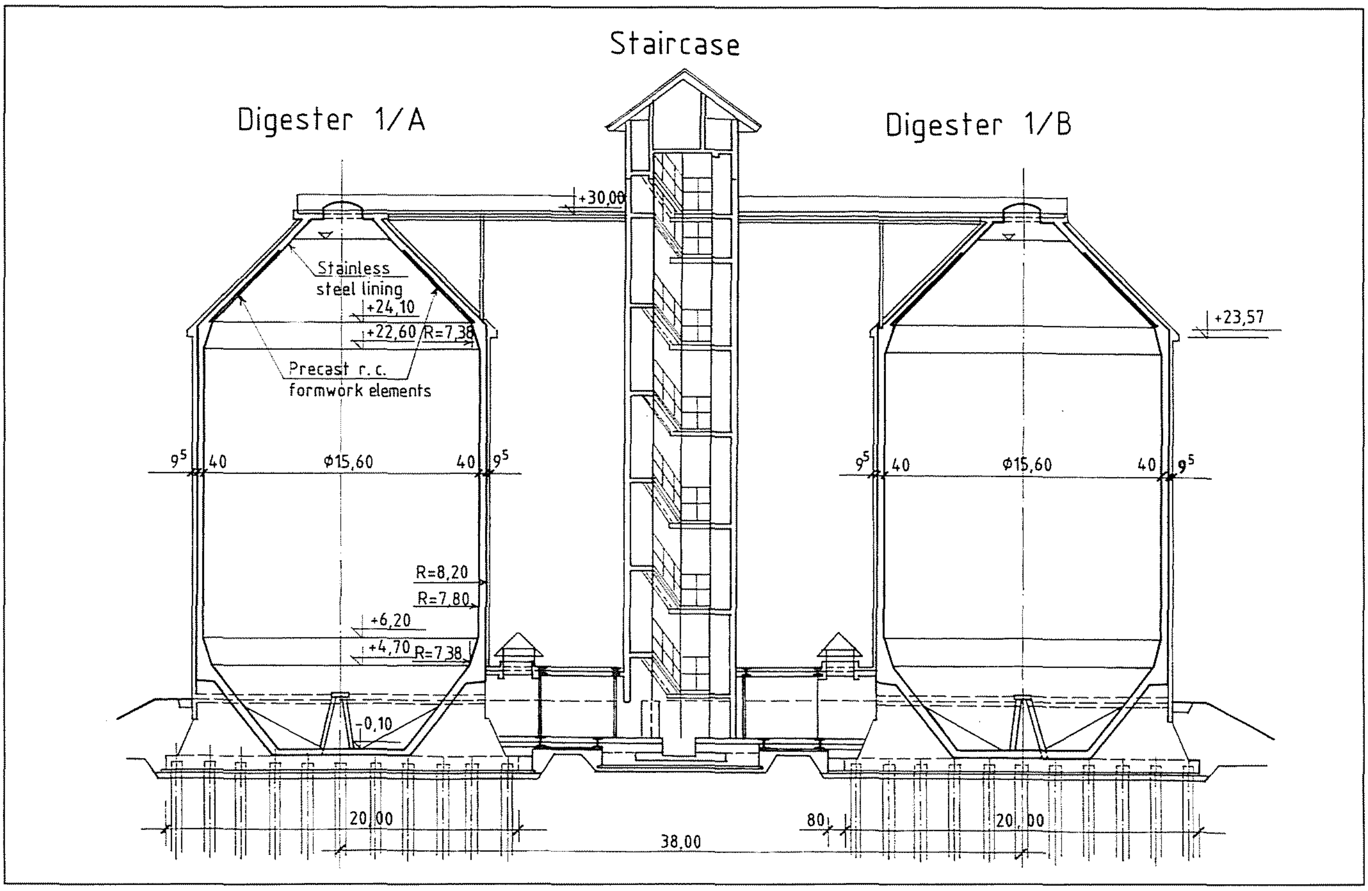


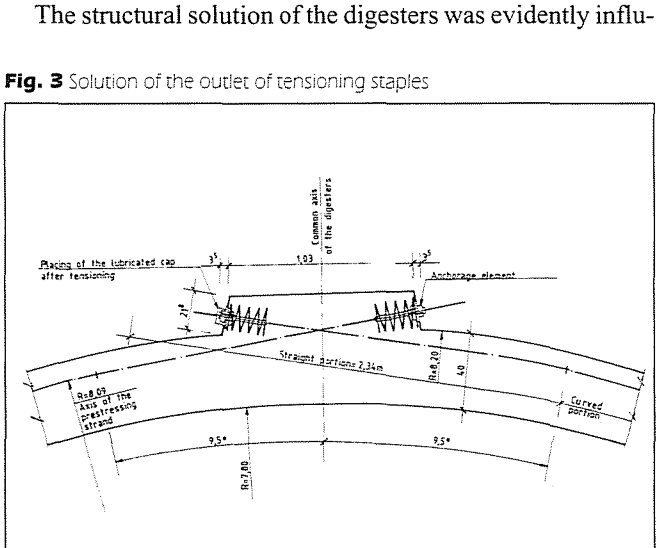
Fig. 2 Perpendicular section of the digesters and stair case

it could be stated, that the possible building area lay at the place of the former bed of creek Tócó, which had been filled. The most suitable set up location should have been found within an area of 20 – 50 m.

In all, miry sand flour, sludge and sand layers were located in various thicknesses under the surface humic top soil of 0,2 – 2,1 thick. Between the mentioned ones, in three different depth, there were interjacent layers of high organic content, which had particularly liability for compression. Sufficient compact soils, which are more suitable for load bearing appeared at about the depth of 20 m. Foundation on piles of this depth could not be taken into consideration for the lack of funds of contractor VEGYÉPSZER Co. The acceptable compromise were a sunk flat foundation, in which Franki type piles of app. 10 m depth were embedded into the 3,0 – 4,0 m thick grey sand soil of medium compactness. The building works were accomplished by the ALTERRA Ltd.

The structural solution of the digesters was evidently influenced fundamentally by the above mentioned circumstances. This extraordinary unfavourable soil character came to light only at the beginning of the construction designing process, and finally a properly rigid reinforced concrete "cup" structure should have been created within very pressing time conditions. The steadiest loading of piles could be assured by this structure, and the concreting of the thick structure of C12 concrete quality could be made without any risk in summertime. In the reinforced concrete "cup", which co-operates with the lower conical part of the water bearing tank, significant annular tension stresses arise.

Fig. 3 Solution of the outlet of tensioning staples



10.2 WATER BEARING TANK

The main sizes and geometric parameters are shown in *Fig. 2*. The vertically stretched form has been reasonable for both structural and technological advantages. The solution of the doubly curved shell structure can not be taken into consideration for the lack of funds of the building contractor.

The way of cradling, namely the building technology conception was a significant professional problem, as well. At least the building firm DÉLÉPSZER Ltd. in common with the main contractor decided to build the cylindrical wall by sliding form work. They undertook obligations of continuous casting, preventing incidental dilatations, building of compact, watertight structure. In such matters designers had many unfavourable experiences, and prestressing in perpendicular direction could not be taken into consideration for cost saving reasons. Though wall thickness of 40 cm was suitable regarding concrete casting conditions, even if prestressing staples had to be placed in addition to the steel reinforcement of two layers.

Building of the top cone had to be made in great elevation in view of the requirements of water- and gastightness. Residual metal form panels border the interior surface of the conic

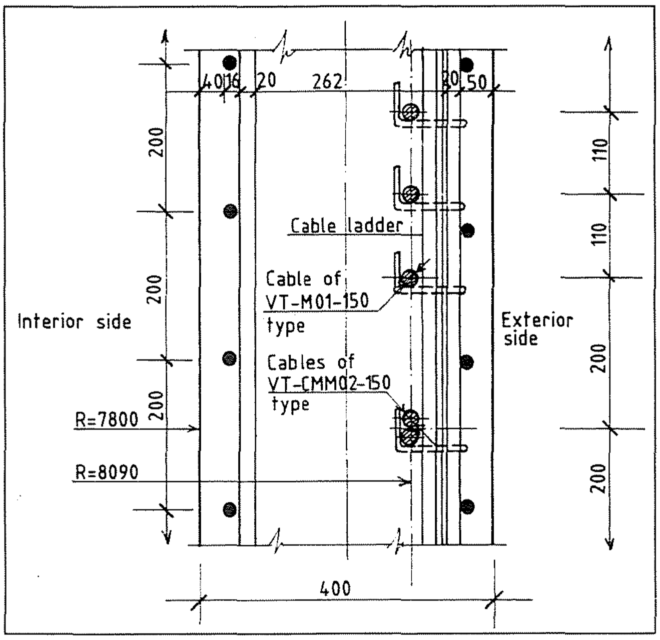


Fig. 4 Reinforcement of cylindrical wall, detail

shell, which were supported by a scaffold made of light weight steel structure. This scaffold was supported by the cylindrical wall. The lower part of the conic shell was created by 10 cm thick prefabricated single boards according to a surface of regular 48 sided pyramid, behaviour of which was that of a membrane shell due to their annular joints in the pouring period. Above these boards a rust proof steel lining of 3 mm constituted the interior surface, which structure as a spatial unit was lifted into its place by crane. This lining was supported by temporary wood boards in the concrete pouring period in order to prevent detrimental deformations. Consequently, a two layered reinforced concrete structure meets watertightness requirement, and a monolithic structure cradled by a continuous steel lining plate meets gastightness demand at the top cone.

10.3 PROBLEMS OF POST-TENSIONING

The annular stress of the cylindrical wall was inevitable. The largest standard tensile force is 1900 kN/m at the lower third of the cylindrical wall. The single and double, so called "slip insert" prestressing staples were provided with plastic lining and were distributed according to the changes of the tensile forces. The supporter was the Austrian VORSPANN – TECHNIK firm, and the prestressing of the St 1570/1770 quality, 150 sq.mm cross sectioned staples were accomplished by MEGALIT Ltd. The necessary end anchors were evidently made by the mentioned system.

In the cylindrical wall the prestressing staples are located at the inner side of the exterior reinforcement along a circle, see in Figure 3 and 4. The outlet of the staples is solved at the 2 so-called "aprons" located tangentially on the common axis of the digesters. By this solution the tensioning of the approx. 50 m long staples could be realised at one place per rings at the two sides of the apron, and also the anchorage of the successive rings at their opposite side. The end anchors were located per 22 cm on the side surface of an apron at the place of the greatest tensile forces, which solution was advantageous considering both static and post-tensioning realisation aspect.

The location of the tensioning staples had to be assured with the prescribed tolerance in order to prevent detrimental losses and bending moments. This was served by the "directors", which determined the place of the staples regarding both floorplan and vertical location. It has a particular importance for the building technology of sliding frame work, according to which the staples had to be continuously drawn-in between the elevating frames of the actual work level and the reinforcement and had to be fixed by binding.

According to designer's decision the digesters are totally prestressed in annular direction and compression remains in concrete cross section even in case of the highest stresses. This is important condition of watertightness.

11. CONCLUSIONS

Experts interested in creating, realisation of reinforced concrete sludge digesters should be aware of the fact that structures are the main elements of a complicated technologic system, foundation of which and adaptation into the natural surroundings call for numerous considerations and a great caution due to their large load. Form of a sludge digesters determines forces being attacked by, and realisation with economic efficiency can be imagine only by thorough grounding in cradling and building technology. Post-tensioning is necessary after the volume limit of 3000 cu.m, and the larger is the volume of a structure the more definitive the demands of force system are, namely the ideal forms are practical.

Sludge digesters of Debrecen justify the fact that there are intellectual capacity and thorough grounded firms available in Hungary, which are able to design, execute in an up-to-date way and operate structures of high level.

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EFFECT OF TEMPERATURE VARIATION AND SHRINKAGE ON CIRCULAR TANKS

Dr. Béla Csíki

Temperature variation and shrinkage of concrete can produce severe stresses in the wall of traditionally reinforced (unprestresed) or prestressed concrete circular tanks. Design codes and recommendations require that these effects be considered in the practical design, but generally do not give sufficient guidance on the methods of analysis or on the distribution and magnitudes of the internal forces to be expected. Closed mathematical formulas are presented applicable for most practical cases and suitable to obtain results for any combination of types of the lower and upper edge supports of the cylinders. Formulas are given for three edge combinations typical in practice. Distribution of internal forces due to temperature variation is illustrated by a numerical example.

Keywords: circular tanks, serviceability, shrinkage, stresses, temperature variation, prestressed tanks

1. INTRODUCTION

When the deformations of a structure are restrained temperature variation and shrinkage of concrete produce stresses. In case of circular cylindrical reinforced concrete tanks with vertical axis of rotation the outer restraints are usually caused by the lower or - in case of roofed tanks - the upper support of the wall. Moreover, temperature (or shrinkage) variation through the wall thickness can cause high stresses, even when the top edge is free and the bottom is supported only against vertical displacement, but can move and rotate freely in radial direction (membrane support).

Stresses due to temperature variation (or shrinkage) produce hoop forces and bending moments in circumferential direction and vertical bending moments along the wall in case the vertical elongation and shortening of the wall is not restrained. These stresses should also be considered in the serviceability design. Cracking is produced when the tensile stress of the wall exceeds the concrete tensile strength or - in case of prestressed tanks - the sum of the absolute value of the tensile strength and the residual compression. The amount and arrangement of reinforcement of *unprestressed tanks* is mainly determined to satisfy crack control. Therefore, the temperature variation and shrinkage effects should be considered in the service load combinations carefully with their proper sign - possibly - not neglecting even the circumstances of the construction to be expected (technology, weather). In case of circumferential *prestressing* after filling the tank remaining of a constant residual annular compressive stress in the wall is usual to counteract tensile stresses due to temperature and shrinkage effects. The value of the residual hoop stress in concrete after prestress losses is commonly taken between 1 and 2 MPa. In their comprehensive study Ghali and Elliott (1992) showed that, this range of residual compression is not enough to compensate the tensile stresses even in not very extreme climates and, the increase of the prestressing stress does not seem to be economic. Moreover, in most practical cases the radial displacement of the bottom edge of the wall is restrained during the prestressing (post-tensioning) process and, as a consequence, circumferential compressive stress cannot be produced in the vicinity of the lower edge (Ghali, Elliott, 1991). Therefore, cracking of prestressed tanks usually should be controlled by provision of nonprestressed reinforcement. When the wall is allowed to slide freely in radial direction during tensioning residual compressive annular stress can be ensured in the wall near the base (Brøndum-Nielsen, 1990, 1998). In such cases the expected temperature variation and shrinkage stresses should also be taken into consideration to have an economic value of the residual hoop stress.

The statements in the preceding paragraph reveal that knowledge of the internal force distribution due to temperature variation and shrinkage of concrete is indispensable to the design of both unprestressed and prestressed circular cylindrical tanks. Methods for considering *uniform and linear* distribution of *temperature variation* through the wall thickness can be found in several monographs (Timoshenko, Woinowsky-Krieger, 1959, Márkus, 1964, 1967). A more general method for considering nonlinear temperature variation over the thickness of the tank is showed in the paper of Ghali and Elliott (1992). Considering linear temperature variation they also present closed form solutions referring to *deep tanks* with three types of base supports and free top edges. However, there can hardly be found a collection of expressions in generalised form applicable in most practical cases and suitable for creating closed form solutions to any combination of types of bottom and top supports. The *aim* of this paper is to present such a set of equations and to show its application. Furthermore, the suitability of the formulas for considering *uniform or varying shrinkage effects* over the wall thickness will be presented.

2. ASSUMPTIONS, MATERIAL MODEL

Usual approximations in the theory of thin shells are considered valid. Moreover, the temperature variation (or shrinkage of concrete) is supposed to be constant along the height of the wall, symmetric to the axis of rotation and linear over the wall thickness. Vertical axis of rotation (symmetry) and constant wall thickness (stiffness) of the tank are also assumed. The elongation or shortening of the wall in the axial direction can be free to occur. The material of the tank is supposed to be *linearly elastic*. This approach strictly applies only before cracking of concrete. In case of totally built in walls at the base - for example - after formation of the first horizontal crack near the base the degree of fixity and the stresses are reduced. If the crack width is controlled by the presence of suitably arranged nonprestressed reinforcement the real stress distribution will tend to be closer to that of a tank hinged at the bottom. Thus, for tanks designed according to the crack width control requirements the results obtained assuming linearly elastic behaviour can be regarded as the upper or lower evaluation of the real stress distribution depending on the idealised type of supports considered in the analysis.

3. SIGN CONVENTION

Sign convention in this paper will be the following. The hoop force in a section is positive when tensile force. Positive bending moments produce tension at the inner face of the wall. The normal displacement of the wall is positive when happens outwards. The elongation and shortening are positive and negative strains, respectively.

4. EFFECT OF TEMPERATURE VARIATION

Assume that, comparing to a starting condition, the temperature variation of a circular cylindrical shell through the wall thickness can be expressed by

$$T(z)=\frac{t_i+t_o}{2}-\frac{t_i-t_o}{2}z=t-\Delta tz \tag{1}$$

where t_i and t_0 are the temperature rises of the inner and outer faces of the wall (*Fig. 1*), while $t=(t_i+t_0)/2$ and $\Delta t=(t_i-t_0)/2$ are characteristic values regarding the uniform and the linearly varying distribution of the temperature variation over the thickness of the wall, respectively.

Geometry of a circular cylindrical shell with radius r, wall thickness h and height l is shown in *Fig. 2*. The radial displacement w of the shell wall in case of thermal load is governed by the following differential equation (Márkus, 1964):

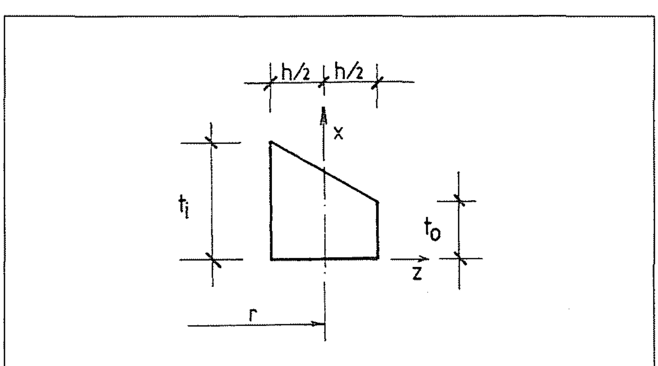
$$\frac{d^4w}{dx^4}+4\beta^4w=4r\beta^4\alpha t \tag{2}$$

where α is the coefficient of thermal expansion and

$$\beta=\sqrt[4]{\frac{3(1-v^2)}{r^2h^2}} \tag{3}$$

is constant, where v is the Poisson's ratio with value 0.166 for uncracked reinforced concrete, which usually approximated in practise by 0.2. Tanks satisfying the condition

Fig. 1 Distribution of temperature variation over the wall thickness



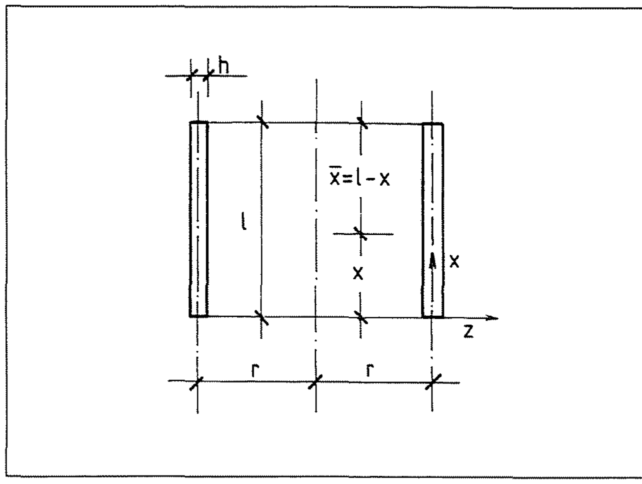


Fig. 2 Geometrical data and coordinate system of analysis

$$l\geq\frac{\pi}{\beta} \tag{4}$$

are referred to as "deep tanks". Substituting the value 0.2 for v the combination of Eqs. (3) and (4) gives the dimensionless parameter characterising deep tanks (Ghali, Elliott, 1992):

$$\frac{l^2}{rh}\geq 5.8 \tag{5}$$

This condition is satisfied in most practical cases. Analysis of these tanks is relatively simple because the strains and stresses due to the lower or upper edge constraints vanish along the wall of the tank and become negligible in the vicinity of the opposite boundary. Thus, the effect of one edge support (boundary) can be analysed independently from that of the other. The present paper addresses the problem of temperature variation (and shrinkage) of such tanks.

As a first step consider the effect of the *lower* edge support of the circular tank with temperature variation. The general solution of Eq. (2) can be expressed by:

$$w=\alpha tr+Ce^{-\beta x}(\cos\beta x+\psi) \tag{6}$$

where the constants C and ψ depend on the boundary conditions (Márkus, 1964). Having the general solution the functions of the rotation and the internal forces of the wall can be determined by applying the following expressions (with the modulus of elasticity E):

$$\vartheta=-\frac{dw}{dx} \tag{7}$$

$$n_\varphi=Eh\left(\frac{w}{r}-\alpha t\right) \tag{8}$$

$$m_x=\frac{Eh^3}{12(1-v^2)}\left[\frac{d^2w}{dx^2}-2(1+v)\alpha\frac{\Delta t}{h}\right] \tag{9}$$

$$m_\varphi=\frac{Eh^3}{12(1-v^2)}\left[v\frac{d^2w}{dx^2}-2(1+v)\alpha\frac{\Delta t}{h}\right] \tag{10}$$

$$q=\frac{Eh^3}{12(1-v^2)}\frac{d^3w}{dx^3} \tag{11}$$

where n_φ, m_x, m_φ and q denotes the hoop force, the vertical bending moment, the circumferential bending moment and the shear force in radial direction, respectively. The derivatives of Eq. (6) included in the previous expressions are as follows:

$$\frac{dw}{dx} = \sqrt{2}\beta Ce^{-\beta x}\sin\left(\beta x + \psi + \frac{\pi}{4}\right) \tag{12}$$

$$\frac{d^2w}{dx^2} = 2\beta^2 Ce^{-\beta x}\sin(\beta x + \psi) \tag{13}$$

$$\frac{d^3w}{dx^3} = -2\sqrt{2}\beta^3 Ce^{-\beta x}\cos\left(\beta x + \psi + \frac{\pi}{4}\right) \tag{14}$$

The three sets of boundary conditions concerning the lower edge ($x = 0$) typical in practice,

fixed edge (built in wall):	$w = 0$,	$\vartheta = 0$
hinged edge:	$w = 0$,	$m_x = 0$
membrane edge:	$m_x = 0$,	$q = 0$

The two integration constants (C, ψ) to each type of boundaries can be determined by solving the actual linear equation system considering $x = 0$ and Eqs. (6) through (14). The expressions of the constants determined separately for both the uniform and the varying distribution of temperature variation are the following.

In case of *uniform* temperature distribution ($\Delta t = 0$),

for fixed edge: $C = -\sqrt{2}\alpha t r$; $\psi = -\frac{\pi}{4}$

for hinged edge: $C = -\alpha t r$, $\psi = 0$

for membrane edge: $C = 0$

In case of *varying* temperature distribution ($t = 0$),

for fixed edge: $C = 0$

for hinged edge: $C = (1+\nu)\alpha\frac{\Delta t}{h\beta^2}$, $\psi = \frac{\pi}{2}$

for membrane edge: $C = \sqrt{2}(1+\nu)\alpha\frac{\Delta t}{h\beta^2}$; $\psi = \frac{\pi}{4}$

(Note: when $C = 0$, ψ can be optional.)

Now, all equations are available to express the static characteristics of a deep circular tank including the effect of temperature variation and also the type of the lower edge support. Only the internal forces will be given here (ignoring the usually negligible shear force) in summarised form, so the expressions include the effect of both the uniform and the non-uniform temperature variation over the thickness of the wall. Introducing the notations

$$\xi_1 = e^{-\beta x}\cos\beta x, \qquad \xi_2 = e^{-\beta x}\sin\beta x,$$
$$\xi_3 = \xi_1 + \xi_2, \qquad \xi_4 = \xi_1 - \xi_2, \tag{15}$$

the formulas of the hoop force and the bending moments will be the following,

in case of *fixed edge*:

$$n_\varphi = -Eh\alpha t\xi_3 \tag{16}$$

$$m_x = \frac{Eh^2}{6}\alpha\left(\frac{rh\beta^2}{1-\nu^2}t\xi_4 - \frac{1}{1-\nu}\Delta t\right) \tag{17}$$

$$m_\varphi = \frac{Eh^2}{6}\alpha\left(\nu\frac{rh\beta^2}{1-\nu^2}t\xi_4 - \frac{1}{1-\nu}\Delta t\right) \tag{18}$$

in case of *hinged edge*:

$$n_\varphi = -E\alpha\left(ht\xi_1 + \frac{1+\nu}{r\beta^2}\Delta t\xi_2\right) \tag{19}$$

$$m_x = \frac{Eh^2}{6}\alpha\left[-\frac{rh\beta^2}{1-\nu^2}t\xi_2 + \frac{1}{1-\nu}\Delta t(\xi_1 - 1)\right] \tag{20}$$

$$m_\varphi = \frac{Eh^2}{6}\alpha\left[-\nu\frac{rh\beta^2}{1-\nu^2}t\xi_2 + \frac{1}{1-\nu}\Delta t(\nu\xi_1 - 1)\right] \tag{21}$$

in case of *membrane edge*:

$$n_\varphi = \frac{E}{r\beta^2}(1+\nu)\alpha\Delta t\xi_4 \tag{22}$$

$$m_x = \frac{Eh^2}{6(1-\nu)}\alpha\Delta t(\xi_3 - 1) \tag{23}$$

$$m_\varphi = \frac{Eh^2}{6(1-\nu)}\alpha\Delta t(\nu\xi_3 - 1) \tag{24}$$

Results for tanks supported along the *upper* edge including also the effect of temperature variation can be easily derived from the previous three sets of equations by the following substitutions:

$$x \to \bar{x} = l - x$$
$$\xi_i(x) \to \bar{\xi}_i(\bar{x}) \qquad i = 1, 2, 3, 4 \tag{25}$$

The distribution of the internal forces along the wall of the tank including the effect of both the *lower* and the *upper* edge supports can be produced by simple summarising. The equivalent constant members of Eqs. (16) through (24) and of equations gained by considering Eqs. (25) – being independent from the boundary conditions – should be considered once only during the summarising process. This way the distribution of the internal forces can be obtained for any combination of the lower and upper boundary conditions of the tank. The *free edge* should be considered by equations referring to the membrane support. A few summarised formulas are showed in the next chapter concerning typical support conditions in practice.

5. RESULTS FOR THREE TYPICAL EDGE COMBINATIONS

Solutions (Ghali, Elliott, 1992) or analysing methods (Márkus, 1964, Márkus, 1967) in the literature usually refer to tanks with free edge at the top. The internal forces of tanks *totally fixed at the bottom, free at the top* (*Fig. 3.a*) due to temperature variation by using Eqs. (16) - (18) and the modified Eqs. (22) - (24) considering (25) are as follows:

$$n_\varphi = -E\alpha\left(ht\xi_3 - \frac{1+\nu}{r\beta^2}\Delta t\bar{\xi}_4\right) \tag{26}$$

$$m_x = \frac{Eh^2}{6}\alpha\left[\frac{rh\beta^2}{1-\nu^2}t\xi_4 - \frac{1}{1-\nu}\Delta t(1 - \bar{\xi}_3)\right] \tag{27}$$

$$m_\varphi = \frac{Eh^2}{6}\alpha\left[\nu\frac{rh\beta^2}{1-\nu^2}t\xi_4 - \frac{1}{1-\nu}\Delta t(1 - \nu\bar{\xi}_3)\right] \tag{28}$$

Tanks *fixed at the base and hinged at the top* (*Fig. 3.b*) are widely used in the civil engineering practice. Eqs. (16) - (18) and the modified Eqs. (19) - (21) considering (25) are used to determine the formulas of the internal forces:

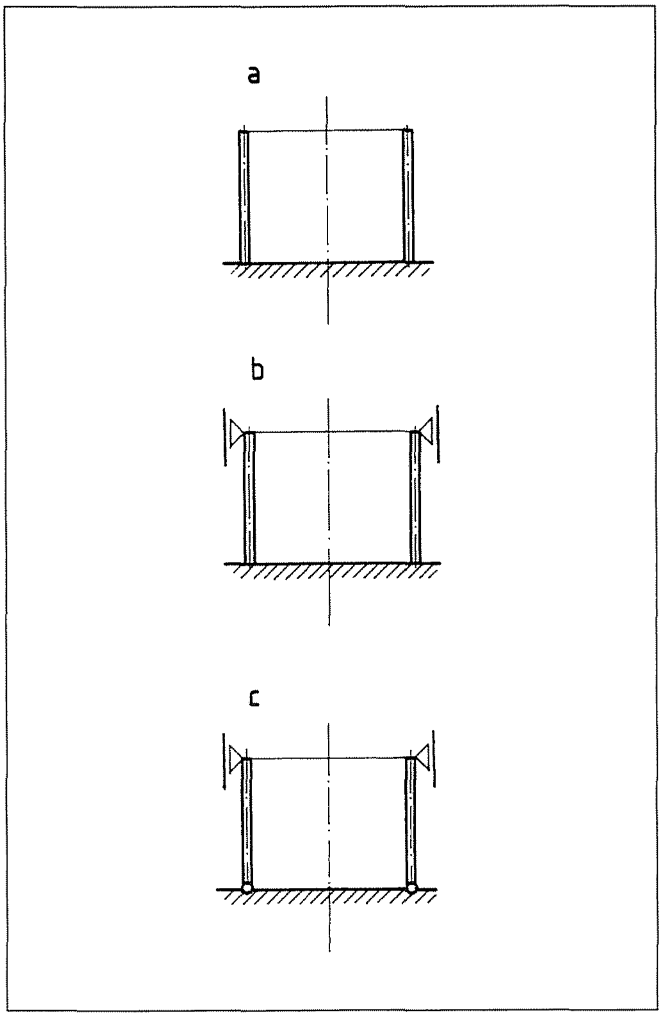


Fig. 3 Typical combinations of edge supports of the wall
a) fixed at the bottom, free at the top
b) fixed at the bottom, hinged at the top
c) hinged at the bottom and at the top

$$n_\varphi = -E\alpha\left[ht(\xi_3+\xi_1)+\frac{1+\nu}{r\beta^2}\Delta t\xi_2\right] \tag{29}$$

$$m_x = \frac{Eh^2}{6}\alpha\left[\frac{rh\beta^2}{1-\nu^2}t(\xi_4-\xi_2)+\frac{1}{1-\nu}\Delta t(\xi_1-1)\right] \tag{30}$$

$$m_\varphi = \frac{Eh^2}{6}\alpha\left[\nu\frac{rh\beta^2}{1-\nu^2}t(\xi_4-\xi_2)+\frac{1}{1-\nu}\Delta t(\nu\xi_1-1)\right] \tag{31}$$

For tanks *hinged at the base and also at the top* (*Fig. 3.c*) the internal forces can be obtained by applying Eqs. (19) - (21) with the necessary modification according to (25):

$$n_\varphi = -E\alpha\left[ht(\xi_1+\bar{\xi}_1)+\frac{1+\nu}{r\beta^2}\Delta t(\xi_2+\bar{\xi}_2)\right] \tag{32}$$

$$m_x = \frac{Eh^2}{6}\alpha\left[-\frac{rh\beta^2}{1-\nu^2}t(\xi_2+\bar{\xi}_2)+\frac{1}{1-\nu}\Delta t(\xi_1+\bar{\xi}_1-1)\right] \tag{33}$$

$$m_\varphi = \frac{Eh^2}{6}\alpha\left[-\nu\frac{rh\beta^2}{1-\nu^2}t(\xi_2+\bar{\xi}_2)+\frac{1}{1-\nu}\Delta t(\nu\xi_1+\nu\bar{\xi}_1-1)\right] \tag{34}$$

Formulas for the remaining six edge combination can be obtained similarly by using Eqs. (16) through (24) considering (25).

6. EFFECT OF SHRINKAGE

Analysis of the effect of shrinkage of concrete is entirely similar to that of the effect of temperature variation (Ghali, Elliott, 1992). But because shrinkage takes a long time to occur, its effect is reduced by creep of concrete. This can be accounted for by using the "age-adjusted" modulus of elasticity of concrete,

$$E_t = \frac{E_0}{1+\varphi_t} \tag{35}$$

where E_0 is the initial elasticity modulus and φ_t is the creep coefficient for concrete.

Besides the age of concrete the value of shrinkage (and creep) influenced also by other effects, such as the relative humidity of the air, concrete mix and wall thickness.

When the relative humidity of the air inside and outside the tank is similar - until first filling for example - the wall is subjected to *constant shrinkage* (ε_{cs}) depending on the time passed from construction and the factors mentioned above. Stresses occur in the wall because the edge supports restrain the free deformations.

After filling the tank the inner surface of the wall is in contact with water for example, and the outer surface often exposed to extreme meteorological effects due to the sun and wind. Thus, the shrinkages at the inner and outer surfaces ($\varepsilon_{cs,\,i}$ and $\varepsilon_{cs,\,o}$) are different and considered from the beginning of the effect causing the inequality (time of filling).

Assuming that distribution of shrinkage strains uniform or linear over the wall thickness and introducing values of "equivalent temperature variation"

$$t = \frac{\varepsilon_{cs,i}+\varepsilon_{cs,o}}{2\alpha} = \frac{\varepsilon_{cs}}{\alpha} \tag{36}$$

$$\Delta t = \frac{\varepsilon_{cs,i}-\varepsilon_{cs,o}}{2\alpha} = \frac{\Delta\varepsilon_{cs}}{\alpha} \tag{37}$$

the analysis of shrinkage effect becomes the same procedure already presented for the effect of temperature variation.

7. NUMERICAL EXAMPLE

Determine the internal forces of a cylindrical tank totally fixed at the base and hinged at the top (Fig. 3./b) due to linear temperature variation over the thickness. The geometrical data are $r = 30$ m, $l = 10$ m, $h = 0.25$ m. Temperature variation of the inner and outer surfaces $t_i = 0°$ *C* and t_0=–30° *C*, thus $t_i = -15°$ *C* and $\Delta t = +15°$. Coefficient of thermal expansion $\alpha = 10^{-5}$ 1/° *C*, modulus of elasticity of concrete $E = 32\,000$ MPa, Poisson's ratio $\nu = 1/6$.

The dimensionless parameter according to Eq. (5) is $l^2/(rh) = 13.3 \geq 5.8$, thus, the tank can be classified as deep and Eqs. (29) through (31) apply, giving the values of the internal forces. The values and distribution along the wall of the hoop forces and bending moments are shown in *Fig. 4*. A tank of the same geometrical parameters is analised in the paper of Ghali and Elliott (1992), but with free edge at the top and with temperature variation of $t_i = 0°$ *C* and $t_0 = +30°$ *C*. Comparing the opposite values of their solution to the results presented in *Fig. 4* one can find that the application of the upper hinged support hardly influences the values and distribution of the bending moments, but increases the hoop forces by almost 50% in the vicinity of the upper edge.

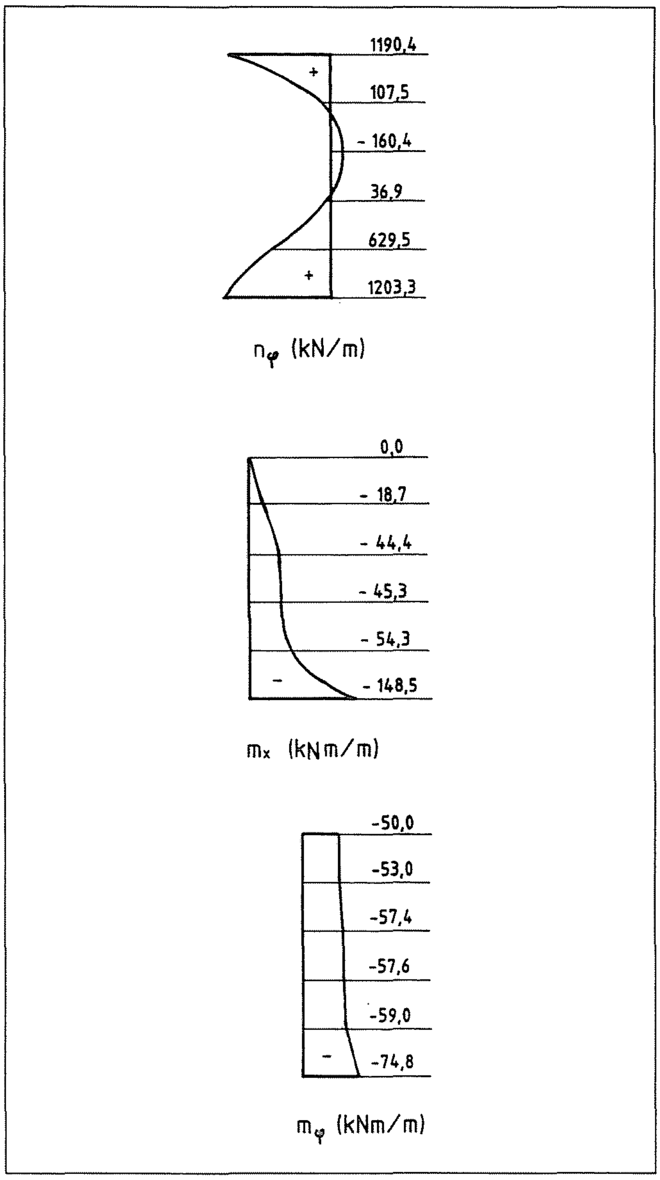


Fig. 4 Internal forces of a circular tank fixed at the bottom, hinged at the top ($t_i = 0\,°C, t_o = -30\,°C$)

8. CONCLUSIONS

Most circular cylindrical tanks in the civil engineering practice are "deep tanks" satisfying Eq. (5) concerning their geometrical parameters. A method was presented for determining the internal forces of such tanks subjected to *temperature variation*. Closed form solutions for the internal forces can be expressed by Eq. (16) through (24) considering also Eq. (25) for any combination of types of the lower and upper edge supports. Formulas for three edge combinations typical in practice are presented in the paper.

Direct applicability of the results for determining the internal forces due to *shrinkage* of concrete is also shown in the paper.

The distribution of the internal forces along the wall of the tank is illustrated by a numerical example. Remarkable, that an average temperature loss of the wall, such as also shrinkage of concrete, cause high tensile hoop forces in the vicinity of the supports, just in the zone where residual compression can hardly be produced even by circumferential prestressing. This fact and the high values of bending moments prove that during the design of nonprestressed reinforcement to satisfy crack control the internal forces due to temperature variation and shrinkage of concrete should not be ignored.

9. NOTATION

h	wall thickness (m)
l	wall height of tank (m)
m_x	vertical bending moment (kNm/m)
m_φ	circumferential bending moment (kNm/m)
n_φ	hoop force (kN/m)
q	shear force (kN/m)
r	radius (m)
t_i, t_0	temperature variation of the inner and outer surface of the wall (°C)
$t, \Delta t$	constants of uniform and linear temperature variation (°C)
$x, \bar{x}$	abscissas along the wall (m)
z	abscissa in radial direction (-)
w	radial displacement (m)
C	integration constant (m)
E	modulus of elasticity (MPa)
E_t	age-adjusted elasticity modulus of concrete (MPa)
α	coefficient of thermal expansion (1/°C)
β	parameter (1/m)
$\varepsilon_{cs,i}, \varepsilon_{cs,o}$	shrinkage at the inner and outer surface of the wall (-)
$\varepsilon_{cs}, \Delta\varepsilon_{cs}$	constants of uniform and linear shrinkage (-)
ν	Poisson's ratio (-)
ϑ	rotation of the wall (-)
$\xi, \bar{\xi}$	functions (-)
ψ	integration constant (-)

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FLYOVER IN DEBRECEN CONSTRUCTED BY INCREMENTAL LAUNCHING

Péter Wellner

The existing bridge near the inner city of Debrecen was unable to cope with the road traffic without serious congestion. It was therefore decided to construct a new bridge next to it. The location of the piers for the new bridge were determined by those of the existing bridge. One part of the structure spanned over nine railway tracks, the other part over the main road No. 4. There was only a very short construction time available and the local conditions were complicated such that two different technologies were required for construction. The superstructure of the bridge was designed as a prestressed concrete box girder. The construction method over the railway segment was incremental launching while the remaining part was assembled on scaffold. The span over the main road had to be built at a higher level and afterwards lowered to its ultimate position.

Finally, the completed structure is a continuous post-tensioned bridge of eight spans, joining to an access branch having a similar structure with two spans. Expansion joints are at the three abutments of the structure. The design procedure started at the end of August 1997 and was prepared by the Technical Department of Hídépítő Co. and STABIL-PLAN Ltd. The execution began in early December 1997 with demolition by explosion of the old branch to Wesselényi street. The bridge was put into operation in October 1998. The overall project duration for the complete implementation was slightly more than 13 months. The formal limits of this paper allow only for a general introduction and discussion with respect to particular details of the design and the construction technology.

Keywords: bridge, design, incremental launching, deck, launching nose, cast-in-situ, post-tensioning, box girder, span, organisation

1. INTRODUCTION

The aim of this paper is to illustrate the benefits of the technical solution and thinking behind the execution. While presenting the implementation I will generally consider all factors which influenced the final result. An explanation describing actual circumstances will make it easier to appreciate the final technical solution.

1.1 THE OLD STRUCTURE

The multi-span reinforced concrete flyover of road No.47 in Debrecen crosses the main road No. 4 and Hungarian State Budapest-Nyíregyháza rail line near the railway station built in 1970. This single-cell monolithic reinforced concrete structure was built on scaffold. The section spanning the railway line was post-tensioned. The two parts were connected to each other by hinged joints. Two access branches were connected to the straight bridge in the direction of Wesselényi street.

1.2 DETERMINATION OF THE TASK

At the time of the construction of the original structure it seemed it would be necessary to build another bridge as soon as the increase of the traffic gave reason for it. For this purpose the foundations of a future bridge were constructed between the tracks.

By 1990, the increased traffic made it necessary to consider the construction of the second bridge. However, it took a good many years to complete the project financing. As the result of the endeavours in this respect made by the Hajdú-Bihar County Road Management Directorate and the Community of the City, the project was started in 1997 with some financial help from the budget. Tenders were invited according to the Procurement Act.

The contracts for design and construction were awarded to Hídépítő Co. and the work commenced at the end of August 1997. However, the long preparatory and decision phase did not modify the deadline for completion which remained October 1998. The construction started at the beginning of November 1997.

Naturally, over the preceding 27 years the circumstances had changed a lot. The traffic over the nine tracks of the railway had significantly increased. Eight of the tracks had been electrified. Additionally, the demands of the road traffic meant that the road cross-section had been widened. The girder of the completed pile-foundations were not long enough for the wide bridge so they had to be lengthened. The existing part was not useless, but its advantage turned out to be far less than anticipated.

According to the tender documentation the requirements increased compared to the original scheme. On the bridge section in the direction of Mikepércs on the Nyíregyháza side, a 4.60 m wide carriageway and cycle track. On the opposite side (towards Budapest) a 0.565 m wide kerb had to be built beside the two 4.25 m carriageways.

Also on the bridge section in the direction of the city centre, the two 4.25 m wide road and the 0.565 m wide kerb is continued (*Fig.1*). On the access branch towards Wesselényi street there is a 5.5 m wide one-way road, a 4.60 m wide carriageway together with a cycle track.

The same layout of the old and new structures was a natural demand and between the tracks it could not be different. The aesthetic similarity between the old and new structures was also taken into consideration in the design.

2. THE DESIGN

The local circumstances, the short time limit and well-founded but severe formal requirements made the design task unusual.

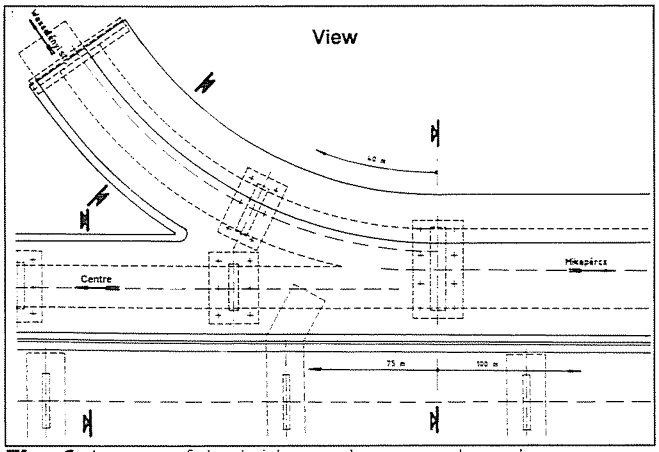


Fig. 1 Layout of the bridge at the access branch

However, it was an advantage that the design and the construction were both the responsibility of the contractor. A design company independent of the constructor would be unable to design a suitable structure in advance and with a technology that could be executed universally. It is another matter, however, that the investor neglected to consider the reasonable time demand of the design compared to the short time limit.

A strong computer background was needed for the design works. Initially, there was a complicated system of statics where different parts of the structure were to be built at different times while passing through different construction phases. Upon completion, they formed a single structure, and, being post-tensioned, it was impossible to complete the design without employing computer software.

Furthermore, the natural demands of redesign and of subsequent correction together with the possibility of producing drawings in repetitive series, justified the use of computer technology.

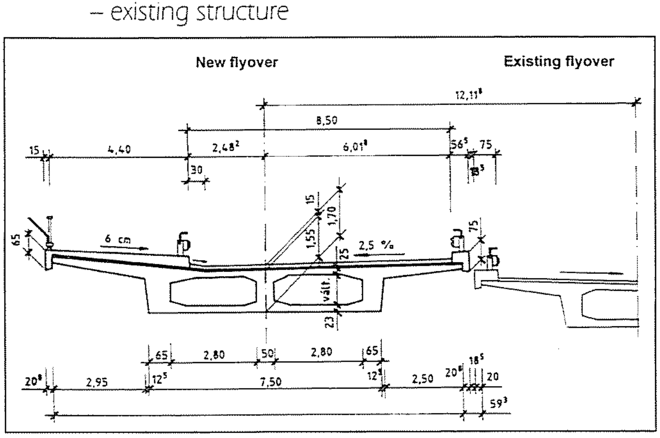
Eventually, the general arrangement plans, setting-out plans, cable layouts and most of the reinforcement designs were produced using computer software. Most of the statical calculations were also executed using the computer, generally assisted by aim-orientated software bought from foreign companies.

Unique drawings not needed for further design and those for which there was not enough computer capacity were produced using more traditional methods.

3. THE NEW BRIDGE DESIGN DATA

The superstructure of the bridge is a prestressed box girder design. Above the railway line it has two cells (*Fig.2*). The access branch (*Fig.4*) and the part above main road No. 4 (*Fig.3*) has one cell.

Fig. 2 Transverse section over the tracks planned structure – existing structure



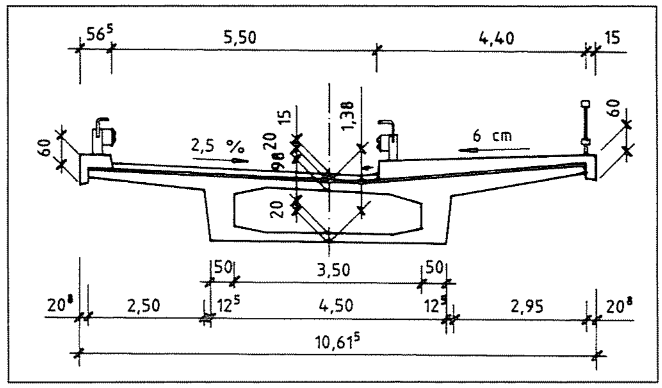


Fig. 3 Transverse section under the access branch

The foundations consist of ∅830 mm drilled piles. The calculated load capacity of each pile is 1418 kN. The loading test proved it with a slightly higher value.

The pier-walls are 0.7 m thick solid structures. They are similar in shape and surface to the existing bridge.

The cross-beams have a unique form. Their dimensions were determined after taking into consideration the technological factors. We had to ensure enough room for the final bearings and the slipping trestles used during construction and also for the lifting jacks and for the lateral guides required. Consideration was also made for the possible change of bearings in the future. We did not and will not need any lifting scaffold for all these.

The longitudinal profile of the bridge had to be formed in order to satisfy the requirements of the chosen technology. For this purpose both horizontally and vertically the road axis had to be straight or a clear arc of circle. As a result the axis of the bridge is straight horizontally and has a summit curve of 2000m vertically.

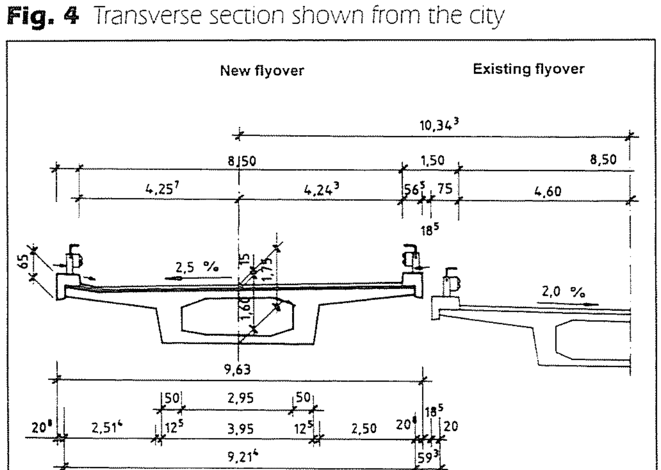
The spans of the bridge are 12.5 – 21.5 – 18.0 – 21.0 – 25.4 – 33.0 – 25.4 – 16.0 m looking from the direction of the city centre. The total length is 172.8 m.

The access branch towards Wesselényi street has two spans of 18.0 and 20.3 m with a total length of 38.3 m.

4. THE BUILDING TECHNOLOGY

In the case of the Homokkert flyover in Debrecen we had to find suitable building methods and structural design. We were expected to disturb the railway and road traffic as little as possible during construction. This is in everyone's interest be-

Fig. 4 Transverse section shown from the city



cause disruptions to traffic flow affect all parties and thus becomes costly for both the railway company and contractor (*Figs.5 and 6*).

This led us to the post-tensioned reinforced concrete structure solution with the incremental launching method. One of its main advantages was that the construction area was independent of the railway tracks. Here we constructed a bridge-section using steel formwork which could be adjusted phase by phase. Because of the heterogeneous but necessary order of spans, the length of the sections varied between 10 m and 16 m. All building phases were concentrated in the construction area, so that it could be accessed easily and mechanised expediently. Quality control also benefited from this arrangement. In any case, the process was more efficient than if it had been carried out above the railway tracks along the length of the bridge.

When the new bridge section reached the required strength we prestressed it to the completed part of the bridge with high-strength prestressing tendons. The lengthened bridge was pushed forward and supported under its ribs with hydraulic jacks supported by gear racks. The bridge travelled from pier to pier this way, until it reached the abutment on the other side.

In order not to have over-long concrete cantilevers we assembled a temporary steel nose on the front of the bridge structure (*Photo 1*). As a result we achieved only a short reinforced concrete cantilever from one pier towards the next as the steel nose was supported by the next pier. Therefore, the stresses became easy to handle.

The launching was executed by two jacks, each with a capacity of 1000 kN. Teflon sheets with 3% friction coefficient were used at the supports and on the construction deck to make movement of the structure possible. The maximum time needed to launch one segment was 3.5 hours.

Each segment was constructed in two phases. The first phase consisted of the bottom slab and the ribs. Their inner steel shuttering was easy to place and remove from above. The shutter of the top slab between the webs was a permanent thin reinforced concrete slab. The outer shuttering was the steel formwork of the construction deck itself.

The reinforcement was lifted into place by a tower crane in several precast units. This tower crane could serve the entire construction area.

The time needed to construct, prestress and push one segment was one week. We prestressed the segments when the concrete was 2.5-3 days old and the concrete reached a cube strength of 26 N/mm^2. New pier under construction next to the existing flyover and the completed flyover are shown in *Photos 2 and 3*, repectively.

Hídépítő Co. has built 18 structures in 14 locations using this technology in the past nine years. The total bridge area of these structures is 36,000 m^2 and their total length is 3,700 m.

5. BUILDING ORGANISATION

Our first concept was to make the construction deck on the Mikepércs end of the bridge behind the abutment. The total straight element could have been launched from this location. The 38 m long access branch, built on scaffold, would have been connected to the 174.6 m long straight bridge.

Unfortunately, we could not use this area because the marking out of the working areas had not been fully considered. We therefore had to locate the construction area on the other side of the tracks and to push the bridge from this side above the railway. However, with this solution we lost the advantage of building the straight part continuously. If we had had enough time, we could have turned the construction deck back, and continued this technology towards the city centre as well. As this would have taken one month, it would have been too much for our time limit. We could not extend the time set down in the contract. The conclusion was to build the remainder on scaffold. This way we could proceed collaterally. On the other hand, this caused some extra cost.

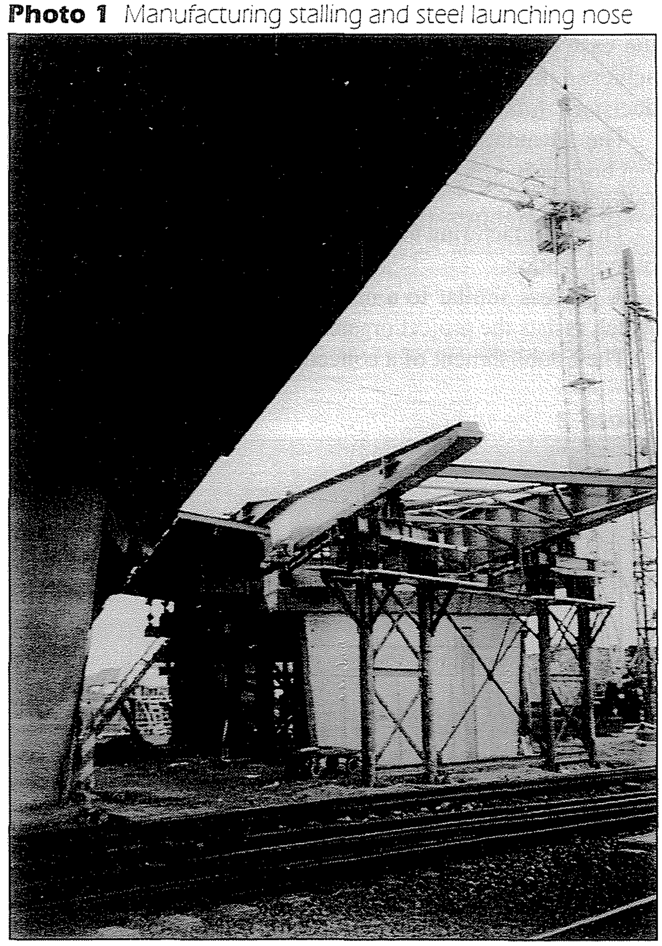
However, this solution disrupted the traffic of road No. 4 and the scaffold structure would have imposed a height limitation for the traffic. Furthermore, there was no way to guarantee that high vehicles would not cause great damage if they missed or ignored the temporary road signs. We did not find any solution to ensure that the above would not happen or the possibility to re-route the traffic.

To avoid the temporary height limitation we decided to build the structure of the span above road No.4 at a higher level, and, after prestressing, lower to its final level. On both sides post-tensioned monolithic reinforced concrete structures connect to this span.

6. CONCLUSIONS

During the design and construction of the structure reviewed we able to appreciate the importance of achieving a balance and harmony between the structural design and the execution technology of the bridge where one element is not more important than the other. The technology must be adapted to the physical possibilities afforded by the site. The structure must be favourable in every respect but the technology and the organisation are factors with similar importance and they must eventually be considered together and as a whole. Though in

Photo 1 Manufacturing stalling and steel launching nose



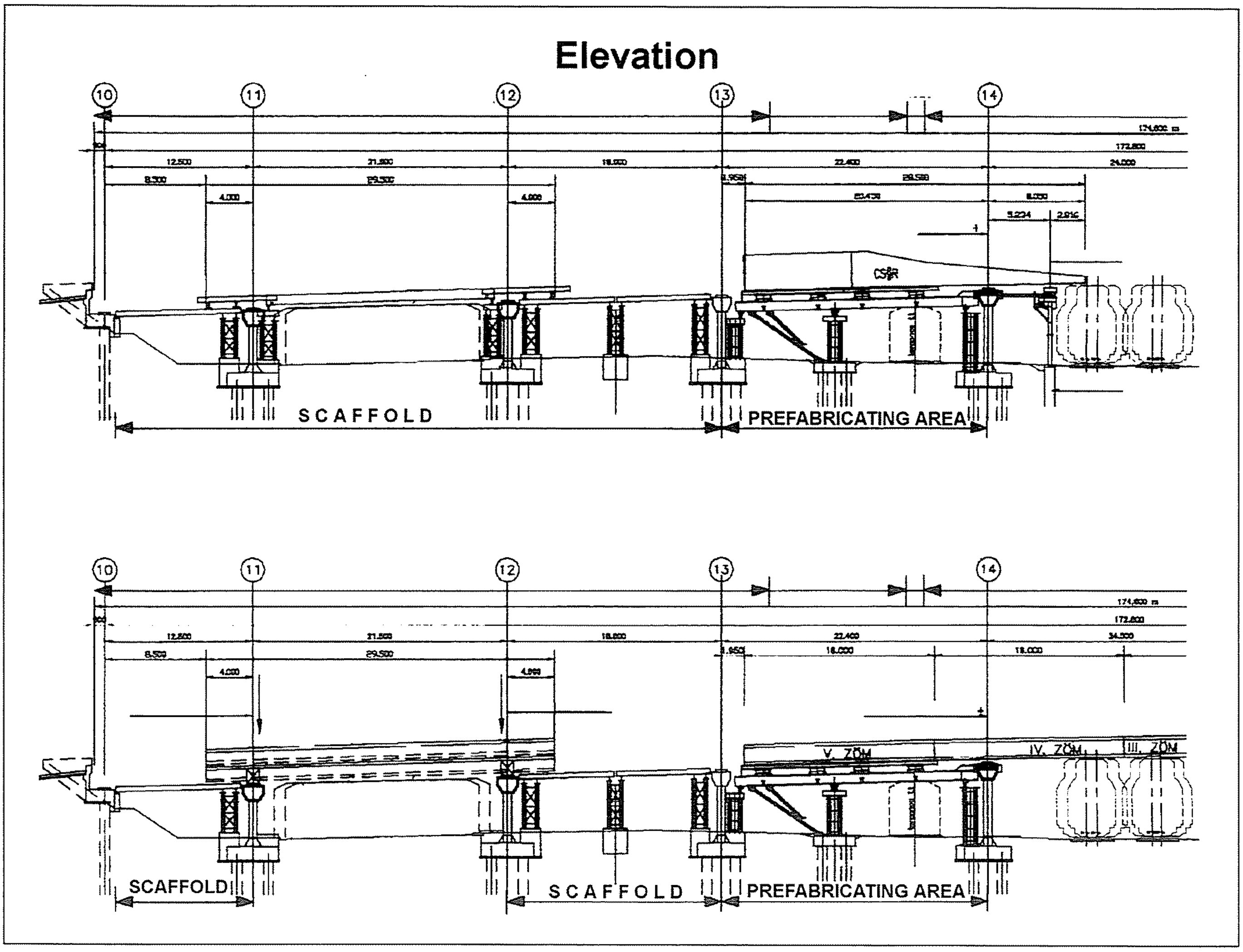


Fig. 5 Structure built on scaffold and the beginning of incremental launching

the case of the flyover described the balance could not be achieved perfectly, the end result shows that our choices and decisions through the process proved well founded.

The advantages of the incremental launching technology can be clearly seen in this structure. These advantages are the following:

The manufacturing the bridge segments can be in a concentrated location.

A process similar to a manufacturing activity is possible which assists the project organisation.

The establishment of a concentrated building site is an effective way of minimising environmental impacts. It is especially true in the case of delicate environment where protection takes a high priority.

The superstructure has no practical affect on the traffic flows of the spanned road or railway track. This advantage is especially great in the case of bridges over rail lines.

The completed construction has certain characteristics with the homogenous structure being notable. By using the prestressing correctly every point of the structure is always free of tensile stresses. This assists in making the structure durable and resistant to corrosion.

Photo 2 The completed flyover

Photo 3 New pier under construction next to the existing flyover

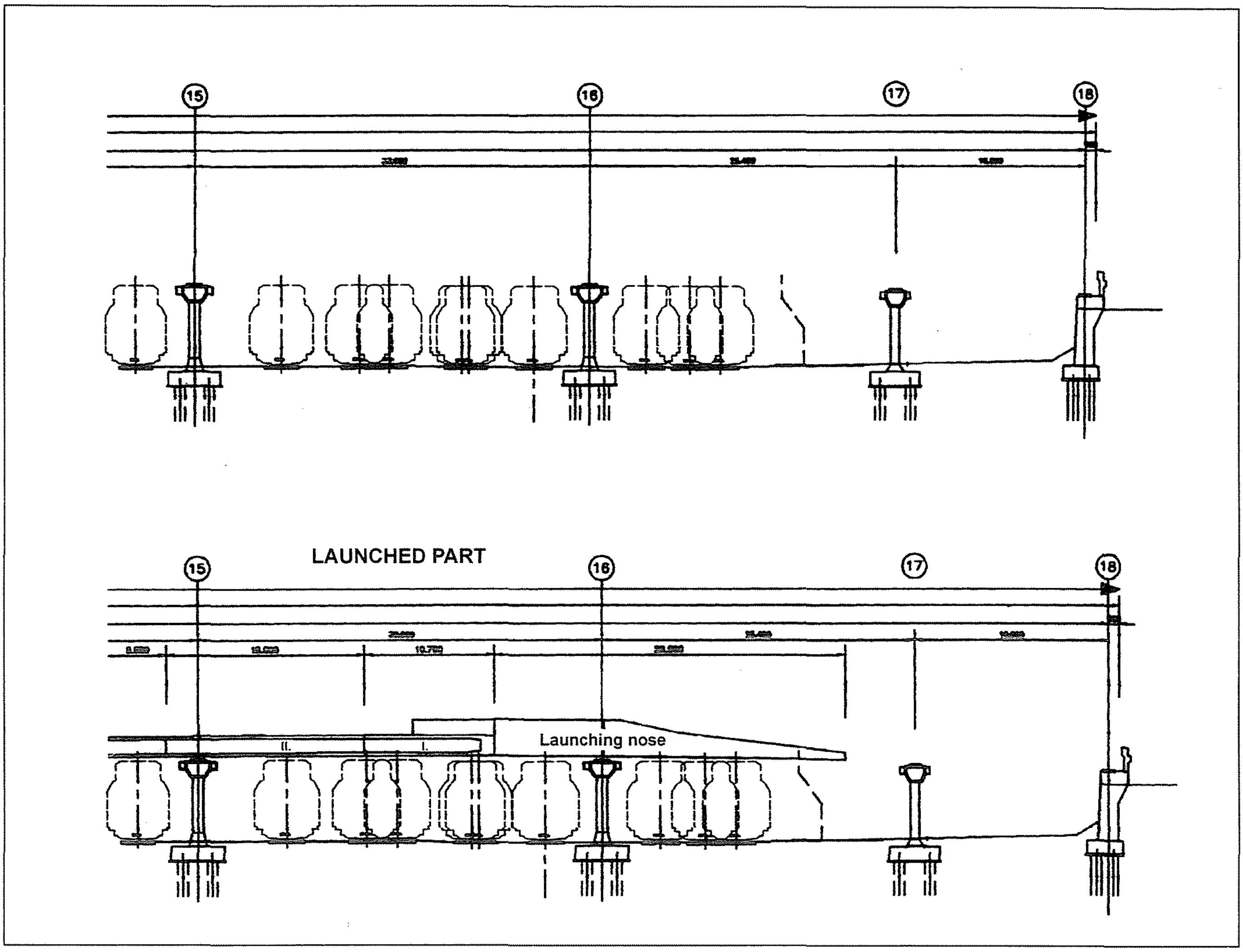


Fig. 6 Construction phases over the railway

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Péter WELLNER (1933), M. Eng. is Head of Department at Hídépítő Co. 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 with 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.

CONSTRUCTION OF VIADUCTS ON THE HUNGARIAN-SLOVENIAN RAILWAY LINE

PREPARATION OF PROJECT

József Vörös

Two viaducts are under construction on the new railway line between Hungary and Slovenia. The longer viaduct (which is 1400 m long) will be one of the longest railway viaducts in Middle-Europe of prestressed concrete. Owing to its size and construction method it gives an excellent engineering task.

Keywords: railway bridge, viaduct, prestressed concrete bridge, tender

1. HISTORICAL REVIEW

Looking back on the history of the 150-year-old railway construction industry we can see that the appearance of railways had a great influence on the development of engineering. The construction of railway lines stimulated the birth of new branches of industry and the modernisation of existing ones. A revolutionary change occurred; first of all, in mechanical engineering followed by significant developments in civil engineering. New bridge-building technology was investigated by the demands of production. Through an emerging need to bridge large spans as well as increasing railway loads – compared with the earlier highway loads – our predecessors had to solve unfamiliar static problems using new materials and new structures at the same time. Due to their knowledge, courage and foresight their work is held in great honour and respect. Let us reflect on the kind of development that occurred in track construction, railway vehicle production of and safety over the last 150 years.

Many Hungarian bridges were constructed along with the railway lines and have been operating for 100-120 years. What foresight, precision and quality was necessary for these structures to meet today's load-bearing rail requirements. We must also bear in mind that these constructions were breaking new technological ground at the same time.

Reflecting on the 150-year history of Hungarian railway bridge construction, three characteristic periods can be distinguished. Each period spans approximately 50 years.

Its essential events are summarised in *Table 1.*

Looking back on the history of railway bridge construction, we can see that the Hungarian bridge constructors always kept pace with economic development.

Collaboration with international organisations formed the basis for the Hungarian Railway Co. (MÁV) to apply the new technical solutions for railway bridge construction in such a short time.

In recent years further new constructional solutions have been applied to the construction of railway bridges. The new Hungarian-Slovenian railway connection created an opportunity for using new building materials and technologies which have not been applied previously at MÁV, but have already been successfully tested abroad.

Have we arrived at the fourth period? Let us leave that decision to posterity.

2. INTRODUCTION

Hungary has direct railway connections with all bordering countries with the exception Slovenia. Railway traffic presently operates connecting the two countries only via Croatia. An agreement was concluded between the two neighbouring states, Hungary and Slovenia, for restoring the previous railway connection.

The planned railway-line section is part of the International Railway Traffic line from Trieste/ Koper – Ljubjana – Budapest – Ushhorod – Lvov – defined as corridor No. 5 in the Pan- European Traffic Conference held in Crete in 1994. The construction is therefore in accordance with the international requirements.

The construction of this railway line has helped to retain local habitants in the region, to develop tourism while also improving international railway connections. The Zalalövő– Bajánsenye corridor passes throughout very difficult terrain in the Hungarian region. The design speed of 160 km/h and the difficult terrain was a significant challenge for the constructors, forcing them to use special solutions in execution.

Table 1 The last 150 years of Hungarian railway bridge construction

First period 1846–1900	– First riveted steel bridge across the Tisza river	1858
Second period 1900–1945	– Design of a trough bridge with rail beams (Ignác Spitzer)	1904
	– Design of concrete railway culverts	
	– Design of concrete arches (Győző Mihalich)	1907
	– Viaduct at Fogaras – Brasso railway line	1912
	max. span 60 m. (Szilárd Zielinski)	1908
	– Concrete arch bridge with 3 hinges (Béla Jámbor)	1913
	– Design of trough bridges	1941
Third period	– Construction of the Danube-branch bridge at Ráckeve	1949
	Langer type reinforced concrete arch bridge (Hugó Székely)	
	– Construction of post-tensioned reinforced concrete bridge with cell box girder (Frigyes Schüller)	1966
	– Construction of a railway bridge with application of prefabricated prestressed beams (Ádám Kemény)	1982
	– Construction of lengthwise sliced prestressed reinforced concrete bridge (László Kassai –Dénes Dalmy)	1994

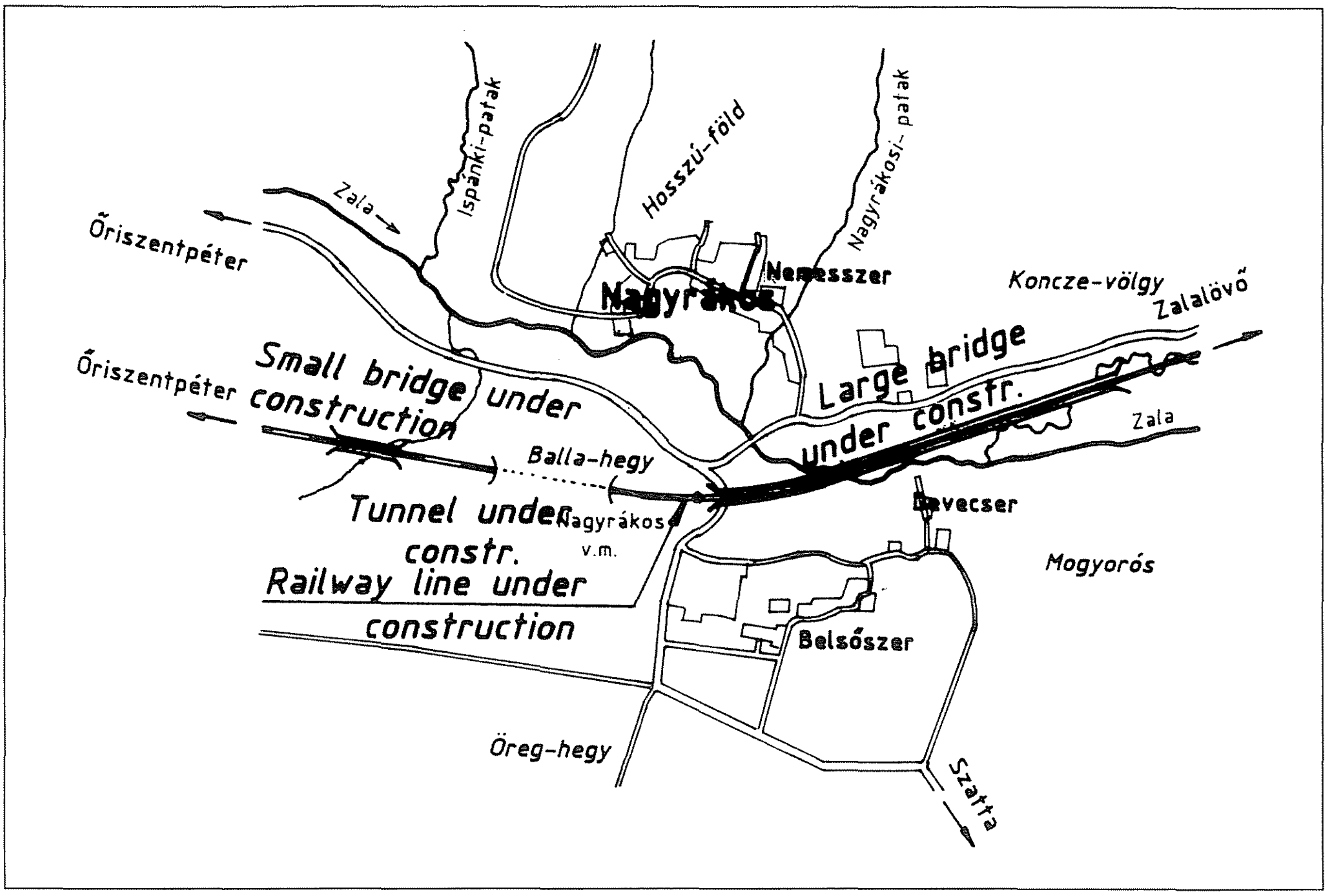


Fig. 1 Site plan of the railway line under construction

Considering the lack of experience in constructing these particular structures in Hungary, a team of engineers was required for selecting, planning and constructing the best solution from all points of view.

The preparatory work for construction started in 1997. According to the treaty, the date of completion was set as the 31st December 2000. The whole expense of the investment is approximately 23 billion HUF, of which 3.5 billion is part of the Budget, and the rest is on credit.

3. REVIEW OF THE LOCATION

The railway line runs from Zalalövő to Nagyrákos with typical level line extension in the 200-300 m wide valley of the upper section of the Zala River. The valley is filled with young deposit. Generally the line passes through deeply waterlogged, moory, dank, meadowy or wooded areas on the gently sloped northern side of the Zala River. The ground level at Zalalövő is at 200-210 m (above the Baltic Sea), and the new railway line runs on a smaller embankment and in a cutting. The planned height of Őriszentpéter station is 260 m above the Baltic Sea level. The almost 50 m difference in height should be gained on a length of 4 km. In this segment the planned longitudinal section has a slope of 11-12 ‰.

In the section near Balla hill a high embankment of 12-17 m is required because of the previously mentioned bad subsoil, and this extensive consolidation would take a long time. Due to the high embankment the distance between the opposite edges of the slope at the fill toe would be 50-80 m. For this reason it is necessary to construct viaducts in this section. In the 200 m long section after Balla hill, the construction of another very high embankment would also be unavoidable. The construction of a viaduct is necessitated at this point because of the potential danger of the erosion and displacement of the steep slope. In the new section it is required to construct a further 50 reinforced concrete structures besides the viaducts (*Fig.1*).

4. FEASIBILITY STUDIES FOR THE CONSTRUCTION OF THE BRIDGES

The choice of the type of structure for the minor bridges did not cause any difficulties at planning stage because many similar structures have already been built in the network of MÁV. However, the situation was different in the case of the two viaducts.

Bridges of that length have only been built in Hungary in relation to reconstruction of existing structures (mostly after the Second World War and generally with the re-use of the existing substructure). As a result of the geometrical restrictions concurrent with available materials and technologies, the large bridges had been built exclusively as trussed structures with open bridge floors.

Obviously such a structural form would not meet today's requirements, either from the viewpoint of aesthetics or execution or with respect to the operation-maintenance costs. In order to choose the most advantageous structure a comprehensive survey has been carried out, firstly in the field of domestic road bridge construction and secondly at other European railway companies.

With the help of ERRI we were able to study bridges constructed within the network of German (DB), Romanian (SNCFR), Swiss (SBB) and Polish (PKP) railway companies.

On the basis of the given technical specifications, bridges

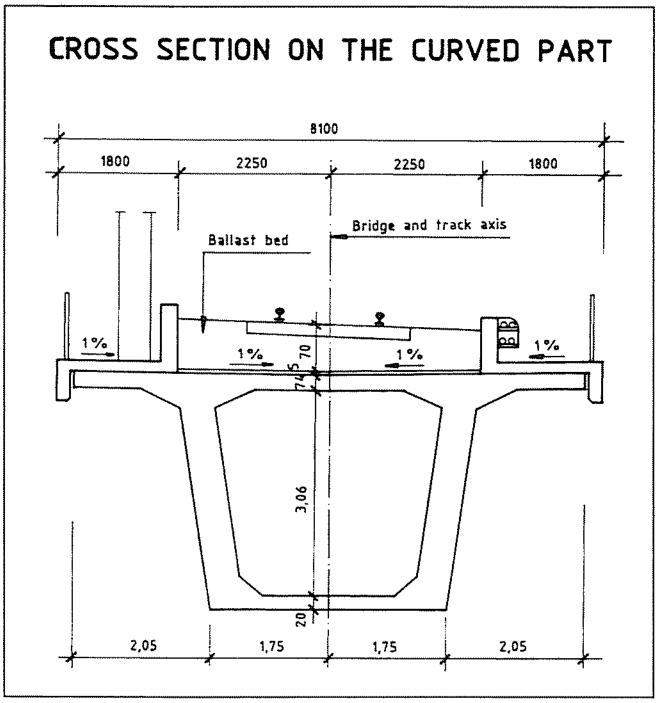


Fig. 2 Transverse section of prestressed concrete bridge elaborated in the tender plan

with 40-50 m spans with ballast on them proved to be the most favourable. Consequently, the following versions have been worked out at feasibility study stage:

- deck-plate girder steel bridge with orthotropic floor slab
- composite girder, compound steel main girder with reinforced concrete deck slab
- prestressed concrete bridge built by incremental launching method

5. LICENCING PLANS, TENDER SPECIFICATIONS

After reviewing and judging the feasibility studies, MÁV Co., as the investor, rejected the deck-plate girder steel bridge with orthotropic floor slab variant and commissioned the composite and the prestressed concrete bridge built by incremental launching method version on licensing plan level *(Fig. 2.)*.

Selecting and designing the structure of the viaducts is an immense responsibility since, owing to its size, the 1400 m long viaduct will be the longest railway bridge in Hungary. It will also be remarkable in European terms – as the fourth largest engineering structure on the European railway network.

It is worth mentioning in order to characterise the structure that the length of the engineering structure to be built on the new railway line represents about 5 % of the whole MÁV bridge span in track length. The planned lifetime of the bridge is 100 years. Thus it is clear that only such technical solutions can be considered which will be able to serve the railway traffic over several generations, at adequate levels of safety and at minimum maintenance costs.

Following MÁV's approval, the Chief Traffic Supervision Authority /Railway Supervision Department (Közlekedési Főfelügyelet Vasúti Felügyelete) issued a permit to the establishment for the submitted plans on 28 May 1997. According to the instructions of the permit, the bridges must be designed on the basis of the Hungarian Standard MSZ-07-2306/1-4 T, but, as an additional criterion, certain specifications of the EUROCODE 1: "Designing principles and effects imposed on the structures. Part 3: Traffic loads of bridges", were also made obligatory.

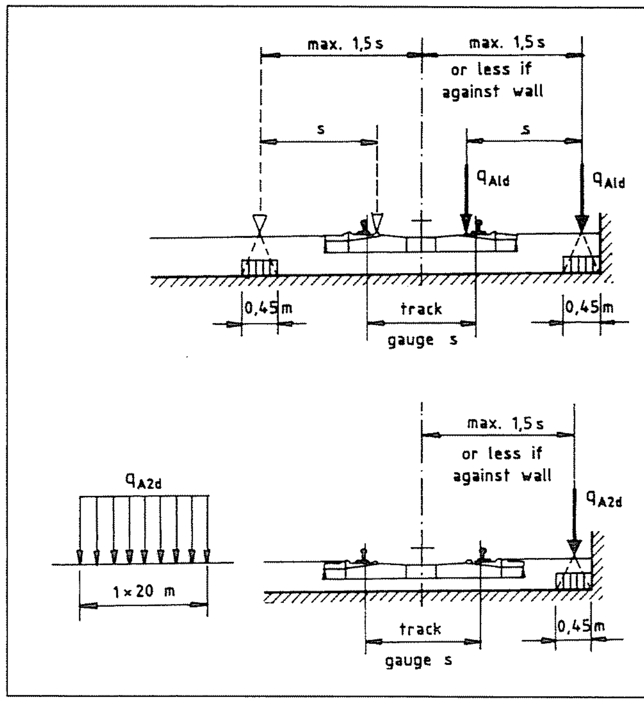


Fig. 3 Examination the effect of a derailed vehicle by EUROCODE

- It was further specified that the dynamic factor must be calculated with a formula related to averagely maintained track.
  $\phi_3 = (216 / L\phi^{0.5} - 0.2) + 0.73,$
  limit : $1.00 \leq \phi_3 \leq 2.00$
  where ϕ_3 = dynamic factor
  L_ϕ = characteristic length (m).
- Since there is no counter-rail designed on the viaduct according to the licensing plans, their superstructure has to be dimensioned (on basis of clause 6.7.1.2, "Derailment on the bridge: Structural requirements and equivalent loads."), for the derailed vehicle load, too. In designing phase 1. those structural elements affected by the derailed vehicle have to be examined in such a manner that the vertical, linearly distributed load substituting the two vehicle loads of q_{A1d} = 50 kN/m should be applied on a 6.4 m length *(Fig 3.)*. In designing phase 2. the stability of the bridge structure is examined for a q_{A2d} = 80 kN/m load, on 20 m length.

The tender specification has been made on the basis of designs with planning permission. The specification enabled the submission of further offers with alternative solutions, but with an obligatory elaboration of an "accelerated" version which would allow the opening to traffic to complete on the deadline of 31st November 2000.

6. THE VARIANTS SUBMITTED

The official opening of the tenders was on 9 April 1999. In response to the call for tendering, 17 tenders were compiled altogether from five tenderers *(Table 2)*. The bid of the ZALAHIDAK consortium, containing seven solutions elaborated to extremely high standards, is worth highlighting. Besides the members of the evaluating committee, the representatives of the bidders were also present during the bid opening procedure. The procedure was conducted by the deputy director of the MÁV. Within the confines of this paper a detailed description of the plan material cannot be given, thus only the list of the submitted bids will be presented here.

After reviewing the bids, the Tender Evaluation Commit-

Table 2 Bids submitted

Tenderer	Symbol	Deadlines Finished structure	 Completion
PORR	1a1	30.06.2001.	30.06.2001.
PORR	1alt1	31.01.2001.	31.01.2001.
PORR		31.03.2001.	31.03.2001.
ZALAHIDAK C.	2a1	15.09.2000.	30.11.2000.
ZALAHIDAK C.	2alt1	31.07.2000.	30.11.2000.
ZALAHIDAK C.	2alt2	31.07.2000.	30.11.2000.
ZALAHIDAK C.	2gy1/M**	15.09.2000.	30.11.2000.
ZALAHIDAK C.	2gy2	31.07.2000.	30.11.2000.
ZALAHIDAK C.	2gy3	31.07.2000.	30.11.2000.
ENGIL-MOTA	3a1	28.05.2001.	30.06.2001.
ENGIL-MOTA	3gy1	30.08.2000.*	30.08.2000.
ENGIL-MOTA	3gy2	30.08.2000.*	30.08.2000.
NCC INTERNATIONAL	4a1	23.03.2001.	25.06.2001.
NCC INTERNATIONAL	4alt1	23.03.2001.	30.04.2001.
NCC INTERNATIONAL	4gy1		01.12.2000.
KEV METRO Ltd	5a1	31.07.2000.	30.11.2000.
KEV METRO Ltd.	5agy1	31.07.2000.	30.11.2000.

Symbols:
a = basic version
alt = alternative version
gy = accelerated
* = exacting time schedule needed
** = modified version, not discussed at tender opening
First number = serial number of the bid
Last number = serial number of version within the bid

tee gave a proposal for further detailed evaluation of seven bids *(Table 3)*.

Major factors dictating the selection were the following:

- Feasibility of the expected deadline
- High technical standard of the submitted plans
- Minimal disturbance of the environment during the construction (usage of preparatory area)

Table 3 Tenders selected for detailed evaluation

Tenderer	Symbol	Type of structure	
PORR AG	1a1	Prestressed concrete	Prestressed concrete
ZALAHIDAK C.	2a1	Prestressed concrete	Prestressed concrete
ZALAHIDAK C.	2gy1	Prestressed concrete	Prestressed concrete
ENGIL MOTA	3a1	Prestressed concrete	Prestressed concret
ENGIL MOTA	3gy2	Composite R.C.deck slab with steel girder	Prestressed concrete
NCC International	4a1	Prestressed concrete	Prestressed concrete
KÉV-Metro Ltd.	5a-gy1	Prestressed concrete	Prestressed concrete

- Maintenance and operation cost demand.

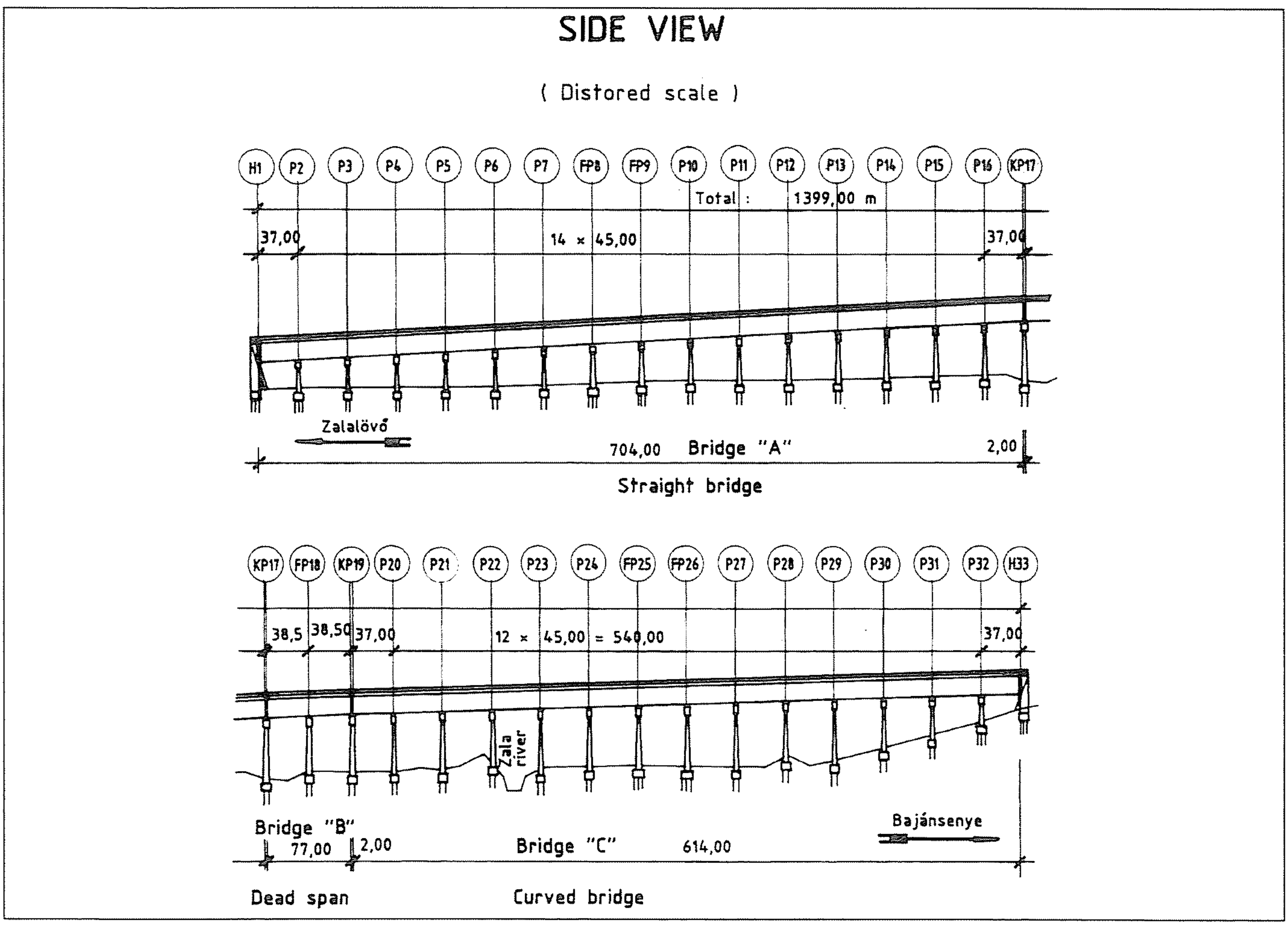
It is essential to mention that almost all of the bids evaluated contained the prestressed concrete bridge version with incremental launching. The composite bridge solution with reinforced concrete deck slab and compound steel girder was proposed only for one of the viaducts in bid 3gy2 of ENGIL-MOTA.

7. THE WINNING BID

Subsequent to a complete examination of the bids and the necessary analyses and comparisons, the ultimate decisions were made concerning the contractor for the 1400 m and the 200 m long viaducts. The award was made to the ZALAHIDAK Consortium supported by the HÍDÉPÍTŐ Co., as a parent company, leaving behind well-known Italian, Austrian, Swedish, Portuguese and other Hungarian firms. Further members of the Consortium: DUMEZ GTM of France and Betonútépítő International Co. The total sum of the implementation contract approximates to seven milliard HUF, including VAT.

The winning version – in respect of both viaducts – is a

Fig. 4 Elevation of the viaduct being built



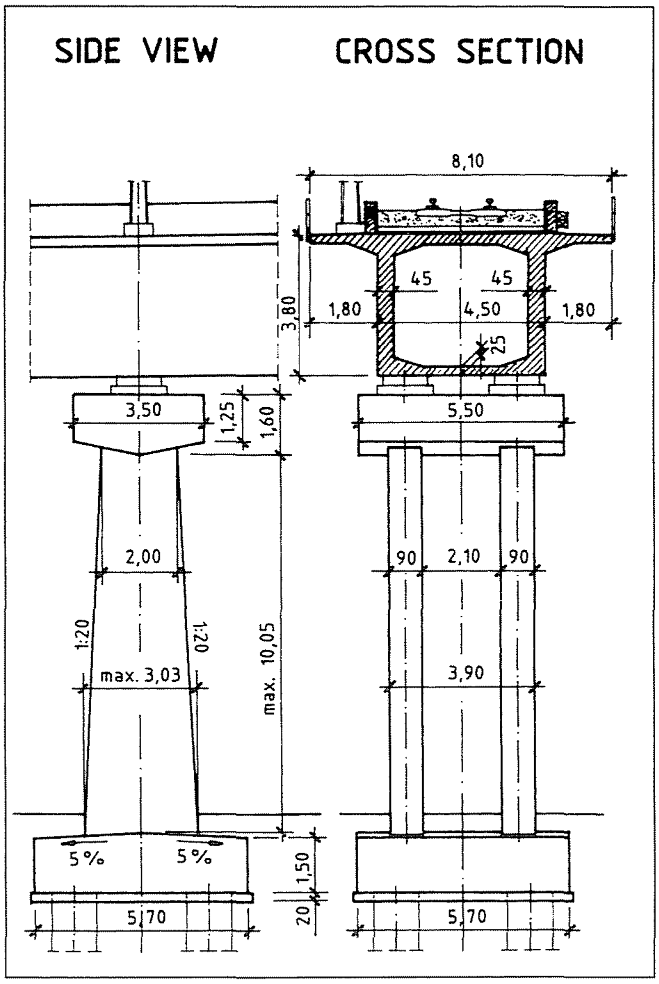


Fig. 5 Structural elements of the viaduct being built

prestressed concrete structure bridge with incremental launching method with piled foundations. The larger viaduct (1400 m) is supported by SOIL-MEC piles (*Fig. 4*). The one-cell box girder superstructure consists of three sections, of which the middle two-span will be built monolithically on scaffold. FRANKI-system piles support the small viaduct (200 m). The cross section of the superstructure of this bridge coincides with that of the large bridge. *(Fig. 5).*

In this paper only the major statements of the judgement are introduced. After examination of the tender plans the evaluating committee found the above mentioned plan to be the most detailed elaboration, possessing complete documentation of implementation-level technical solutions, detailed static calculations and adequately detailed organisation plans.

The elaboration level of the plan documentation was an essential element in the judging procedure because of the extremely short deadline. Only those plans were accepted which could guarantee high viability of elaboration.

The above version deviated least from that approved at planning permit stage with respect to structural type. The deviations were justified partly by the more accurate static calculations and partly by implementation aspects. The great merit of this solution is that there is no additional construction required beside the substructure works. The works can be organised in a shift-line system and the substructure works do not impede the superstructure construction.

Furthermore, the technology is deemed to be the most developed regarding domestic applications and has adequate references which is a guarantee for the provision of the necessary personal and asset conditions.

Attainability of the time schedule of the tender is guaranteed by the following facts:

- The substructure and the superstructure are to be constructed independently.
- The superstructure construction can be effectively winterised.
- The multiple work tasks can be concentrated and scheduled in parallel.
- The superstructure construction is a fixed, closed technology.

Advantages of the structure proposed:

- Stiff with low deformation characteristics.
- Favourable maintenance and operation costs.
- Least harmful effect on the environment in respect of construction and maintenance.

Disadvantages of the structure proposed:

- In case of elementary damage, the restoration is more complicated than in the case of simply supported structures.
- Due to the higher dead load and the statically undetermined structure, the foundation is more expensive.

Further information will be given in a following paper which will detail the preparatory works of the execution of the planned viaducts.

ESTABLISHMENTS

According to the inter-state treaty, the closing date of completion of the bridges is at the end of the current year. This presents an extraordinary challenge, not only for the investor and the constructor but also for all whom take part in the execution of this project.

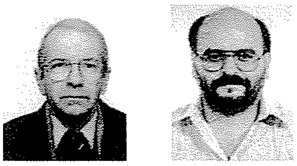
It is a fixed task for the contractor in the contract, that the viaducts should be suitable for track construction trains by the beginning of September. The completion date is 30th November 2000. (The deadline for operation is 31st December 2000)

REFERENCES

"Stresses and Strength of Reinforced and Prestressed Concrete Decks on railway Bridge" (ERRI Question D 183)

József VÖRÖS (1946) qualified civil engineer, director of the Bridge Division of MÁV RT. The national introduction of prestressed concrete bridges characterises his effective professional work. His activity in connection with the first balanced cantilevered bridge is recognised with a State Prize. He controlled the construction of the first balanced cantilevered prestressed concrete bridges which were built by incremental launching method.
He has been teaching at the Construction Execution Department of Technical University of Budapest since 1992 and is a member of the Hungarian Group of *fib*.

RESULTS OBTAINED FROM NON-DESTRUCTIVE TESTING OF BUILT-IN PRESTRESSING TENDONS



Dr. Lajos Imre – György Posgay

Changes in the stresses of prestressing tendons built-in prestressed reinforced concrete beams were monitored using a magneto-strictive, non-destructive testing method. This method meets the requirements of accuracy required from the points of view of economical application, reproducibility and structural inspection practice. During the application of the method, mechanical or electromagnetic vibrations generated nearby may have a disturbing effect and therefore they should be avoided.

Keywords: non-destructive testing, magnetic Barkhausen-noise, prestressing tendon, prestressed concrete

1. INTRODUCTION

Since the end of the 1940's the construction of prestressed reinforced concrete structures has been rapidly spreading in Hungary. Structural architecture first began to apply this type of structure mainly in the form of prestressed slabs and columns. Later these structures found applications in the field of bridge construction where they have been used mainly for road bridges, primarily in pre-tensioned form. Nowadays, both pre-tensioned and post-tensioned structures have a wide range of applications. However, the condition of such structures can have several problems which have a number of causes. These can be summarised as follows:

- Due to design or construction reasons, or due to corrosion damage caused by air pollution which has increased to a catastrophic extent in many places. In addition, damage is caused by the use of deicing salt on the roads in the winter periods.

- Due to relaxation, the actual tensile force can not be determined without the application of an established methodology and without which the actual structural behavior and load-bearing capacity can not be calculated accurately.

Particulaly the second problem (relaxation) gives reason for concern in case of bridge structures which are situated under aggressive atmospheric conditions for a long time. With this structure, along with the relaxation of prestressing tendons safety is affected by the corrosion of concrete. Thus control of changes in properties of the prestressing steel is of high importance.

Because of the above reasons there has been an ever-increasing need for non-destructive testing of the condition of built-in prestressing steel reinforcement (cables and tendons). The methods, such as the method of Teller, Suhler and Matzkanin (1990), used for determining the extent of corrosion will not be dealt with in this paper.

A possible testing (measurement) method for determining the actual tensile stress will be described. A method based on the measurement of several magnetic features (Dobman's 3 MA method, 1990) and an ultrasonic method (Gaydecky et al, 1991) are also known, but the method which will be described here has been found more suitable for testing built-in tendons during their lifetime.

2. DISCUSSION OF THE BASIC PHYSICAL PRINCIPLES OF THE TEST METHOD AND THE MAIN ASPECTS OF APPLICATION

2.1 The phenomenon behind the method

In 1917 a German physicist, H. Barkhausen, observed that while magnetising ferromagnetic materials a stochastic 'noise' was generated, the intensity of which (besides the crystalline structure) depended on the mechanical stress that was present in the material. During the examinations carried out later on this relationship it was found to be nearly linear in the elastic range. Below and above this range (depending on the composition of the material, etc.) the steepness of the characteristic curve gradually decreased, but it could be generated and set to be constant through controlled calibration. The basic requirement is that the element to be tested and the specimen used for calibration should be made of the same material and also use the same technology so that their chemical composition and crystalline structure are also the identical. (E.g. both should be rolled plates or wires of the same material.)

2.2 Effect of the measurement conditions

Depending on the intensity of magnetic excitation, the distance (air gap) between the measuring head and the element to be tested greatly influences the size of the signal obtained. Thus, the efficiency and expected accuracy of the measurement can be affected. In case of testing a particular tendon, the maintenance of the air gap at a low value (0.4 – 0.6 mm) can ensure that the standard deviation of the signal will be within a few percent.

The temperature of the surrounding materials and ambient air should also be taken into account. Therefore, the measuring equipment has to be designed to endure temperatures up to +80 °C which occur during fabrication (casting the concrete, steam curing) and temperatures between –20 to +30 °C occurring in extreme cases during the lifetime of the built-in beam. The measurement itself is normally carried out in a temperature range of –10 ~ +25 °C. In this range the temperature affects the measured result only within the measuring error.

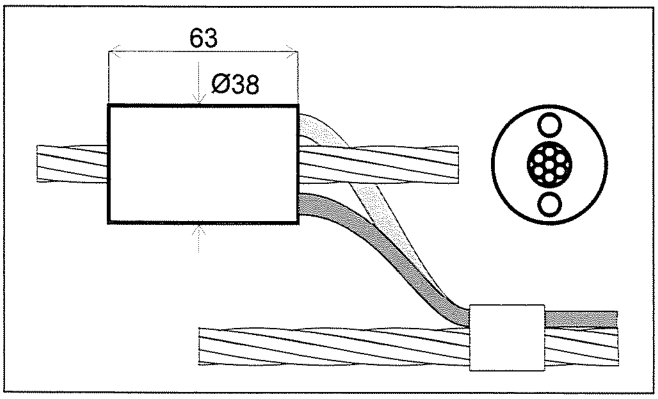


Fig. 1 Sensors pulled onto tendons built-in UBx typr prefabricated, prestressed concrete beams

According to our experiences mechanical vibrations influence the measured results in cases where their frequency is nearly identical with or less than that of the excitation. Therefore, performing a stress test at a time when there are vibrations generated by construction or by vehicular traffic is not recommended.

An electromagnetic field in the vicinity of the measurement has an interfering effect when its frequency is between 500 s^{-1} and 200000 s^{-1}. For example, welding being carried out nearby may have a disturbing effect. If there is a suspicion of interference the visible signal on an oscilloscope will indicate whether it is 'clean' or jammed by a spurious signal.

The change in the stress of tendons during the fabrication process of a prestressed reinforced concrete beam is influenced by the following:

- The order of stressing;
- casting and strengthening of the concrete,
- thermal motion during steam curing.

The effects of relaxation of the tendons develops later on. This effect can be monitored during the lifetime of the built-in beam.

2.3 Design of the measuring probes

Two types of measuring head have been developed for the test under discussion:

- The built-in measuring head installed during fabrication of the beam. This is the so-called through-coiled head which can be pulled onto the tendon as a ring (*Fig. 1*).
- for performing measurements on a beam not provided with a built-in measuring head, a mobile attachable measuring head is required.

It is important to protect the measuring heads against mechanical damage by providing appropriate solid cover around them, which should also resist the previously mentioned thermal effects. The electrical cables made for the measuring heads should be secure connections for taking possible later measurements. Leading the cables out of the properly designed end of the beam is recommended.

The test measurements were performed using a computer-controlled instrument - STRESSTEST 20.04 - designed for measuring Barkhausen-noise. The arrangement of the measuring heads on the tendons is shown on *Fig. 2*.

3. PERFORMING THE MEASUREMENTS AND THE FINDINGS

3.1 Preliminary experiments

In order to examine the relationship between the tensile force and the magnetic Barkhausen-noise (MBN), a calibration process was carried out on 6 tendons, the standard quality deviation of which remained in the permissible range. It is notable that during the tensile test of a solid rod the value of MBN increases with the tensile stress, in our case it was decreasing. This phenomenon is shown in *Fig. 3*.

Although there can be several explanations for the foregoing, Finnish researchers (Moilanen et al, 1992) hold the opinion that the reason for the phenomenon is the matrix structure of the cables. We think that this phenomenon is caused by the compressive stress occurring on the surface of the cables during stranding.

For certain advantages in assessment the so-called inverse Barkhausen-noise measurement method has also been performed during the calibration.

The signal-force relationship was found to be suitable for the assessment, but the standard deviation between the individual tendons (presumably due to the remaining stress resulting from the fabrication process) was quite considerable, i.e. ± 10% for 120 kN. Therefore, calibration is recommended for the actually built-in tendon.

3.2 Control measurements on the fabricated beam

The following measurements were taken on the Ubx-type bridge beam selected for the control measurement:

Fig. 2 Arrangement of tendons in an UBx type prestressed concrete beam and the location of sensors

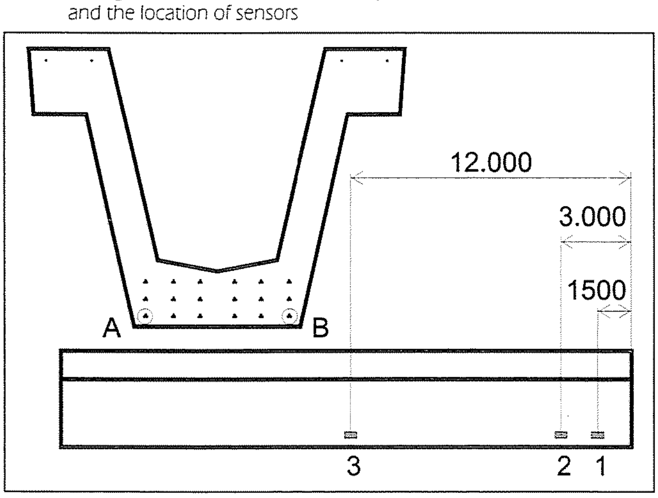
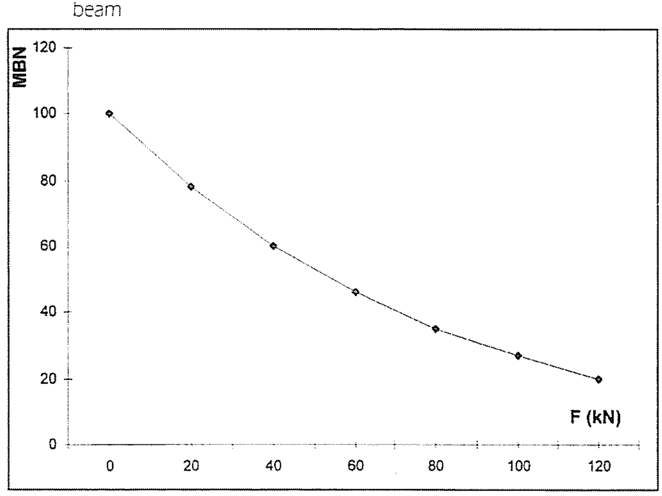


Fig. 3 Calibration curve of the tendon of an UBx type prestressed concrete beam



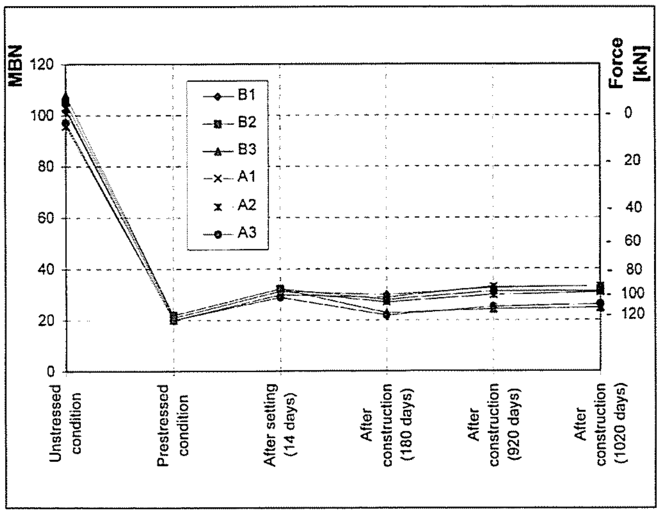


Fig. 4 Changes in the MBN values in the function of time

- basic measurement on the unstressed tendon,
- measurement on the prestressed tendon,
- control measurement at the concrete age of 14 days,
- measurement taken on the built-in beam (after 180 days in 1995),
- control measurement at the age of 920 and 1020 days in 1997.

3.3 Test results

The results of the measurements performed on 6 tendons of the bridge beam are shown in *Fig. 4*.

The reasons for the changes observed (Fig.4) in the individual values measured on the tendons were as follows:

- The decrease in the prestressing force that took place in the first 14 days after casting the concrete is caused by the relaxation of the concrete.
- The relaxation of the tendons also played a role in the 5-10% decrease in prestressing force observed on the built-in 3-year-old beam (measured after 920 and 1020 days).

It is believed that further control measurements should be performed in order to fully observe the processes taking place in the beam tested.

4. CONCLUSIONS

The non-destructive measurement method was found suitable for observing the changes in the stress condition of the built-in prestressing tendons. Proper accuracy can be expected of this method if during construction individual values are specified for each tendon by means of basic measurements. A nearby electromagnetic vibration with frequencies between 500 s^{-1} and 200000 s^{-1}, and mechanical vibrations with frequencies near to or less than that of the excitation can have disturbing effects. For the checking of prestressing tendons in existing beams, the provision for direct access to the tendons is necessary.

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György POSGAY (1958), a certified physicist, has been the managing director of METALELEKTRO Technical Development Ltd. since 1991. In Hungary he is one of the founders of the material and stress measurement method based on the measurement of the magnetic Barkhausen-noise. His name is linked, among others, with the development of measurement methods for determining the neutral temperature of continuous rails (Rail Scan), the bolt-stress of high strength tensioning bolts (BoltStress) and the originality test of the chassis number of road vehicles (VinTest).

CRACKS IN FLOOR SLABS OF BASEMENT CAR PARKS

 

Miklós Armuth - Prof. György Deák

During their recent work as experts, the authors have prepared several authoritative opinions on floor slabs of basement car parks. A common impairment of buildings of diverse size with varied ground plans is the occurrence of numerous cracks on the concrete flat slabs of the basement. In most cases, cracks were discovered when thawed salty snow or water originating from vehicles leaked through cracks in the slab and damaged the bodywork of other vehicles (which was revealed by spots of changed colour and which led to litigation in some cases). This paper introduces common structural layouts of the investigated basement car parks and describes typical crack patterns. At the end of the paper, the authors provide a list of conclusions drawn from the analysis of the causes of crack formation, the consideration of which may indicate ways of effectively reducing the risk of damage to similar structures.

Keywords: car park, diaphragm wall, flat slab, hydration heat, shrinkage, cracking, watertightness

1. INTRODUCTION

In Budapest in the 1990s (and the present), a common consideration related to new-build offices was the provision of adequate car parking places. Investors are obliged to provide sufficient parking places not only to meet users' demands but also to comply with construction regulations which are becoming stricter. Although there are several solutions to this issue (for example, the erection of parking structures in the vicinity; the use of the ground floor and possibly the lower floors of the building; the provision of parking places on nearby disused sites, etc.), in all the buildings studied, it is the basement floors below that are designated as car parking areas. Normally, 2 or 3 parking levels are constructed, but there are examples of 5-level basement car parks as well. Load-bearing structures of office buildings which are often lock-ups or are built on narrow sites are quite similar: The design and techniques that seemed to work were typically applied. It is quite obvious that these structures of very similar layout suffer the same sort of impairment: Through-cracks form in the floor slabs of basement car parks below ground level in most cases. Though crack formation of such a type is not experienced exclusively in Hungary (Springenschmid, Fleischer 1993; Grasser, Luy 1998), no generally accepted and applied method of prevention has been developed so far.

This paper analyses the causes of crack formation and gives proposals and suggestions for investors, designers and constructors on how to solve or at least alleviate problems concerning cracks.

2. LOAD-BEARING STRUCTURES OF BASEMENT CAR PARKS

In the case of all buildings studied, the sides of the building pits were supported by anchored diaphragm walls of 500-650 mm thickness, the panels of which were sunk one-by-one. Panels are 1.5-4.5 m wide and are anchored by one or two ground anchors. Anchors were normally placed in a single line (at the same height) but in case of deeper working pits, two rows of anchors were used. Diaphragm walls run down 1.5-5.0 m (depending on ground quality) under the level of the cast-in-place reinforced concrete foundation raft. Both top and bottom surfaces of the foundation rafts are flat, and they are 0.6-1.0 m thick. (Slab thickness depends on the load the building has to support, the spacing of the column-grid and the physical properties of the ground.) Foundation rafts were designed to bear underground water pressure if necessary, but in cases of buildings with light loads (and a high risk of floating), foundation rafts were holed at some locations to facilitate the pumping out of sub-surface water to prevent the increase of upward pressure. A foundation raft and a diaphragm wall may be either structurally independent (if significant relative movement is expected vertically), or structurally connected. Foundation rafts were made of watertight concrete in all cases. And movement joints were never provided in the foundation (even if there were some in the structure above ground level). This construction can be observed despite the fact that some buildings are longer than 100 m.

Vertical load-bearing structures of the buildings are normally in-situ reinforced concrete columns and walls. Characteristic of the ground plans, column spacing (generally 5.00-7.50 m) was determined according to the requirements of the basement floors: that is, the dimensions of parking places and aisle-ways. Reinforced concrete walls are mostly applied as walls of staircases and lifts, as firebreak walls, and as supports to ramps.

Floor slabs are typically two-way spanning, reinforced concrete flat slabs cast in-situ. There are two types of slab support - columns in the middle and by walls on the edges. In most cases, when there is no separate reinforced concrete wall inside the diaphragm wall, the floor slab is supported by the diaphragm wall (*Fig. 1.a*). Here, notch-forming elements are fitted to the steel reinforcement when being sunk in the trench for the diaphragm wall supported by bentonite suspension. In some rare cases, diaphragm walls only support the sides of the building pit and bear ground pressure, and therefore do not contribute to the support of vertical loads of the building. In such cases, the edges of the basement floor slabs are supported by reinforced concrete walls parallel to the diaphragm wall. The internal wall and the diaphragm wall are isolated by plastic drain plates (*Fig. 1.b*). Out of 8 buildings studied, this type of structure was found in two cases. Floor slabs are 23-25 cm

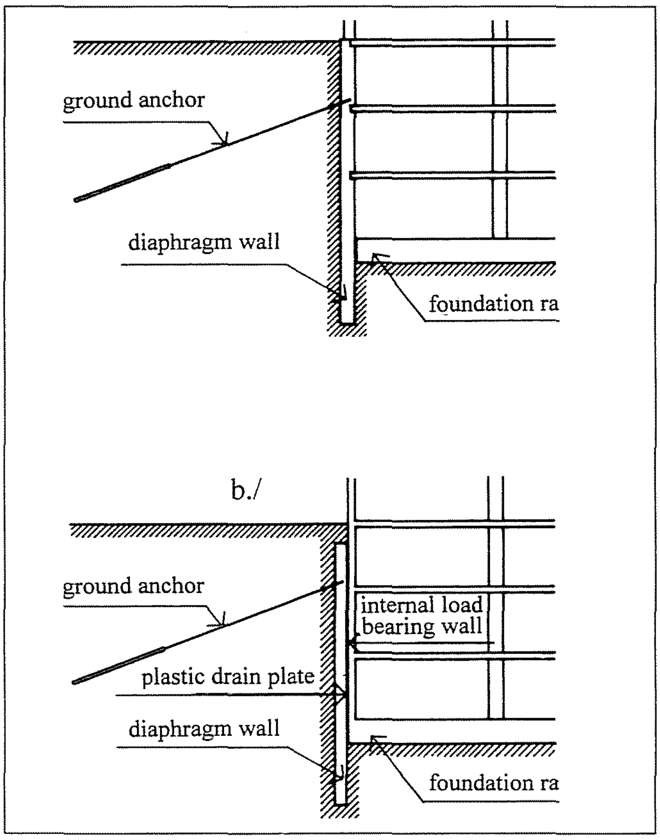

Fig. 1 Connection of diaphragm wall and basement floor slabs.
a) Floor slab edges of the basement supported by the wall.
b) The diaphragm wall acting as an external wall to support ground pressure: floor slabs are supported by a separate wall.

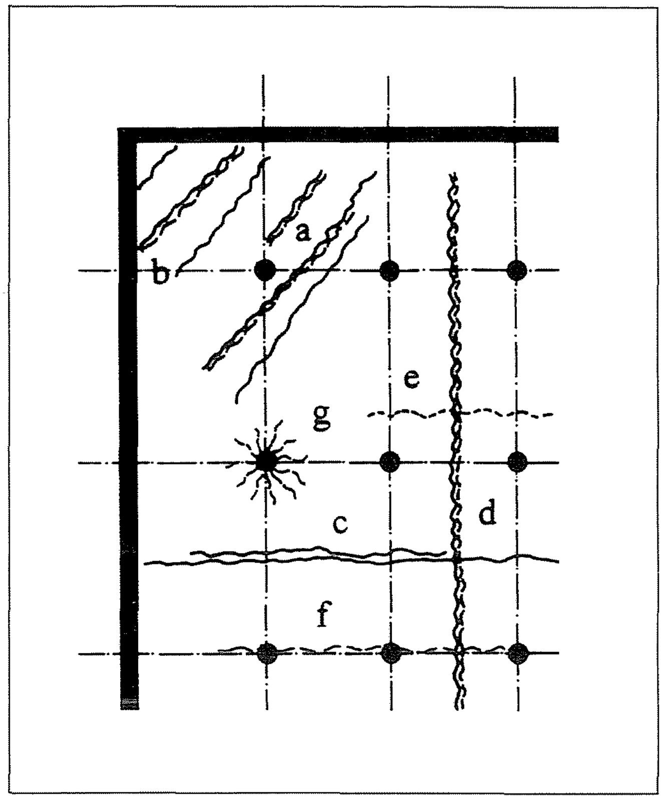

Fig. 2 Typical floor slabs cracks:
(a) a through-crack at an angle of approx. 45° at the joint of diaphragm walls.
(b) a crack on the bottom surface of the floor slab at an angle of approx. 45° at the joint of diaphragm walls.
(c) cracks on the bottom surface of the floor slab at the location of the maximum positive moment.
(d) through cracks at the mid-span.
(e) a crack on the top surface of the floor slab around the line where the moment is zero.
(f) a crack on the top surface of the floor slab at the line where the negative moment is zero.
(g) radial cracks on the top surface of the floor slab around the columns.

thick, mostly with flat top and bottom surfaces; double slab thickness in a 2x2 m area around columns (i.e. a transfer plate) was only found in one building. Movement joints in the structure above ground level were applied only at buildings longer than 50-60 m. The applied concrete strength class was C20/25 or C25/30. (Neither the designer nor the constructor could provide more details on concrete mixture.) B 60.40 or B 60.50 quality steel was used for the reinforcement of floor slabs, which is welded-wire fabric in some buildings.

The bottom surface of the floor slabs is exposed concrete: neither plastering nor suspended ceiling is applied and the surface is painted only in one case. On the top surface of the floor slabs no special architectural layers have been placed: In most cases, a simple wearing course 1-3 cm thick is applied. A 30-mm thick asphalt layer is generally applied but in one example building the floor surface is finished by simply levelling the top surface of the reinforced concrete slab. (A more elaborate set of layers is only applied to floor slabs at ground floor level.)

3. CRACKS IN FLOOR SLABS

Cracks on the bottom surface of slabs were easy to record in all but one case (where thermal insulation panels, type "HERATEKTA", were fitted on the slab). However, inspection of the top surface of floor slabs was often hindered by floor coverings since some inspections were carried out in buildings after the construction work was finished. Floor layers on the ground floor made the inspection of the ground floor slab basically impossible. However, in the case of the other floors, a thin, single-layer cover rigidly attached to the slab only slightly influenced the visibility of cracks. Some cracks were registered immediately after their occurrence, during construction while others were noticed when the building was already in use and when water was leaking through the slab.

Besides several cracks forming no special pattern, there were many typical cracks observed on many floor slabs. These were formed in a similar way. Cracks without a discernible pattern can be attributed to unique building layouts and construction; factors concerning concrete casting (weather conditions, construction joints, etc.), circumstances related to treatment during curing and removal of formwork as well as further accidental effects. Typical cracks are shown in *Fig. 2*. Different crack types are indicated by letters ***a-f***. The width of the cracks on the tension side of the floor slabs, around the location of maximum moments (cracks type ***c, f, and g***), was 0.1–0.3 mm. The width of through-cracks (visible both from the top and from the bottom), type ***a and d*** was 0.1–0.8 mm. (However, note that top and bottom cracks at the same location which are not through-cracks might have been induced by moment of change and faster concrete shrinkage close to the surface. In the current case, this assumption is not confirmed by the nature of the effect of actions and small thickness of slabs. The existence of through-cracks was proven by water leakage.)

In all cases, there were wider cracks in greater numbers on floor slabs supported by diaphragm walls, than on slabs independent of diaphragm walls and supported by separate reinforced concrete walls inside them. A similar observation was made within one building when comparing floor slabs of different floors: more and wider cracks were formed in slabs of lower floors (e.g. crack width at the corners, at 45° angle, above level –3, was 0.5–0.8 mm; while it was 0.1–0.2 mm above level –1).

In many buildings, 0.1–0.2 mm wide cracks occurred in the slab of the drive-down ramp to the car park, close to the entrance, perpendicular to the longer axis of the ramp and also on the walls and floor slabs around the entrance.

4. SIGNIFICANCE OF CRACKS

Aspects to be analysed in relation to cracks are load-bearing capacity, stiffness (deformation), aesthetic and psychological effect, corrosion, and watertightness.

In professional practice, cracks induced by tensile stresses are considered inevitable. However, crack width in a service situation should be limited. Concerning the visual (aesthetic) effect, both the Hungarian Standard and ENV 1992-1-1 (EC2), specify an allowable value of 0.3 mm. Regarding protection of reinforcement against corrosion, the allowable value of 0.1 mm in the Hungarian Standard may be applied for parking structures (since they are either "structures exposed to water on occasion" or "structures exposed to aggressive gases and liquids..."). According to ENV 1992-1-1, parking structures are classified under environment aggressiveness class 3 (humid environment with frost and de-icing salts). However, the allowable value of 0.3 mm is still acceptable. In the case of prestressed structures, both regulations specify stricter requirements because of the higher sensitivity of the reinforcement to corrosion.

As for the requirement for watertightness, 0.2 mm is permitted in the case of cross-sections having a concrete compression zone. According to the Hungarian Standard, 0.1 mm is acceptable in the case of through-cracks. The problem of watertightness in relation to buildings is not handled in ENV 1992-1-1. According to the specification on the design of liquid retaining and containment structures (ENV 1992-4:1998), if watertightness is required, either a 50 mm high concrete compression zone (not cracked) is necessary. Otherwise, through-cracks of at most 0.2 mm or 0.1 mm wide are acceptable, depending on whether self-healing of cracks due to cement hydration is expected or not. Nevertheless, no open nor through-cracks are allowed at all in the structure if no water bleeding is permissible even locally.

The approach of the Eurocode introduced above is supported by the observations of the inspections described in the previous section. *Water bleeding was discovered in several floor slabs examined. Moreover, traces of steel corrosion were observed in association with cracks 0.1 mm wide.*

Self-healing of cracks is generated by the further hydration of cement. Sometimes water leaking through the slab is not enough for hydration but constant damp condition of concrete is necessary. Besides this, a condition of "stillness" or a rest period is also needed according to the EC2 specification mentioned above: Fluctuation of the specific strain of concrete is limited at $\Delta\varepsilon_{ct} = 150\times10^{-6}$. These requirements are also confirmed by the experience of the authors. In the case of a specific structure inspected, 0.1–0.2 mm wide cracks were found and water bleeding occurred before putting the basement into use. About one and a half years later, water leakage through cracks that had developed earlier was still a problem in the basement car park already in use. (And further cracks occurred, too.)

Added to the above facts, investors usually wish to save the costs of watertight floor coverings. This helps to clarify the significance of the problem of cracks in the floors of basement car parks. Water and salty, thawed snow leaking through the cracks of floor slabs not only leads to staining of the floor slab, the aesthetic depreciation of the structure and corrosion of reinforcement, but it can also severely damage cars parking on the floor below. This in turn can put the operator of the building to greater expense and increases the costs of structural repair.

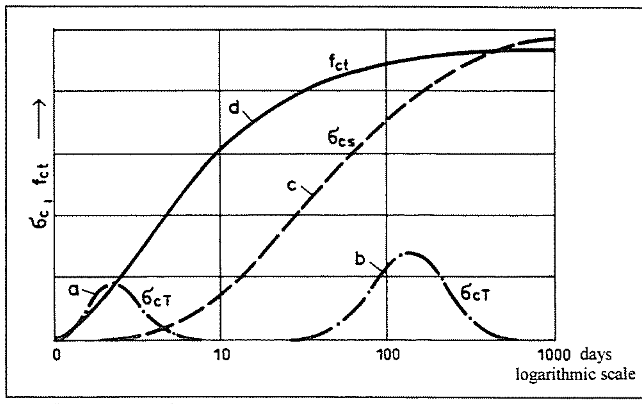

Fig. 3 Evolution of tensile stresses and tensile strength in the concrete.
(a) σ_{cT} – tensile stress induced by hydration heat.
(b) σ_{cT} – tensile stress induced by climatic heat.
(c) σ_{cs} – tensile stress induced by concrete shrinkage.
(d) f_{ct} – tensile strength of concrete.

5. CAUSES OF CRACK FORMATION

It is important to summarise the nature and circumstances of crack formation, since it has great relevance:

- Most cracks occurred before putting the building to use even though no considerable load beyond the self-weight of the structure acted previously.
- There were some additional cracks which had a unique crack pattern: e.g. along the line where the moment is zero and at the junction of diaphragm walls.
- A great many cracks were through-cracks with a width approximately the same at the top and bottom.

Flat slabs, especially the intersection of middle strips, normally remain uncracked even in a service situation (the minimum amount of reinforcement specified in codes is relevant). Apparently, imposed deformation (i.e. displacement or specific deformation) was the determining factor at the inspected structures: restrained deformation induced by hydration (and perhaps climatic) heat and concrete shrinkage.

Evolution of imposed deformations and the tensile strength of concrete are shown in *Fig. 3*. (The order of the proportion of stresses to the tensile strength is shown in the figure.)

5.1 Hydration heat

It is well-known that hydration of cement (dissolution and crystallisation) generates heat and the temperature of concrete increases during curing. In the case of modern, finely ground, high-strength, fast-hardening types of cement, this effect is quite significant. The increasing temperature of concrete further accelerates hydration and related heat generation. This process is illustrated in a document issued by Heidelberger Zement (*Fig. 4*). The diagram represents the increase in temperature in subject slabs of different thickness where the cement content is 300 kg/m³.

Naturally, the increased temperature of slabs with a larger cooling surface/weight ratio (i.e. thinner slabs) is smaller.

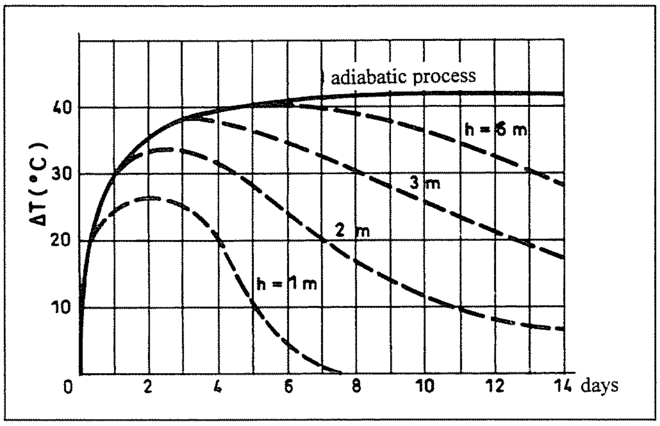

Fig. 4 Increase of the concrete core temperature due to hydration. (300 kg/m³ Portland cement CEM I 32.5 R)

However, concrete may also warm due to intense solar radiation or if covered with thermal insulation.

During the continued cooling of concrete already hardened it is difficult to trace tensile stresses. This is due to restrained shortening since not only the temperature of the concrete, but also its tensile strength, modulus of elasticity and creep coefficient change in the course of time. Assuming that the coefficient of thermal expansion of hardened concrete is: $\alpha_{cT} = 10 \cdot 10^{-6}$ °C^{-1} and its ultimate strain is approximately: $\varepsilon_{ctu} \cong 0.15 \cdot 10^{-3} = 0.15$ mm/m, then $\Delta T = -15$ °C uniform cooling along the height of the cross-section may induce through-cracks, and non-uniform cooling may start the formation of such cracks, if the deformation is restrained.

Note that sections of Eurocode dealing with loads (ENV 1991-2-5: 1997; ENV 1991-2-6: 1997) draw attention to the significance of hydration heat, but give no information on its magnitude.

5.2 Climatic heat

Effects of *high external temperature* and intense solar radiation during concrete curing were referred to earlier. However, *climatic cooling* is more likely to take place in normal operation. In the case of basement car parks below ground level and thermally insulated from the top, it can hardly lead to general cooling of the floor slabs to the extent mentioned above. It is rather the entrance of the car park and the top section of the ramps that are more likely to be to exposed local cooling. Local crack formation caused by this effect was observed at some basement car parks.

The superimposing of the two effects, concrete shrinkage and external climatic effects, should be anticipated to some extent.

5.3 Concrete shrinkage

Concrete shrinkage (deformation independent of stress and temperature change) is a combination of three more or less separate processes:

- decrease of volume due to cement hydration,
- equalisation of external relative humidity and that internally in concrete pores: drying of the concrete,
- decrease of volume due to carbonation of calcium hydroxide formed from cement (mainly close to the surface of the structure).

It seems that professional opinion in Hungary together with the appertaining regulations tends to under-estimate the significance of concrete shrinkage. As applied to floor slabs of basement car parks (inside, slab thickness up to app. 200 mm), ENV 1992-1-1 approximates the ultimate value of concrete shrinkage at $\varepsilon_{cs} \cong 0.6 \cdot 10^{-3}$, while the Hungarian Standard specifies a value of $0.4 \cdot 10^{-3}$ for the same. Since both values are a good deal larger than the ultimate tensile strain of concrete ($\varepsilon_{ctu} \cong 0.15 \cdot 10^{-3}$), the concrete will crack if shortening is entirely restrained. However, in case of partial restraint, we may assume the process of shrinkage to be relatively slow: 50% of the final shrinkage takes place in about half a year. Thus, the effect of shrinkage adds to the effect of cooling after the generation of hydration heat only to a small extent (Krüger, Binder, 1988). Further, the speed of relaxation is close to that of shrinkage which can consequently discard a certain part of tensile stresses due to shrinkage. Also, the occurrence of some cracks can "soften" restraining effects and thus, prevent further crack formation induced by imposed deformations (ENV 1992-1-1; Gilbert, 1992).

5.4 Factors restraining tensile deformation of floor slabs

Owing to the superimposed deformations described above (cooling after hydration, shrinkage), points of the floor slab with the possibility of free deformation will move towards the geometric centre of the slab.

The main elements restraining displacement are the diaphragm walls, provided that besides serving as external basement walls they also serve as the lateral support to the floor slabs and that the joint of the slab and the wall hinders horizontal displacement of the slab. Cooling and deformation of diaphragm walls after hydration takes place prior to the construction of floor slabs. The process of shrinkage is relatively slow because of the wall thickness and soil moisture. Its ultimate value is smaller than the shrinkage of the floor slab.

Sections of the diaphragm wall restrain the shortening of the connected floor slabs to a different extent (*Fig. 5*). In the middle of the building, some displacement of the diaphragm walls takes place inwards, perpendicular to the plane of the wall, due to ground pressure and the tensile force transferred from the floor slab. The fact that diaphragm walls are *clamped in* the ground does not influence upper levels of the basement considerably. (This partly explains why crack formation is more intense on floor slabs of lowermost levels than in upper slabs.) Anchors act as load-bearing "soft" supports and their effect is terminated if they are cut off later. However, at the corners of the building, the joining of two diaphragm walls has a significant stiffness in its plane, and in consequence this section may be considered as a perfect restraint. This model is supported by crack patterns (mainly the 45° cracks at the corners) recorded at inspections.

Concerning the deformation restraining effect of *stiffening walls*, which support lateral loads and stabilise the building, it is important to understand that their arrangement has more significance than the size of the given building section horizontally (Rosman, 1998) (*Figure 6*). Walls with axes perpendicular to the main displacement direction hardly restrain deformations of the floor slab (*Fig. 6*). However, walls with axes parallel to the displacement act in a similar way to the diaphragm walls at the corners of the building. A stiffening core placed around the centre of the building (*Fig. 6*), which may be combined with stiffening walls according to Figure 6.a, is also a favourable arrangement. On the contrary, in the case where two or more stiffening cores are situated in the same

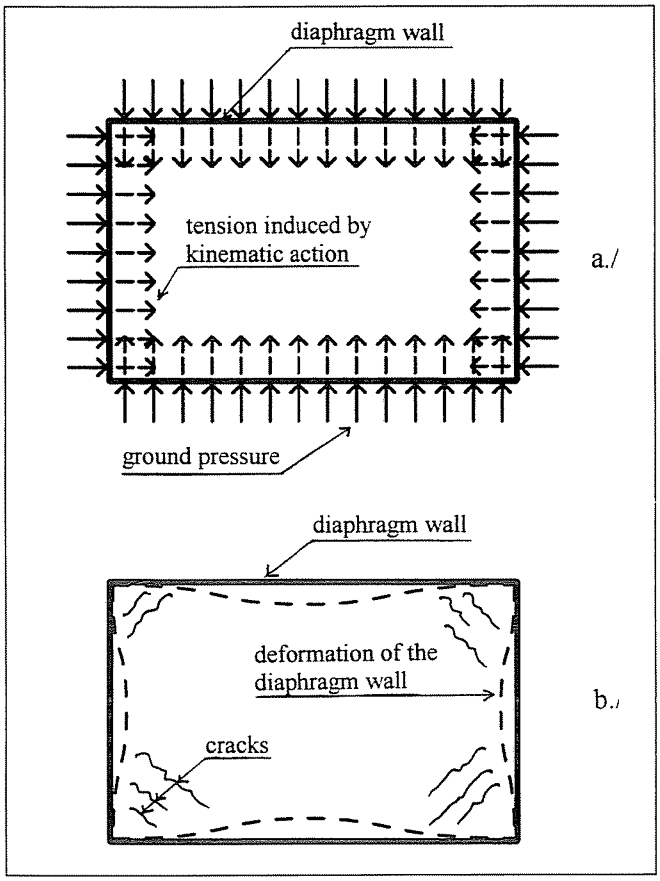

Fig. 5 Deformation of diaphragm wall
(a) horizontal loads acting on the diaphragm wall
(b) deformation and cracks of the diaphragm wall

dilatation unit, the occurrence of through-cracks due to shrinkage is almost inevitable (*Fig. 6.c*).

Concerning the restraining effect of columns, we can conclude that this is generally neglected, partly because of the slenderness of columns but also because relative displacement of floor slabs one above the other is relatively small. (This applies mainly to the effect of shrinkage, and not to that of hydration heat.) Even so, columns of the lowest level are in a significantly different situation. Horizontal deformations of a thick foundation raft and a floor slab above may differ considerably from each other in terms of absolute magnitude as well as time. This can be accounted for by considering the differences in the timed sequence of construction together with the factors of slab thickness, the drying process and also because of the friction between the foundation raft and the ground. The collective restraining effect of short columns of large cross-sectional area with massive reinforcement and which are rigidly clamped in the foundation raft may induce cracks in the lowermost deck slab.

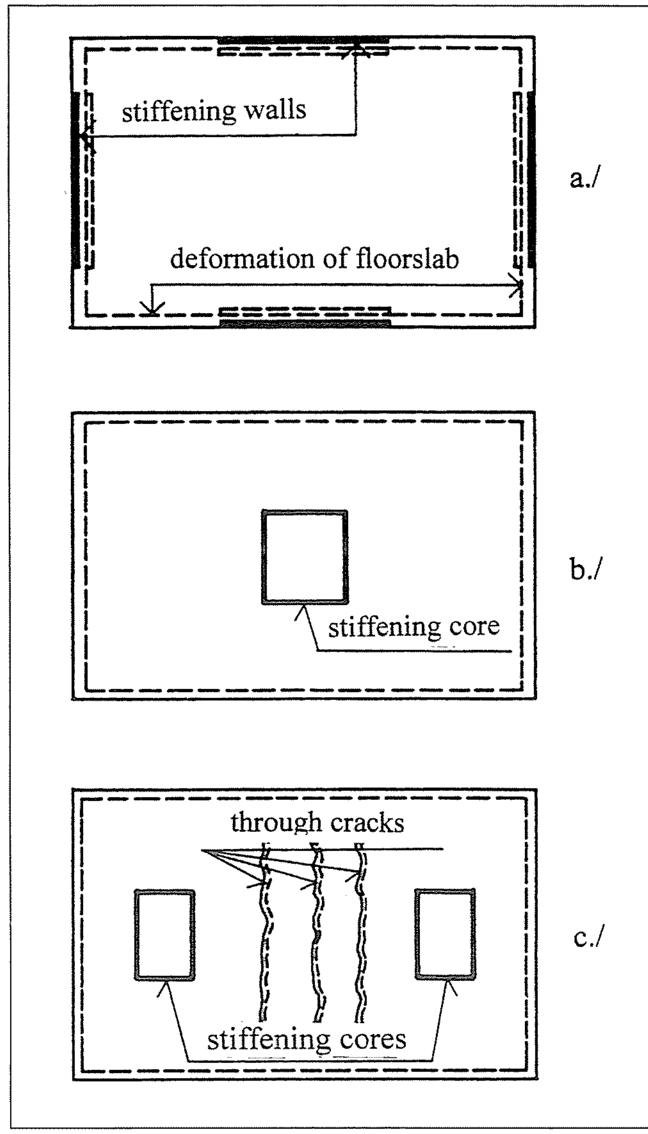

Fig. 6 Effect of the arrangement of stiffening structures:
a) favourable
b) favourable
c) unfavourable arrangement

tions is not sufficient. Instead, the risk of formation of through-cracks (and water leakage) should be reduced to a minimum. The following methods may be applied for that:

- keeping water away from possible cracks locations,
- limiting the magnitude of imposed deformations,
- allowing free deformation due to imposed deformations (heat, shrinkage), and limiting restraining effects,
- post-tensioning of floor slabs.

These options should be properly combined with economic considerations to reach an optimal conclusion.

6. PREVENTION OF THROUGH–CRACKS OF FLOOR SLABS

As shown by the above descriptions, through-cracks of floor slabs of basement car parks are dangerous mainly because of (aggressive) water and salty, thawed snow originating from cars which can leak through the slab. According to the experience of the authors, there is agreement with the related specification of ENV 1992-1-1 (see section 4), in the sense that limiting through-cracks width induced by imposed deforma-

6.1 Keeping water away from cracks

Water encroaching upon on the floor slabs of car parks originates mainly from rain, thawed ice or salty snow dripping from vehicles. Alternatively, cleaning operations are a source of ingress. To provide efficient drainage, *floor slopes* should be formed by taking *deflections of the slab* into account. Additionally, floor drains should be arranged accurately so that water collection in puddles is prevented.

If a floor covering is implemented (even in post-commission repair situations), this should be either a water-proof or watertight covering that is capable of spanning cracks of the reinforced concrete structure it is placed upon. Synthetic resin based, high-strength, self-spreading but rigid claddings with high adhesive strength are *inadequate* to this purpose. Floor coverings that tolerate displacements of the floor slab are offered by some producers and water-insulating floor coverings consisting of several layers is also an alternative solution, though more expensive.

6.2 Limitation of effects restraining the deformations

The restraining effect of diaphragm walls on the shortening of floor slabs connected to them may be reduced to a minimum. In principle, this can be achieved if the applied connections provide continuous or discontinuous support for the edge of the slab but facilitate horizontal displacements of the slab both parallel and perpendicular to the diaphragm wall at the same time. At the corners of the building it is especially important to use such connections. However, the authors have never seen such connections during their work as experts.

In the case of one of the inspected example buildings, another type of structure was used where the diaphragm wall served solely as an external wall of the basement and the floor slabs are supported by a separate wall inside and which transfers its load to the foundation raft (described at the beginning of this paper). There are a number of ways to solve the problem of water insulation between the two walls (i.e. drainage water that gets through the diaphragm wall). The inside wall was built at about the same time as the floor slabs, so their cooling and shrinkage are not much different. Moreover, this wall is much thinner than the diaphragm wall so it can more easily adjust to floor slabs displacement horizontally at their edges. In the building where this structure was applied, the authors found hardly any through-cracks in the floor slabs.

The location of the floor slab above or below ground level does not make a significant difference in the effect of stiffening cores on crack formation of floor slabs. Aspects to be considered at the arrangement of stiffening walls and cores (wall systems) were presented in section 5.4.

A further way of preventing the restraining effect of diaphragm walls and stiffening elements on deformations is to apply movement joints. Unfortunately, designing a sectioned load-bearing structure to be live-loaded with moving vehicles and constructed using a joint that will not get damaged easily with the provision for a durable sealant to resist water drainage, are sophisticated and expensive. This explains why both designers and constructors seem averse to this concept. (Cuts in floor slabs, such as those along the drive-down ramps act partly as expansion joints.) For car parks with adjacent floor sections shifted by half a level, displacement of floor slabs is facilitated in one direction, so it is sufficient to provide movement joints in ramps.

6.3 Limiting the magnitude of imposed deformations

Imposed deformations inducing tensile stresses (causing shortening) in floor slabs of car parks in instances of restrained deformations (cooling after hydration and concrete shrinkage), may be effectively limited by utilising specific concrete technology methods. In order to limit the generation of hydration heat it is advisable to use slow-hardening cements that generate less heat (e.g. blast furnace slag-Portland cement). In addition, the use of the least allowable amount of cement is recommended. The foregoing is also favourable with regard to the limitation of concrete shrinkage. Given the cement type and quantity, a lower water/cement ratio will result in minimising concrete shrinkage. Further, instead of applying a high water/cement ratio, the use of plasticizers is advised where the concrete is to be pumped.

Where high external temperature prevail (e.g. above 30°C), *cooling of fresh concrete* may be necessary, for instance, by supplying it with liquefied nitrogen. Vibrovacuum processing (i.e. removing excess water and compaction of concrete) greatly reduces concrete shrinkage (Armuth, A., 1960). Crushed ice is sometimes also employed also for these purposes. Concrete curing is also an important factor: Maintaining the concrete in a constantly damp condition and protecting it against intense solar radiation over a period of approximately 7 days.

With regard to concrete mix selection, cement quality and quantity, the *designer and the constructor are adverse parties*, as the latter wishes to use fast-hardening concrete in order to facilitate removal of formwork, etc. as early as possible. Thus, the designer needs to specify other data apart from the compressive strength of concrete and the two parties are advised to agree on details of the concrete mixture, method and duration of curing by consulting a concrete technologist. While the low w/c ratio is favourable in controlling shrinkage, it will on the other hand increase the rate of hardening. Nevertheless, a compromise between the differing demands of designer and contractor may be finally achieved.

The method of *casting concrete in sections* is quite widespread in structures of massive reinforced concrete in order to reduce the effect of imposed deformations. Following the casting of a number of concrete sections with gaps between them, cooling after hydration and concrete shrinkage can partly take place with the joints in-filled afterwards. Tensile strength of the concrete at these construction joints is lower than at ordinary locations. This technique leads to construction delays, however, which explains why it is rarely applied in practice.

6.4 The effect of reinforcement

As is well-known in practice, steel reinforcement cannot prevent crack formation due to tensile stresses, but it can limit crack width by limiting crack spacing. In connection with cracks in the tension zone due to bending, crack spacing and crack width must be checked according to MSz 15022/2 (Hungarian Standard) or ENV 1992-1-1. The latter also includes regulations in cases of tensile forces due to imposed deformation acting in the cross-section. In addition, requirements related to minimum reinforcement ratio, layout, bond distance and splicing must also be met.

Owing to the fact that steel and concrete are close in their coefficients of thermal expansion, change of temperature does not induce different deformations in the two materials, so there is no transfer of forces between them and the presence of steel does not influence concrete stresses.

However, the role of the compression zone due to bending is rather controversial. In relation to the watertightness of the slab, the steel reinforcement is rather more harmful than useful, since it reduces the thickness of concrete's compression zone. (Note that the requirement of a minimum of a 50-mm thick compression zone in ENV 1992-4: 1998 is probably specified for the case of water pressure acting on the structure. If there are puddles on the floor slab, even a concrete compression zone a great deal thinner is considered watertight.) Steel reinforcement in the compression zone due to bending may have such a restraining effect that no concrete compression zone is formed, so the concrete cracks because of shrinkage. This partly explains why the authors observed through-cracks at the location of maximum moments.

Nevertheless, steel reinforcement in the compression zone can be considered favourable in respect of crack formation with more cracks of smaller width being formed instead of fewer cracks of greater width.

6.5 Post-tensioning

Although no post-tensioned cast-in-place flat slabs have been built in Hungary yet, this structure is widely applied abroad for parking structures and other buildings, too. The main advantages of such structures are the reduction in the dead-weight of the slab and the height of building elements (staircases, ramps, facades) by reducing slab thickness, and increasing punching shear resistance. By applying post-tensioning the camber of the floor slabs is also provided. Moreover, crack formation (including the occurrence of through-cracks) in service situations may be prevented via the compressive force and bending moment due to prestressing. For the foregoing appropriate structural connections are needed in order to eliminate the restraining effects described earlier (especially for the period of tensioning), so that prestressing forces will not be supported by diaphragm walls and stiffening structures but by floor slabs instead.

It is highly desirable that such structures should be built in Hungary in the future.

7. CONCLUSIONS

No method has yet been developed for the prevention of through-cracks formation in spacious floor slabs of basement car parks, which can be said to be effective in all respects. This true both in Hungary and internationally. The main reasons of the occurrence of such cracks, (which are rather problematic concerning water-tightness and corrosion), are hydration heat and concrete shrinkage. Software applying the finite element method for the analysis of floor slabs is unable to take these effects into account. The risk of the occurrence of through-cracks may be greatly reduced by favourable structural design (the application of a structural wall separate from the diaphragm wall, proper location of stiffening cores and ramps and application of movement joints, etc.), and by designing concrete mixture, casting and curing more carefully than usually. Harmful effect of cracks may be significantly reduced by applying floor covering of higher quality (and thus more costly than those usually used), fitted using flexible adhesive or placed on a concrete base with movement joints capable of tolerating displacements of the floor slab.

8. NOTATION

h overall thickness of concrete section (mm)
f_{ct} tensile strength of concrete (N/mm^2)
ΔT temperature difference (°C)
α_{cT} coefficient of thermal expansion of concrete (1/°C)
ε_{ctu} ultimate tensile strain of concrete (%)
ε_{cs} shrinkage strain of concrete (%)
$\Delta\varepsilon_{ct}$ fluctuation of tensile strain of concrete (%)
σ_c concrete stress (N/mm^2)
σ_{cs} concrete stress due to restrained shrinkage (N/mm^2)
σ_{cT} concrete stress due to heat (N/mm^2)

9. ACKNOWLEDGEMENT

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HISTORY OF REPAIR OF CONCRETE AND REINFORCED CONCRETE STRUCTURES

Dr. Károly Kovács

Concrete and reinforced concrete structures often suffer damage due to environmental effects. Owing to the deterioration of the material the need for repair often arises. For a long time, damaged RC elements were repaired in accordance with rules of the so-called masonry and cast stone industry. Current repair principles require that repaired sections should be integral with the structural behaviour. These requirements need accurate diagnostics, good surface preparation, enhanced adhesion and an adequate application method. The aforementioned have been elaborated with scientific care, utilising the findings of up-to-date material science, the results of the plastics industry and the experiences gathered during the manufacturing of artificial fibres, etc.

Keywords: causes of damages, surface preparation methods, repair materials, repair deficiencies

1. INTRODUCTION

Structures made of concrete or reinforced concrete are artificial stone. (For the sake of simplicity I shall use the term reinforced concrete, or RC.)

In the shaping of structures, concrete has advantages over natural stone as a material due to casting methods in formwork and the use of other relevant forming techniques. Concrete is therefore very tolerant and can serve the ideas of the designer with regard to practical form and aesthetics and can also meet the increased static and dynamic requirements of modern industry. The following enumerates the effect of these advantages on the stability of concrete structures with respect to material science:

- A higher energy level of any given material or structure causes a greater loss of energy to the environment which in turn causes degradation. The corrosion of the steel in RC and the degradation of the concrete matrix itself arises through such causes (law of entropy).
- As a consequence of the material's capacity to be formed, the structural material obtains a texture which establishes a strong interactive link with the surrounding environment (such as the exchange of liquids and gases).
- Due to complicated formal and static patterns, problems can arise during the use of the structure leading to the loss of continuity (fissures).
- It is becomes more difficult to assess the maximum values of higher intensive loads which has led to overloads and a greater frequency of total structural failure.
- As a construction material, reinforced concrete was developed in an era when the level of environmental contamination was low. More recently environmental conditions have changed this situation significantly and failures can be traced back to these causes. Therefore, either effective protection of structures or a significant modification and their material structure is indicated in addition to improved repair techniques.

Owing to such causes, RC repair plays an increasingly important role in the prolongation of the service life of RC structures.

In this article I deal only with the recent history of the repair of modern RC structures. The material and structural characteristics of the Roman and medieval cement-bound structures are basically different from the structures of today and are not considered here. In any case the technical literature concerning these materials is rather incomplete.

The article relates mainly to the question of materials and does not cover procedures related to strengthening methods. Thus it deals only with the history specific to the knowledge needed for restoring structures to their original state.

2. THE CAUSE OF DAMAGE TO REINFORCED CONCRETE STRUCTURES AND THEIR APPEARANCE

Brux (1978) summed up the causes of damage to RC as follows. I attach also notes upon their probable outward form and appearance.

- in the course of the carbonation, the atmosphere's carbon dioxide transforms the lime and the silicates of the cement matrix, the pH value drops, the reinforcement corrodes. Consequence: fissures and loosening on the surface, the cross section of the reinforcement decreases.
- intrusion of aggressive materials (sulphates, salts, chlorides/acids, oil, etc.). Consequence: corrosion of the cement matrix and/or the corrosion of the reinforcement, its cross section decreases.
- mechanic abrasion (erosion due to use, or to natural agents). Consequence: uneven surface, decrease of the concrete cover
- atmospheric damages (frost, thermal shock → abrupt change of temperature, damages due to storm). Consequence: peeling, loosening, fissures.
- thermal stress, creep, shrinking, etc. Consequence: cracking.
- overload (rearrangement of loads). Consequence: cracking, rupture.
- damages caused by shocks and accidents (vehicles, explosion, etc.). Consequence: cracking, rupture, scuffing, failure of the reinforcement.
- consequences of damages caused by fire: peeling of the surface, failure of the cement matrix, fissures, elongation

of the reinforcement, effects of carbon dioxides, chlorine gas in presence of PVC, later arising of hydrochloric acid.
- consequences caused by poor workmanship in concreting: pockets, uncompacted concrete, insufficient concrete cover, failures of the formworks, early striking, weak cement matrix caused by faulty maturing, fissures, disruptions.

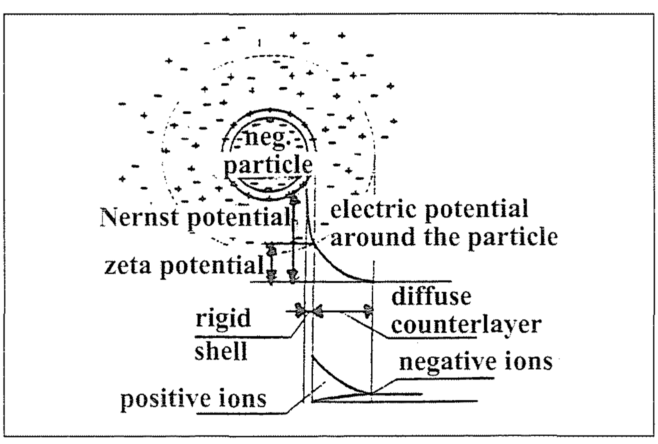
The outward form of these failures can be grouped as follows:
- active and inactive cracks.
- shortages of the concrete cover and cracks caused by this.
- breaking of edges and corners.
- pockets and coarse pores, impure surface.
- rust seepage.
- wash-out of lime, drip cups.
- other discoloration
- disruption of the reinforcement, distortions of the structure due to excessive deformation, unfavourable distribution of forces caused by the failures.

3. HISTORY OF REPAIR OF REINFORCED CONCRETE

We noted in the previous sections that the repair of the RC is an important factor in the service life of a structure and is consequently not only a question of aesthetic character. As can be seen, even in during the early period of the concrete's use, serious failures can occur so that the repair process should commence relatively early.

In earlier periods concrete blocks were repaired by use of so-called cast stone methods. The cast stone trade worked according to a set of severe rules which dictated the treatment of repair work. Strictly speaking, the cast stone trade emerged from the repair methods employed in the case of the natural stones. In the course of the repair of natural stone, the damaged part of the stone was cut out or chiselled into such a form to enable the replacement piece of stone of similar structure to be inserted. The old and the new pieces were then mortar jointed. To ensure a long-lasting and strong joint, the two parts were fastened together with metal pins. From the very early times, agents for increasing adhesion and for limiting shrinkage were added to the mortar. Such additives included cattle or ox blood and egg. In the cast-stone-method a formwork was arranged round of the concrete's cutout section and the replacement material was cast in. The fabric of the repair material copied the fabric of the original cast stone with the surfaces being lapped or picked together. Thus, the repair method

Fig. 1 *Development of the electrical double-layer around the particle*



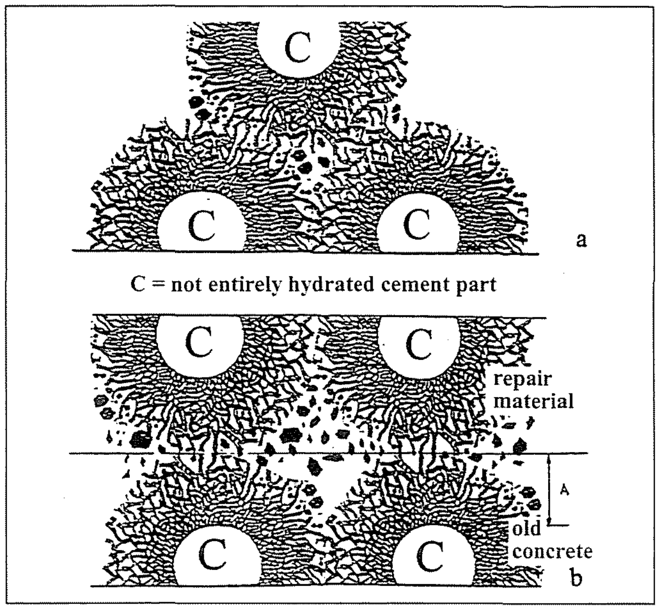


Fig. 2 Sketch of the joint of a silicate frame, a) in a state when they are not completely grown together, b) their principal scheme

for concrete was established in a similar way. Because the cementitious binder of concrete requires other adhesion masses than the natural stone, materials were sought which were compatible with the cementitious binding. Ground flour of freshly burnt brick, fine limestone dust (and incidentally) gypsum were often used.

The theory of silicate bonding was established by Derjaugin-Landau-Verwey-Overbeck and is connected with the Nernst's theory of the Zeta potential and of the electric double layer. The Zeta potential acts to keep the particles separate from other particles in close proximity. Their arrangement is shown in *Fig. 1.*

The relatively weak joint is based on spherical effects and, due to Bier (1988) it can be characterised by the arrangement displayed in *Fig. 2.* The energy of the bond is weakened further in circumstances where in the course of setting of the cement, lime is released within the cement matrix. This lime is dissolved in the capillary liquid always present. Because the interior of the cement matrix consists of a quasi-adiabatic environment, the temperature rises during solidification. This heat pushes the liquid containing the dissolved lime toward the surface. At or just before the surface the liquid evaporates and the lime deposits. The lime then carbonates due to the air's carbon dioxide. This very thin but at the same time tight and decomposing layer acts on the surface of the concrete as a dividing element (as if is caked in flour). This deposit prevents the adhesion of the following concrete or mortar layer. It is easy to understand that a load-bearing structural joint is not possible to attain given these conditions, Cement bulletin (1980).

This kind of repair method persisted until the 1950's and 60's. In this period serious efforts were made to understand the relationships concerning the bonded concrete surfaces. The reasons for theses investigations were as follows:
- To extend the application of stressed concrete structures, for which the old repair method was unsuitable.
- International agreements prescribed the assurance of "black road surfaces" i.e. ice-free surfaces are required on main roads in winter, which led to de-icing technology using predominantly salts. The Concrete related to traffic infrastructure were prone to salt contaminated within distances of 100–200 m. This led to heavy damage both to reinforcement as well as the concrete itself.

- Industrial development and consequently heavier traffic tended to increase contamination of the environment which in turn has led to both greater pollution of the atmospheric and that of natural water courses.

As RC structural failures became wide-spread and repair continued using the traditional methods of masonry, their ineffectiveness became apparent due to the subsequently short life-span of repaired RC elements and objects. This indicated that under significant stress, repaired elements deteriorated rather quickly and load transmission characteristics did not improve. The first proof of this was visible surface detachment and consequently materials were sought with increased adhesive attributes. Clearly, only materials which could be used easily on concrete surfaces of wet systems could be taken into consideration,

The aqueous dispersion of PVAc (short for polyvinyl acetate), revealed itself as a promising material. This repair material ensured a greater adhesive strength than the cohesive energy of most concrete of average quality. The research indicated that the material never failed on the surfaces repaired.

Aqueous PVAc dispersion was first employed as a cementing agent for paints, glues and special adhesives for tiles, or as their repair material. The effect of the polymer dispersion method presented itself in the fact that in water, dispersed synthetic drops are absorbed in the pores of old concrete under the effect of the movement of the embedding water. This process is presented in sketch of *Fig. 3*.

Aqueous polymer dispersion endowed the repair materials with further advantageous properties. These included the improved relationship between bending-tensile and compressive strength. It is known that one of the disadvantages of lithoidal rigid materials is that the relation between their compressive and bending-tensile strengths is high, 8–12:1. For concretes of medium strength the relation is about 10:1.

Through a reasonable addition of 5–15% cement to the polymer dispersions the tensile strength can be improved. However, the compressive strength is reduced only slightly; hence the ratio changes to a more favourable relation, e.g. 3:1. In this way we can get a tougher and more flexible material.

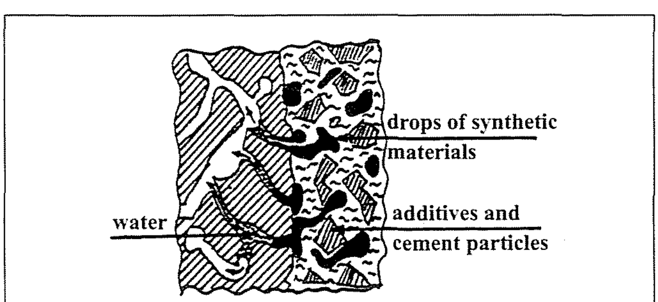
For a short time this material produced excellent results. But problems soon appeared. In a basic environment the PVAc saponifies and as a consequence, the adhesive property decreases. As a consequence a lot of damage occurred in this period, the 1970's.

The damaged concrete surfaces were treated with high quality epoxy and polyester resins, partly as patches and partly as continuous coating. The application was based on the fact that the resins adhered excellently to the dry concrete surface.

Only pure resins were applied because the resins of the day were not compatible with the aqueous cement binders. In such cases two important problems were to be solved:

- The concrete surface was to be dried out and it was to be ensured that up to the end of the setting, it should not become wetted again from the concrete side.
- Because resins are viscously flowing materials, the mortar and concrete made of them is not able to stay on sloped, vertical or overhead surfaces as it drops down. So thixotrope agents were to be added. This was often a difficult task.

Fig. 3 Anchoring effect of the polymer dispersions



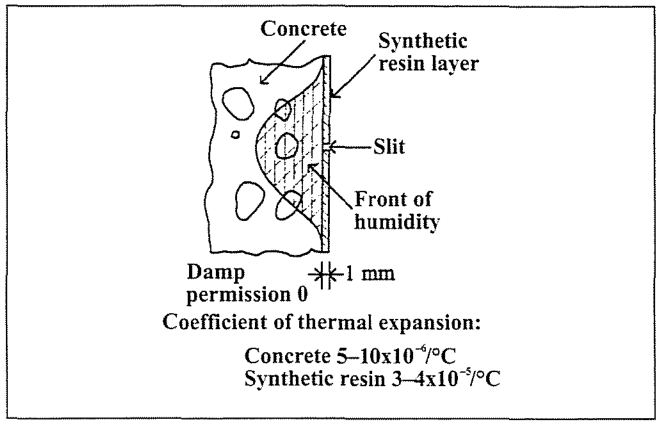


Fig. 4 The water front spreads below the damaged resin layer, it can not dry out

But failures of this PC (Polymer Concrete) system also soon appeared:

- Both the mortar resin and the coat were impermeable to damp, therefore the vapour, migrating from the concrete side, condensed on the boundary surface. The interface became saturated with water, causing a lot of problems (e.g. freezing in winter, scaling).
- In spite of the silicate additives, the resin's coefficient of thermal expansion is about one order of magnitude higher than of the base concrete. Therefore shear stresses appeared on the boundary surface, leading quickly to total failure.
- The resins are ageing and consequently their surfaces are shrinking. Sometimes they crack and scale from the concrete.
- Due to their brittle behaviour, the coatings can not follow deformations of the structure (they crack). More recently times elastic resins have been developed.
- Exposed to mechanical actions, the coatings damage easily, the water intrudes via capillaries but later it can not escape through the cracks. (Kovács, 1993) (*Fig. 4.*).

Due to these failures in the 1980's the use of the so-called PC repair materials became rare for cases of concrete exposed to weathering. In the meantime the research was aimed at the development of non-saponifying, resin-bonded repair materials of watery dispersion (PCC, Polymer Cement Concrete systems). Monomers, type VAc have been polymerised with other monomers, and in this way different co- and terpolymers came into being. These have endured sufficiently and steadily in the concrete's basic environment.

Furthermore, other polymers have been produced for this purpose (acrylates, acrylic nitril-butadiene-styrene, etc.). The characteristics of this PCC system can be modified through a wide range. In the meantime concrete repair developed into a specialised, large-scale industry. In order to ensure a steady material quality, the repair materials were prepared in a dry state and in such a way that for the site use it was necessary only to add water. To this the so-called 'redis', the re-dispersible synthetic dispersions and fluxes were to be elaborated. However, this fact limited the range of usable polymers but due to the manufacturing advantages, these systems are used almost exclusively at present (Schulze, 1991).

According to the technology, the repair materials themselves have also become specialised. The use the mechanised technologies are gaining ground because through their use adequate building technologies could be employed (guniting). Today a repair material is a mixture of a series of harmoniously co-operating chemical products. Still the simplest product consists of the following types of materials: cement, aggregate fractions, silica powder (SF, Silica Fume), flux, polymer dispersion, anti-foam additive, superficial humidifier, accelerator, etc.

The use of SF or microsilica started in the eighties because it reacts under the influence of cement and promotes the internal stability and density of the structure and results in increased adhesion. Using SF powders such technological solutions came into being where the repair materials rendered increased strength and their adhesion to the substrate also became greater.

The seventies and eighties saw the start of intensive activity which dealt with the connection of the substrate and the repair material, the method of the substrate's preparation, the harmonious connection of the repair materials and of the substrate and with the harmonising of their strength and behaviour. The principles of this work are the following:

- The strength of the substrate should reach at least C16 concrete quality class (let us mention that in the NAD of prEN206/DIN the class C30/37 is required for industrial floors), where the adhesion to the surface measured by bond-test procedure should be at least 1,5 N/mm^2.

If this condition is not be fulfilled, the adhesion to the surface is to be improved by special methods:

- The porosity of the surface is to be 'opened', i.e. to achieve sufficient adhesion; it is to be opened by the use of some 'cleaning' procedure. (Sand- or shot blasting, treating with water jet, milling, chiselling, etc.).
- On the base of the surface diagnostics the concrete is to be opened up to a prescribed depth. Where the concrete or the reinforcement is endangered by contaminating agents they are to be removed, usually by milling.
- For the purpose of repair the concrete is to be pre-treated, making it more suitable for the application of the repair material.
- Neither the strength nor the modulus of elasticity may be significantly higher than the characteristics of the substrate, because in such cases a long-term co-operation is not possible. The repair material must be more flexible than the substrate to enable it to follow its movements and to bridge over the microcracks.

Therefore the value of the strength of early (high-strength) material has been reduced to a half for increased flexibility.

- The set must take place quickly and the repair material should not require an intensive curing period because this is problematic in the case of thin layers. For such cases hardening accelerators are used and also the High Alumina Cements (HAC) (cement fondue) is re-applied.
- The repair system is to include a moisture repellent surface barrier which prevents repeated contamination later but which does not limit vapour movements at the same time.
- The repair material shall not change the structure's vapour and heat system. Any change of the vapour management's circumstances produces so-called local macroscopic aerating cells in the reinforcement, which accelerate the reinforcement's corrosion in the unrepaired sections. For such reasons, the separate treatment of the reinforcement in repaired sections is unfavourable.
- Both the thermal expansion and thermal conduction of repair material and of substrate shall be similar to each other.

For the repair materials it is rather difficult to fulfil these requirements completely. Therefore, these materials are mostly multiple composites, i.e. beside a concrete-like composition they have also plastic-like components. More and more materials also included fibre-components in the nineties. Fibres play a role in reducing the tendency to crack.

The synthetic fibres prevent early cracking after the repair. Steel fibres are also load-bearing elements. In case of point-type dynamic loads their role is important. For such reasons, synthetic and steel fibres are used together. The dimensions of the fibres are to be adjusted very carefully to the layer's thickness.

Finally, the complexity of recent repairs is to be mentioned where the preparative measures play a very important role. This is especially when the reinforcement is extremely contaminated and the removal of the contamination is of utmost importance. It is advisable to carry out this procedure in a non-destructive way where the removal of the entire surface is not possible anyway.

This method is applicable when the distribution of the contamination is relatively regular and the unit of the structure can be identified. The principle of the method is electrolysis. The circumstance is utilised where the reinforcement is completely coupled within the structural element. For establishing an electrical coupling, the reinforcement is to be freed in one or two places.

Using isolating spacers, a thin steel net is attached to the surface of the structure. This net is embedded in a conducting sludge or paste. Coupling the reinforcement as a cathode and the steel net as anode and using a direct current, the contaminating anions are easily removed from the concrete. Following this the auxiliary armatures can be removed from the surface. This method is very useful for the removal of salty contamination from bridge decks and from objects on the seashore.

Also other methods are used for the decontamination. In such cases absorptive, ion exchanging or other chemical packing is used on the concrete's surface (re-alkalisation, chloride-removal).

After this phase, repair works can be carried out (water jetting, concrete implantation, surface barrier).

4. CONCLUSIONS

The exciting history of concrete repair goes back many decades. At the beginning some faulty detours such as the rules of masonry and the cast stone industry were followed. These methods were not able to produce enough load transmission and the loaded joints became loose and often failed. In accordance with the actual state of the plastics industry, repair materials were applied on concrete surfaces, but their physical, chemical and rheological behaviour were not compatible to concrete.

In the seventies and eighties serious efforts were made in elaborating the theory of repair work. During these years many principles were established which enabled long-term bonds between the repaired concrete and the repair material to be produced. Equipment for preparation of the surface and proper methods were developed into a separate trade. Since the eighties, depending on the type and dimensions of the deterioration, repair materials and their application methods have been available for a wide and varied range of problems. Resolving problems entirely is typically complex.

Successful repair of concrete structures needs the following steps:

diagnostics → determination of the repair method → (first surface preparation) → chemical–electroosmotic treatment → final surface preparation → repair of the concrete and replacement of reinforcement → protection of surface to prevent further deterioration.

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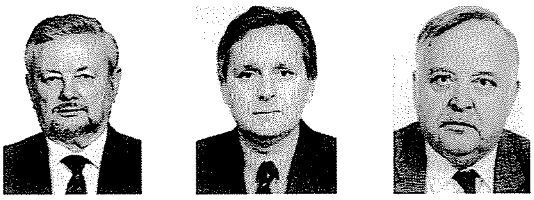
STRENGTHENING OF A 2000 WAGON CAPACITY REINFORCED CONCRETE GRAIN SILO LOCATED AT MARCALI IN HUNGARY



Prof. Árpád Orosz – György Csató – Dr. János Tamáska

The 20,000-ton reinforced concrete grain silo was completed in 1975. Imperfections were apparent during construction and after several years of operation vertical cracks appeared. During heavy rainstorms water ingressed into the cracks and seeped through to the cells, causing damage to the stored material. Due to the corrosion of the reinforcing steel, separation of the protective cover layer and the limitations that had to be placed on the operations of the silo, eventually a complete renovation could not be postponed. After analysing and comparing various strengthening techniques it was decided to employ a wire mesh covered with fiber reinforced shotcrete. Use of this single technology would provide corrosion protection, structural integrity and a new aesthetically pleasing surface. To prevent or limit the appearance of cracks the silo was filled with grain during the application of the shotcrete. This essentially eliminated all tensile stresses in the outer cover. A protective cover was specified that would bridge the cracks and would not prevent the escape of moisture and which could also protect the surface. Since completion of the strengthening the silo has been in continuous operation, proving the effectiveness of this method.

Keywords: silo, strengthening, maintenance, shotcrete, fibre reinforced concrete (FRC), corrosion

1. PRECEDENTS

The engineering plans of the grain silo at Marcali were completed in 1970-1971 at IPARTERV Inc.

The reinforced concrete silo is classified as a 2000 wagon capacity silo. This is characterised by the general arrangements illustrated in *Fig. 1,* where it is shown that the structure consists of 16 circular-shaped cells, each 7.5 m in diameter. The machinery takes up the space of two cells located internally in the structure. The internal spaces between the cells are also used for storage, except those adjacent to the machinery. In the space reserved for the machinery, the staircase, an elevator and ventilation ducts can be found. The height of the structure is 48.7 m. The area between the conical hoppers located at 6.5 m and the cover plate located at 41 m is available for storage *(Fig. 2)*. The plan called for a slipform construction method with cell wall thickness of 180 mm; the determining factor was protection against rain. At the time of design it was anticipated that hairline cracks could appear during construction and for this reason a surface protection material was specified. The brand name of this material was Elasztolen and it was chosen for its ability to bridge the cracks and also for its availability.

Fig. 1 Horizontal section of the silo

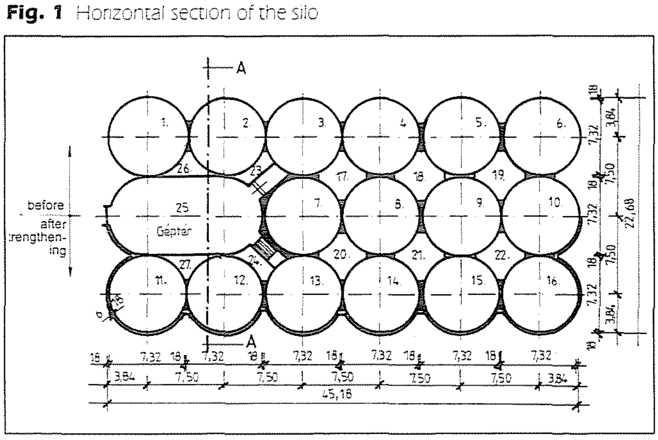
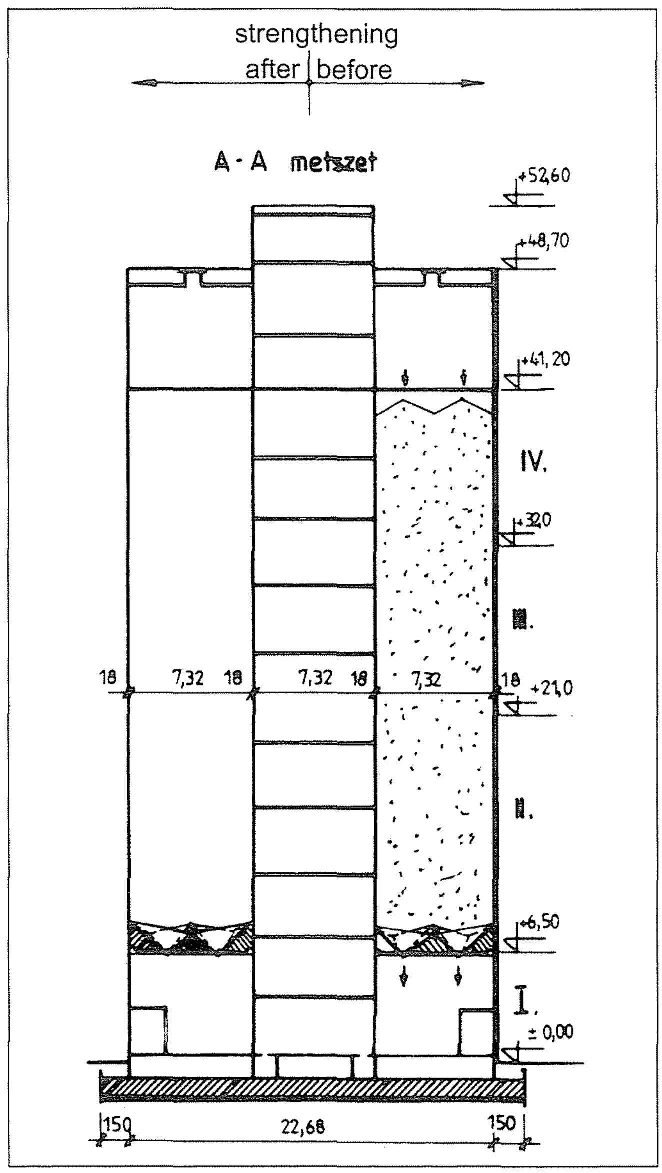


Fig. 2 Vertical section of the silo



During the construction in 1975, significant faults resulted which were mostly due to the non-adherence to the specifications relating to the use of the slipform technology. These can be summarised as follows:

- Placement of concrete was not to specification.
- Extensive pauses during raising of the formwork resulted in spalling and horizontal cracks. The resulting uneven surface was repaired with a cement-based mortar.
- The placement of the reinforcement was inaccurate. Instead of the Ø10/200 mm horizontal reinforcement that was called for in the specifications, an average of ϕ10/245 was used in the cell walls.

The advisory paper prepared at the Technical University of Budapest in 1975 indicated that due to the deficiencies observed during the construction, vertical cracks were likely to result. This prediction was borne out in subsequent years. The periodic inspection records of the silo document the slow deterioration of the structure. The Elasztolen surface protection layer applied to the outer walls did not meet the expectations and the material was unable to bridge or cover the cracks that appeared. Rain was able to penetrate through the cracks and damage the stored grain. In order to reduce the additional pressure that occurs when the stored material is evacuated, a method of removing the grain through the intermediary cells was originally planned. This method was utilised since measurements taken at Hódmezővásárhely conclusively proved that emptying the silo through the intermediary cells resulted only in the usual pressure due to storage. There was no additional pressure caused by the emptying of the stored material (Orosz and Simurda, 1985). However, there was a departure from the original plan and a pipe system attached to the outer silo walls was constructed. These pipes attempted to simulate the method of emptying the silo through the intermediary cells. Measurements of crack width were subsequently taken in 1987 by the Technical University of Budapest to evaluate the effectiveness of these emptying pipes. The measurements verified that a 50% reduction in pressure could be achieved compared to emptying the silo through four openings. The effectiveness in reducing pressure falls short, however, compared to when the silo is emptied through the intermediate cells (Orosz and Simurda, 1986). As time went by the condition of the emptying pipes deteriorated and the silo fell into disrepair. The danger caused by inadequate reinforcement and subsequent corrosion resulted in the curtailment of the silo's operation. Limited operation was mandated in the 1993 inspection report which was accompanied by recommendations either to reinforce the silo by applying shotcrete externally or by constructing an internal coat by slip-forming or the application of shotcrete (Herkó, 1993). The repetitive damage to the grain due to water seepage and the operational limits placed on the silo added urgency to completing the renovation and an additional inspection was performed in 1997 (Zsoldos, Csathó). Core sampling and 'Schmidt' hammer examination of the concrete confirmed that the strength of the concrete was adequate and that the reinforcement did not show signs of extensive corrosion. Therefore, corrosion did not have to be taken into account in the structural calculations. The recommendations included unloading the silo using the intermediate cells, repair of all surface imperfections and the application of a waterproof elastic cover layer with an ability to bridge the surface cracks. A change of technology regarding the removal of the material was also recommended. These recommendations impacted the economics as well as the operating characteristics of the silo. For this reason Concordia Inc, the owner of the silo, required a re-evaluation of all previous inspection results and recommendations.

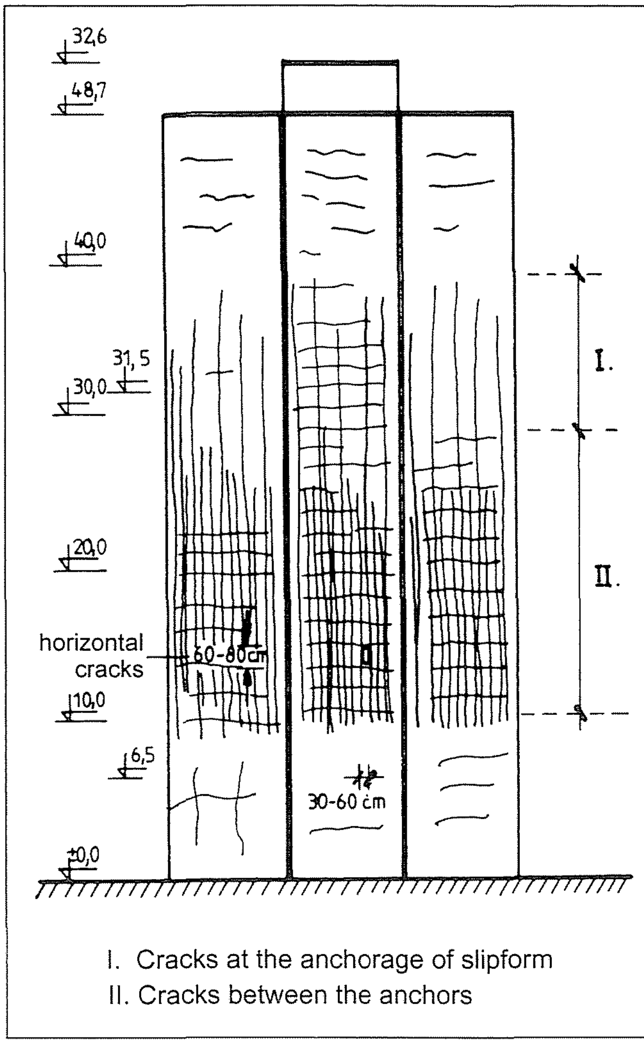


Fig. 3 Characteristic cracks in the silo walls

2. METHODS OF STRENGHTENING A REINFORCED CONCRETE SILO

Typical construction defects and methods of correction are discussed in detail by Farkas & Dalmy (1999). Therefore, only the most important points will be re-iterated herein.

Typical construction defects.

Upon examining the reinforced concrete silo, the following was found *(Fig. 3)*:

- Vertical cracks appeared 0.3 - 0.8 m apart due to actual pressure exceeding the calculated pressure.
- Horizontal cracks and spalling was found 0.6 -0.8 m apart due to improper raising of the slipform.
- Often less reinforcement was found to be placed than was called for in the design.
- The density and the strength of concrete generally did not meet specifications.

The vertical and horizontal cracks are generally propagated across the full wall cross section, allowing water (rain) into the silo cells. The cracks first appeared at the location of the anchors of the slipform and, in later years, in the lower third of the silo between the slipform anchors. The crack width generally exceeded 0.3 mm, often 1–2 mm, and there were instances exceeding 5 mm. The latter case could result in the corrosion of the reinforcement.

Strengthening using a constructed internal shell:
For situations where the absence of adequate reinforcement is a

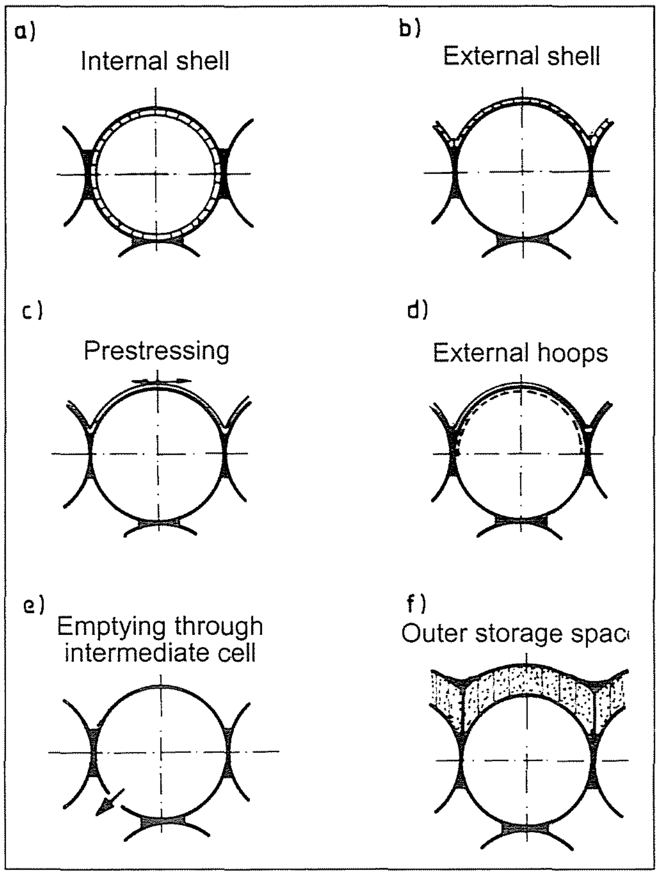


Fig. 4 Methods of strengthening

significant factor and the structural deterioration is extensive, the construction of an 80-120 mm internal shell cover is appropriate *(Fig. 4a)*. Several silos were stiffened this way, firstly one at Csorna followed by silos at Kiskunhalas and Miskolc. Designs for these repairs were completed by Dezső Herkó and slipforming was used to erect the internal shell. Farkas and Dalmy described the construction of an internal shell using shotcrete. The external repairs were a separate task in this case.

Strengthening using an external cover shell:
One of the advantages of repair completed with shotcrete applied over reinforcement is that both the strengthening and corrosion protection issues are solved simultaneously *(Fig. 4b)*. Special attention must be given to the tensile forces of the outer layer at the intersection of cell walls. These forces must be anchored in a reliable manner (Collins, 1997), (Farkas and Dalmy, 1992).

Strengthening using outside prestressing:
An optimal solution to strengthening a silo as well as attaining a crack-free structure can be achieved by using properly anchored prestressed and corrosion protected tensile tendons. The method is effective when the concrete of the cell wall is of good quality. In case of marginal concrete strength the tensile stresses are reduced due to creep and shrinkage. The Freyssinet method of prestressing the outer cell walls was successfully applied at several silos (Farkas and Dalmy, 1999) *(Fig. 4c)*.

Strengthening with hoops:
In Yugoslavia, non-prestressed reinforcement was widely used to fabricate inner or outer hoops to reinforce a silo. The disadvantage of this method is a lack of corrosion protection of the silo and loosening as the result of the warmth of the sun *(Fig. 4d)*.

External and internal hoops, connected with bolts through the cell wall was designed and constructed for strengthening some silo cells (Herkó. 1997).

Emptying through intermediate cells:
Experiments abroad and in Hungary (Hódmezővásárhely, Orosz and Simurda, 1985 and 1986) verified that emptying a silo through the intermediary cells resulted only in the bin pressure due to the storage of the material while overpressure due to emptying is eliminated. If the strength of the cell wall is adequate to resist bin pressure, no additional strengthening is required. The method requires the alteration of the material transport system and machinery of the silo which is a disadvantage from an operational point of view *(Fig .4e)*.

Strengthening by constructing an outer storage space.
The capacity of the silo is substantially increased; the original outer cell walls will become internal cell walls. Both are an advantage of this method *(Fig. 4f)*. Dimensions must be chosen to assure that the new cells are supported by the existing foundations and the integrity of the structure has to be maintained.

Selecting the method of silo strengthening
A detailed static analysis of the silo and an examination of the current state of the structure are required in order to decide which of these strengthening methods outlined is optimal. The strength of the concrete of the cell walls, the quantity and condition of reinforcement are deciding factors. Another important consideration are the limits that need to be placed on the operation of the silo or whether the silo's operation can be maintained during the period of the construction work. Analysing the costs associated with each of these methods revealed no significant difference between them. Therefore, in our opinion the engineer is in a position to select the optimal solution for each situation.

3. THE RESULT OF THE STATIC ANALYSIS

The detailed static analysis was guided by the internal standards and specifications of Gabonatröszt Inc. and by the measurements taken by the Department of Reinforced Concrete Structures of the Technical University of Budapest (Orosz, 1997). The most important results are summarised below:
Data:
Weight of grain per unit volume: $\gamma = 9$ kN/m^3
Coefficient of internal friction for the material stored: $\phi = 30°$
Factor of lateral pressure: $\lambda = 0.6$
Factor of wall friction: $\mu = 0.4$
Internal diameter of the silo cell: $D_i = 7.32$ m
Wall thickness: $t = 0.18$ m
Limiting depth: $z_0 = D_i/4 \cdot \lambda \cdot \mu = 7.32/4 \cdot 0.6 \cdot 0.4 = 9.63$ m
Maximum pressure under storage:
Vertical pressure:

$$p^f_{v,max} = \gamma \cdot z_0 = 9 \cdot 7.63 = 68.6 \text{ kN/m}^2$$

Horizontal pressure:

$$p^f_{h,max} = \gamma \cdot z_0 \cdot \lambda = 68.6 \cdot 0.6 = 41.11 \text{ kN/m}^2$$

Vertical frictional pressure:

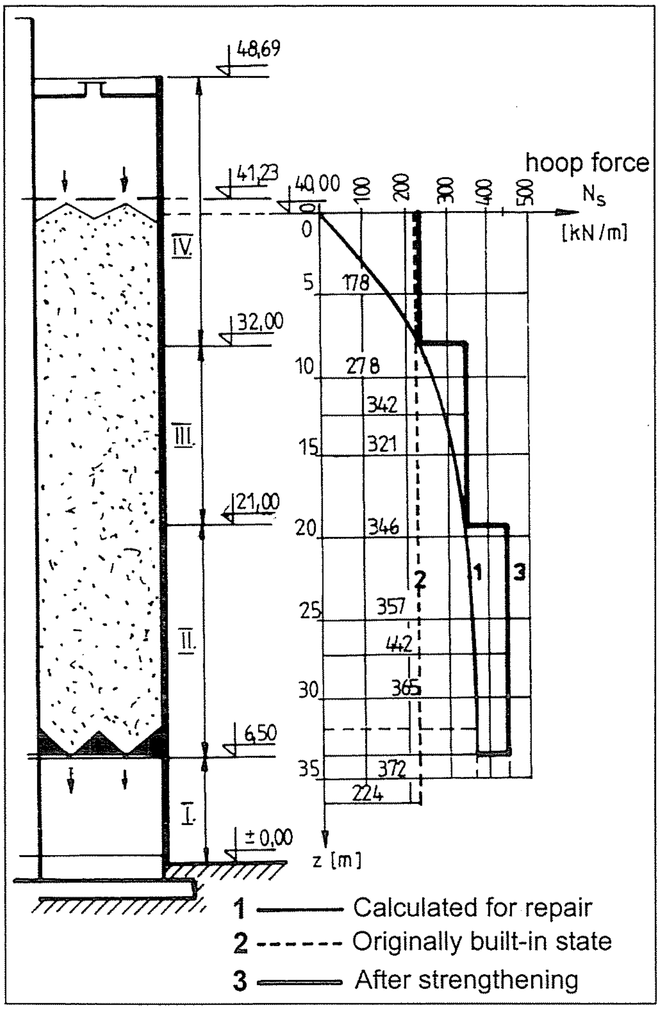


Fig. 5 Limit hoop-forces and ring-forces

$p^f_{s,max} = \gamma \cdot z_0 \cdot \lambda \cdot \mu = 41.11 \cdot 0.4 = 16.45$ kN/m²

Lateral pressures from emptying:
Emptying factors: $n_e = 1.4$ and $k_e = 1.24$

The pressure can be calculated at depth of z by the following formula:

$$p = p_{max} \cdot (1 - e^{-z/z_0})$$

The maximum horizontal pressure from emptying:

$$p^e_{h,max} = n_e \cdot k_e \cdot p^f_{h,max} = 1.4 \cdot 1.24 \cdot 41.11 = 71.1 \text{ kN/m}^2.$$

The calculated values of static pressure and emptying pressure are in good agreement with the measurements taken at the silo at Hódmezővásárhely and at Orosháza, these two silos being similar in size and in cell arrangements (Orosz and Simurda, 1978).

The tensile forces in the cell walls, calculated by the boiler-formula are:

$$F_h = n \cdot p^e_{h,max} \cdot R_i = 1.3 \cdot 71.1 \cdot 3.66 = 338.0 \text{ kN/m}$$

using a 1.3 factor of safety.

The effect of daily or seasonal temperature changes can be taken as 10% of the emptying pressure (Orosz, 1978), therefore the design lateral force is (Fig. 5 curve 1)

$$F_{h,d} = 338.0 + 34 = 372 \text{ kN/m}.$$

The limiting value of the hoop-force in case of using reinforcement specified in the design as B60.40 quality, in mesh of Ø10/200,
the area of reinforcement:

$$A_{s,d} = 785 \text{ mm}^2\text{/m}$$

the limit stress:

$$f_s = 350 \text{ N/mm}^2$$

$$F_{h,d,lim} = A_{s,d} \cdot f_s = 785 \cdot 350 = 275 \text{ kN/m}.$$

Calculating the hoop-force with reinforcement used in construction (as-built) Ø 10/245,

$$A_{s,b} = 641 \text{ mm}^2\text{/m}$$

$F_{h,b} = A_{s,b} \cdot f_s = 641 \cdot 350 = 224$ kN/m (Fig. 5 dotted line).

It can be seen that with regard to reinforcement, neither the designed nor the actually built structure can meet the design requirements.

The vertical distribution of hoop-forces as a function of the silo's height is presented in Fig. 5. Due to inadequate reinforcement and other shortcomings the silo must be strengthened. Considering the distribution of pressure, the outer walls can be divided into four regions. The surface imperfections could be repaired with a 30-40 mm shotcrete layer without reinforcement in region I and IV. For region II a welded wire mesh specified as Ø 8/100-200, for region III a welded wire mesh specified as Ø 6/100-200 will provide the necessary reinforcement. For both of these regions (II and III) a shotcrete layer 60-70 mm was specified.

Using reinforcement specified as B60.50 and considering the reinforcement already in place the hoop-force in region II was 224 kN/m.

The additional hoop-force allowed for the ϕ8/100 wire mesh is:

$A_{s,w,II} = 503$ mm²/m, and $f_{s,w} = 420$ N/mm²,

$$F_{h,w,II} = 503 \cdot 420 = 218.0 \text{ kN/m}.$$

The total limiting hoop-force:

$$F^{tot}_{h,II} = F_{h,b} + F_{h,w,II} = 224 + 218 = 442 \text{ kN/m}.$$

In region III using ϕ6/100-200 wire mesh the total limiting force is:

$$A_{s,w,III} = 282 \text{ mm}^2\text{/m},$$

$$F_{h,w,III} = 282 \cdot 420 = 118.0 \text{ kN/m}$$

$$F^{tot}_{h,III} = 224 + 118 = 342 \text{ kN/m}$$

(presented in Fig. 5, stepped continuous line no. 3).

Examination of Fig 5 reveals that the limiting hoop-forces exceeds the design hoop-force at all elevations. We mention here that the crack sensitivity factor recommended by Swedish researchers was considered, and the silo wall was found adequate, if

$$k = A_s / D_i^2 = 0.24$$

A_s = horizontal reinforcement [mm²/m]
D_i = internal diameter of the silo [m]

With the designed reinforcement: $k = 7.85/7.32^2 = 0.147 < 0.24$
With the as-built reinforcement: $k = 6.41/7.32^2 = 0.12 < 0.24$

Therefore neither the designed nor the as-built silo meet the condition.
After the strengthening the total horizontal reinforcement was

$A_s = 1{,}144$ mm²/m

The crack sensitivity factor is: $k = 11.44/7.32^2 = 0.214$,

which is lower by only 10% against the recommended value.
The idea of filling the silo with grain for the duration of shotcrete application was considered. By evacuating the material, hoop-forces and dynamic pressures can be induced and the outer cover can be constructed without tensile stresses.
This can be achieved if adequate reinforcement exists to take on the hoop-force caused by the emptying of the silo. The hoop-force in this case excludes the factor of safety.
In this instance, calculating with the actual weight of grain per unit volume:

$\gamma = 8$ kN/m³

The hoop-force is:

$F_{h,0} = (0.8/0.9)\cdot 71.1\cdot 3.66 = 231.0$ kN/m > 224 kN/m.

This exceeds by about 3% the as-built limit for the hoop-forces. In our judgement this short-term effect - for about 10 days during the curing of the concrete - presents an acceptable level of risk.

4. GUIDELINES FOR THE STRENGHTENING METHOD

The review of the documentation of all previous studies and the completion of a structural analysis resulted in the recommendation for strengthening the silo with an outer cover of shotcrete applied over a wire mesh (Orosz, 1997). Based on this recommendation, the owner of the silo engaged the Engineering firm of Zsoldos and Csató in 1998 to complete a detailed design for the strengthening of the silo.
The solution that was finally implemented was the one that met both the requirement for adequate structural strengthening and provided protection for the outer surface. The most important steps concerning the strengthening were:

- cleaning of the outer surface with dry sand blasting,
- the removal of loose concrete from the cell wall,
- corrosion protection of all exposed reinforcement,
- placement of the wire mesh reinforcement,
- filling the silo with grain prior to applying the shotcrete, the stored material staying in place for approximately 14 days,
- applying the shotcrete, creating a smooth surface,
- application of a surface protecting paint that is elastic, able to bridge the cracks which does not trap moisture, lets vapour diffusion and reflects sunshine.

The relatively thin 60-70 mm outer cover was used to assure that the coefficient of expansion of the old cell wall and the outer cover is compatible. A rigid structure would resist the hoop-forces and would likely cause cracks to appear.

5. CONSTRUCTION, INSPECTION AND QUALITY CONTROL

KÉV-METRÓ Inc. was selected to be the main contractor for the strengthening of the silo with some of the work subcontracted to Betonplasztika Inc. The owner, Concordia Inc. engaged Dr. Árpád Orosz, the first author who is a retired professor of the Budapest University of Technology, to assure quality and to conduct inspection of the ongoing work on their behalf.

5.1 Surface preparation

The original plan of wet sandblasting was abandoned in favour of dry sandblasting to prevent damage to the stored grain due water entering the silo cells through the cracks.
In some areas the removal of the original Elasztolen surface protection was difficult, in other areas it was easily removed. This confirmed that the Elasztolen surface protection was not applied uniformly in the silo surface. After completing the sandblasting an inspection of the cleaned-up surface revealed all the defects that were due to improper placement of the concrete at the time of construction. The cracked condition of the cell walls also became apparent. After sandblasting and chiselling off loose concrete all exposed reinforcement was covered with PROXAN Metallgrund brand corrosion protecting compound *(Photo 1)*.

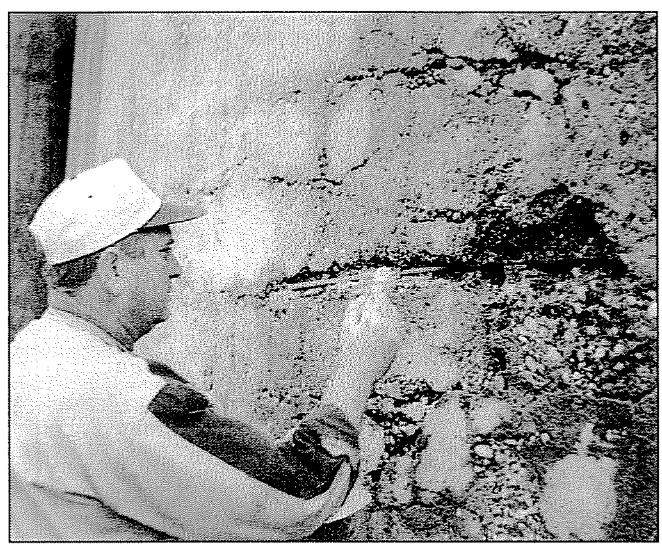


Photo 1 The wall of the silo after sand blasting and the corrosion protection of reinforcing bars

5.2 Examination of the existing concrete

Previously prepared studies of the silo considered in detail the strength of the concrete of the silo walls. In spite of the deficiencies and the existing cracks it was classified as C12 strength concrete. As part of this examination 50 mm diameter round core samples were obtained. These samples revealed that at the lower portion of the cell wall a tensile stress of 1.1 N/mm² value can be attained or even exceeded, while the upper parts of the wall (above 31.5 m) the average was 0.67 N/mm² with occasional 0 values. For this reason at the upper portion of the cell wall to increase adherence, additional reinforcing was used.

5.3 The characteristics of the shotcrete

5.3.1 Requirements

To achieve the desired strengthening of the silo the strength of the concrete used is a decisive factor. According to the plans, the concrete to be used must have strength of class C20, must be free of cracks or with a minimal sensitivity to cracks. These criteria are important, because:

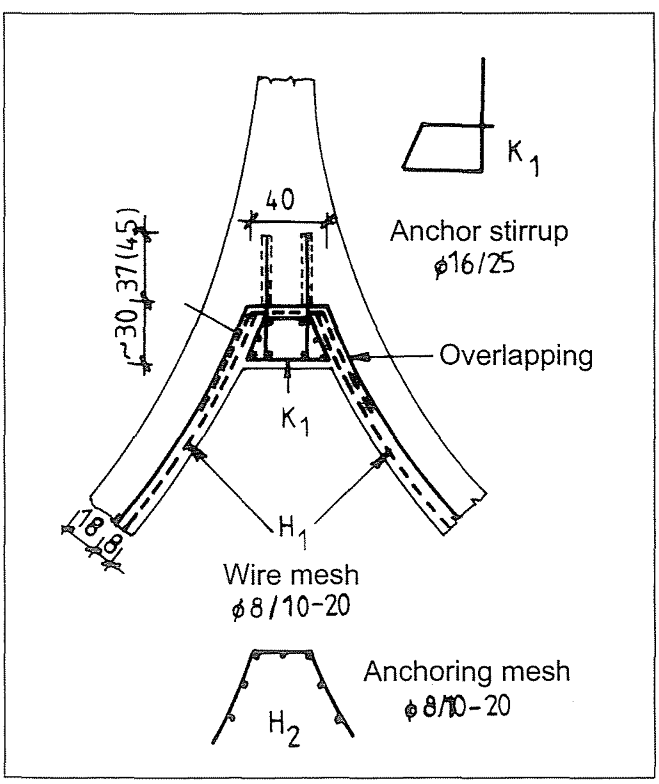
- The relatively thin, 50–60 mm thick reinforced shotcrete layer tends to dry quickly after application, resulting in shrinkage cracks.
- The 30–40 mm shotcrete layer without reinforcement is especially sensitive to drying too quickly considering the largest aggregate measures D= 0.8 mm.
- The existing concrete cell walls resist the shrinkage of the freshly applied shotcrete resulting in tensile stresses in the shotcrete.
- Sunshine or strong wind increases the rate of drying of the shotcrete; therefore proper curing of the shotcrete after application is especially important.

The situation calls for concrete with an above average plasticity in relation to the compressive strength. Fibre reinforced concrete is an excellent candidate for this application, therefore a ZENTRIFIX SB-04 concrete suitable for shotcrete, containing polypropylene (Fibrin) fibres as an additive was chosen.

5.3.2 The composition of the concrete

The composition of the concrete and the method of placement were controlled at the site and samples were taken to be examined in a laboratory. The 25 kg bags of ZENTRIFIC SB4 concrete were mixed with 25 kg, 0–1 mm washed sand from Barcs, 15 kg 4-8 mm sifted gravel and with 10 kg. Beremend cement

Fig. 6 Anchorage of wire mesh



CEM-I,52.5 (Pc45). Mixing in a relatively high cement ratio (~400kg/m³) resulted in a high strength shotcrete. The continuing inspection and testing of the concrete during the construction verified that a uniform concrete quality was achieved. This was due to the careful and precise mixing techniques employed at the site. Considering only the strength of the concrete, a lower cement ratio would have sufficed, but to assure the reliable adherence of the shotcrete and to reduce the quantity of concrete falling back the high cement ratio could be justified. The control of shrinkage was made possible by the addition of fibre reinforcement.

5.3.3 Strength testing of the shotcrete

The strength of the shotcrete was verified using three methods, as follows:

- On-site on a daily basis 70.7·70.7·70.7 mm sample cubes and 40·40·160 mm Hegermann prisms were taken and subjected to a compressive test in a laboratory.
- The Schmidt hammer examination, a non-destructive testing of the shotcrete placed on the cell walls.
- Circular shaped pucks were glued to 50 mm diameter cylindrical-shaped metal anchors embedded in the concrete. The pucks were pulled off to establish the tensile strength of the concrete.

5.3.4 The classification of the shotcrete

Based on the destructive material test of the core samples and on the examinations conducted at the site, it was concluded that the shotcrete:

- exceeded the planned strength of C20 value; a C30 classification was achieved.
- its quality was uniform, attesting to the high quality construction methods followed.

5.4 Assembly of the reinforcement, inspection of the wire mesh

Between levels of 6.05 and 31.05 metres the design prescribed the attachment of welded wire mesh reinforcement to the silo walls. Between levels of 6.5 m and 20 m, 8 mm diameter horizontal elements of wire mesh were placed 100 mm apart with the vertical elements placed 200 mm apart. Between levels of 20 m and 31.05 m, 6 mm diameter reinforcement was used and placed in a similar fashion.

5.4.1 Classification of the reinforcement

The design called for reinforcement of B60.50 classification. The on-site inspections revealed that the reinforcement used was the classification specified.

5.4.2 Examination of the anchoring of reinforcement

The wire mesh fastened to the outer surface of the silo is subjected to tensile stresses due to the silo load; therefore the reinforcement must be anchored at the joints formed where the silo cell walls intersect. A specially designed saddle shaped mandrel placed in a 400 mm deep hole served as the anchor. This was recommended by the designer, the great advantage of this solution is that during the attachment of the reinforcement labourers do not need access to the interior of the silo cells. This assures that during the assembly of reinforcement the silo can be continuously operated and the silo can be filled for the duration of shotcrete placement *(Fig. 6 and Photo 2).*

The significance of applying the shotcrete while the silo is full is apparent when considering that tensile stresses due to bin pressure will not be created in the shotcrete layer. When the silo is emptied compressive stresses will be present in the

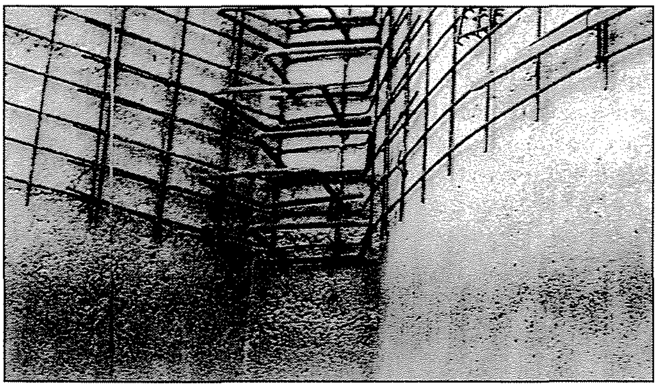


Photo 2 The anchorage and adjustment of wire mesh

shotcrete layer. The probability of cracks in the shotcrete layer due to repeated material movement is reduced.

The examination of the effectiveness of the anchoring mandrel assumed great significance. At the site, tests were conducted to establish the load bearing capability of the anchors. The purpose of the tests was to establish the force necessary to pull out the reinforcement from the anchor and to obtain data to compare the various compounds used to glue the anchors. There is a significant difference in price for these compounds.

The design called for the ability to resist a 50 kN pull-out force, as the reinforcement distribution was designed to resist 300 kN/m tensile force occurring when two adjacent cells are emptied simultaneously.

The tests revealed that the best results are obtained using an epoxy compound. The SIKA Pronto compound responded to the 50 kN force with a minor slip, the PROXAN repair mortar responded without or with minimal slip. The tests were conducted after a few days of curing and time will definitely improve on the results. After the test results were considered a decision was made to use the PROXAN mortar.

5.4.3 Attachment of the welded wire mesh

The wire mesh was fastened to anchors with bristled interiors embedded in holes drilled into the silo walls and cemented in place with the PROXAN mortar. Anchors were distributed in 500·500 mm raster.

The raster size was established to assure the proper stiffness of the wire mesh during the shotcrete application. A loose, flexible wire mesh would have significantly increased the quantity of shotcrete falling off.

The raster distribution of the anchors for the 6 mm and 8 mm reinforcement used is nearly identical with considering the required stiffening of the wire mesh, therefore the designed anchoring was used. The distance of the wire mesh reinforcement from the surface of the outer wall of the silo was set at 10 mm in the design. Spacers were used to assure the uniformity of the distance.

Experience has shown that with the given cell size the 8 mm reinforcement can be bent to follow the surface geometry, easing the assembly of the reinforcement. Using a wire mesh with reinforcement diameter exceeding 8 mm would have caused difficulties in bending the reinforcement to the proper shape.

5.5 Application of shotcrete, surface finish and treatment

Shotcreeting was carried out by ALIVA type machinery. An adhesive layer was placed on the wet and dull surface at the necessary locations prior to shotcrete application.

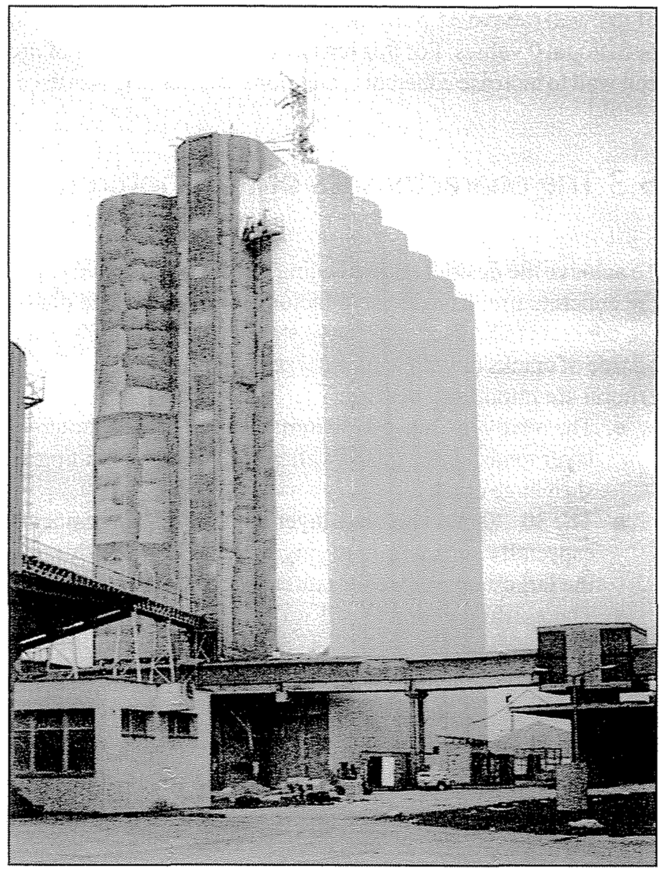


Photo 3 Base and cover painting of southern and eastern side

Creating a smooth concrete surface presents an aesthetically pleasing appearance and it is necessary for strong adherence of the surface protection painting.

5.6 Surface protection

The design called for the application of a surface protection layer to the shotcrete surface. The ECME Colour brand was chosen from the available products. First a white primer and then a beige covering layer was applied. *(Photo 3)*.

The adhesion of the paint was tested using the adhesive disks test method and it was determined that the surface adhesion on the average can be taken as 0.84 N/mm^2. Experience at the site has shown that the elasticity of the paint should be appropriate and the thickness of the paint layer adequate.

It is noted that surface protection is necessary, as the appearance of cracks must be anticipated for these reasons:

- The strengthening was carried out on an already cracked surface,
- the tensile stresses due to the stored material and
- the temperature and shrinkage effects all contribute to crack formation.

These observations are supported by the experiences encountered at a Canadian silo, where a few years after shotcrete application, vertical cracks appeared. The life expectancy of the applied surface protection layer can be estimated at 10 to 15 years. Periodic inspection of the condition of the silo surface is advisable after five to ten years *(Photo 3)*.

6. CONCLUSIONS

This paper deals with the strengthening of a grain silo in Marcali, Hungary.

The operating restrictions due to a reduction in load-bearing capacity, the cracked condition of the silo walls and the impaired condition of the silo surface protection allowing water into the silo cells, added urgency to and justified the strengthening of the silo.

There are several advantages of the strengthening method based on the consultant studies and the design of Zsoldos and Csató. The fibre reinforced shotcrete applied over a wire reinforcement mesh solved at once the challenge of structural strengthening, the need for repair to the outer surface of the silo, the requirement of corrosion protection and the prevention of water entering the silo cells. An aesthetically pleasing solution was achieved without the need to curtail operations or to make changes to the technology used in operating the silo. On the basis of the methodical inspections conducted and the quality control in place during the work in progress the following conclusions can draw:

The silo surface was properly cleaned and prepared by dry sandblasting. The shortcoming of this surface cleaning technology is counterbalanced by its advantage in avoiding water entry into the silo cells. The assembly and attachment of the welded wire mesh reinforcement was completed as planned. Holes were drilled to receive anchors that were secured in place by mortar. Any exposed reinforcement was treated with a corrosion-protecting compound. Regular site examination as well as laboratory testing of core samples assured the composition and the strength of the shotcrete. On the basis of this testing it was determined that the resulting shotcrete exceeded the designed C20 class and a C30 class was achieved. The high strength achieved was due to the higher than normal cement ratio used in mixing the shotcrete. Special attention was given to the preparation of the work area and to the necessary after-treatment of the shotcrete. This greatly contributed to the near elimination of hairline cracks in the silo wall. In preventing the appearance of cracks the advantage of polypropylene fibre reinforced shotcrete is conclusively proven when the shotcrete is applied in a thinner than average layer (Orosz, 1999). Creating a smooth surface ensured the aesthetically pleasing appearance of the silo.

A surface protection painting was selected with a moisture permeability property and with an ability to bridge hairline cracks. This paint was applied in base coat and then with a covering coat. Examinations provided an assurance that adhesion between the coats as well as adhesion to the concrete is adequate. A pleasing appearance resulted when the final colour coat was applied.

In conclusion, the strengthening and repair method employed is effective. It can be implemented in four months and it assures the uninterrupted operation of the silo for years to come.

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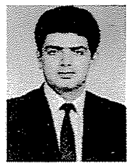
STRENGTHENING WITH CARBON FIBRES – HUNGARIAN EXPERIENCES

Prof. György L. Balázs – Mazen M. Almakt

New materials have been developed over the last decade for strengthening of structural concrete. They are materials which are non-corrosive, have high strength and are easy to handle owing to their low weight and minimal bulk. These materials are the fibre reinforced polymers, especially the carbon fibre reinforced polymers. Carbon fibres were first used in Hungary as a strengthening material in 1996, since then it has been applied to a wide range of structures or structural elements such as slabs, beams (by eliminating an internal support), natural stone balconies, silo ring beams, prestressed pre-tensioned bridge girders, slab openings and concrete truss girders. This paper aims to summarise the main characteristics of carbon fibre strengthening and to discuss major projects and applications in Hungary.

Keywords: strengthening, fibres, matrix, strip, fabric, CFR P, failure modes, durability

1. INTRODUCTION

The strengthening of any given structural member may be required *by an increase in loads or by a decrease in load bearing capacity*. The modification of a structural system may also induces higher sectional forces. Reduction of load bearing capacity can be caused by mechanical damage or by material deterioration. These include corrosion of reinforcement (e.g. due to inadequate concrete cover or attack by de-icing salts) or functional modification of the construction (e.g. opening of slabs for elevators or staircases).

Fibre reinforced polymers (FRP) became available for civil engineering purposes, especially for structural strengthening. The material composition of fibres can be *glass, aramid or carbon* (Balázs, 1996; Kollár-Kiss, 1998). The diameter of fibres is 8-10 ìm and their strength is 3000 to 5000 N/mm^2, exceeding considerably the strength of prestressing tendons. The elastic moduli and failure strains are different for different fibres. Elastic moduli of glass fibres and aramid fibres are lower than that of steel. Carbon fibres are available with moduli lower, equal or higher than that of steel. All these fibres follow Hooke's law and fail rigidly. Considering all strength attributes, fatigue strength and durability, *carbon fibres* seem to provide the most advantages characteristics from the three types of fibre for structural engineering purposes (Taerwe, 1995). Therefore only carbon fibres are dealt with in this paper.

Typical applications for carbon fibre strengthening technology include buildings, silos, reservoires, bridges, etc. Type of structural member can be practially any, i.e., *beam, slab, coloumn or wall.*

Material of the structural member to be strengthened can be all kinds of material which provide an appropriate anchorage of strengthening element by bond (generally without any mechanical anchoring device) such as:

- concrete, reinforced concrete, prestressed concrete
- natural stone
- steel
- timber and
- masonry.

Majority of strengthenings with carbon fibers was carried out on concrete members, however, examples are available for strengthenings on elements by the other materials mentioned above.

Strengthening strips can be applied *prestressed or non-prestressed*. Application of prestressed strips still provides practical difficulties and in-situ prestressed application is not yet widely known, even if tests are running in Switzerland, Germany and the UK. One possible solution for the application of prestressed strengthening strips is published in the reference Luke, Leeming and Skwarski (1998). However, in most cases prestressing of strengthening strips is not needed.

The advantages of strengthening with carbon fibres can be summarised as follows:

- Fibres are electrolitically non-corrosive
- They possess high strength
  - under short term loading
  - under long term loading and
  - under repeated loading
- Owing to low specific weight (i.e., 17-18 kN/m^3 depending on the fibre to matrix ratio) fibres:
  - have low transportation costs
  - are easy to apply even in restricted spaces
  - have no need for scaffolding to support the strengthening strips or fabrics during hardening of the adhesive
- They have no limitation in length
- Due to their thickness (1 to 1.4 mm) fibres:
  - Contribute to no significant decrease in structural height
  - Strips can be crossed
  - Applications are easy to cover, thus fulfilling aesthetic requirements
- economical solution (considering the entire cost of strengthening).

Nevertheless, there is also a list of possible disadvantages:

- Carbon fibres themselves are relatively expensive
- Mechanical protection is to be provided if vandalism is not excluded
- Service temperature in the vicinity of strengthening is not allowed to exceed around 50 °C owing to the embedding matrix of fibres.

Strengthening with externally bonded carbon fibre reinforcement provides an alternative to *steel plate bonding* and in some cases to strengthening with external prestressing.

Steel plate bonding was a widely used flexural strengthening method (Bódi-Farkas, 1995; DIB, 1995) that had some deficiencies. These included the limited length of strengthening plates, their heavy weight, the need for support scaffolding to steel plates during hardening of the adhesive and the deterioration of the bond between strengthened members and the steel plates owing to corrosion.

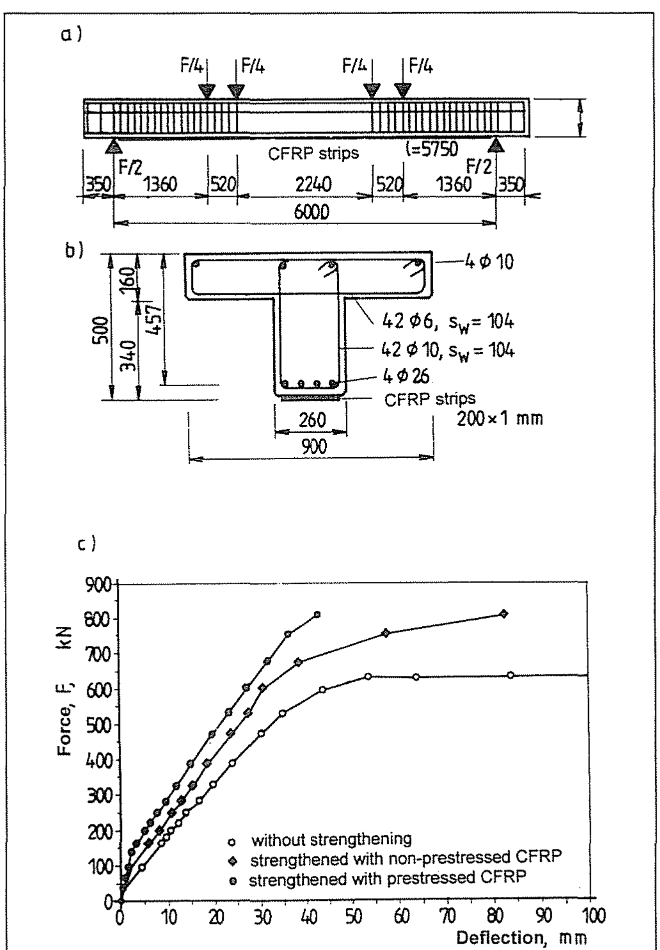
The main objectives of this paper are to review the characteristics of strengthening with externally bonded carbon fibre strips or fabrics and to summarise the applications in Hungary. This methodology is relatively new. However, it has potential to increase in importance if the promising initial experiences are a guide.

2. BASIC BEHAVIOUR

The application of externally bonded carbon fibre reinforcement in Europe goes back to the beginning of the 1990's. There were also numerous applications in Japan and North-America (JCI, 1997).

Tests were started in 1987 by EMPA at the initiation of Sika AG, Zürich (EMPA, 1995). Based on promising test results, the first application of the method was carried out in Luzern, Switzerland. Strengthening was needed owing to the failure of a steel cable on a continuous timber bridge. The strengthening was carried out by bonding of carbon fibre strips of 6,5 kg altogether. Using the alternative method – strengthening with steel plates - would have required 175 kg of steel. Consequent

Fig. 1 Results of a comparative experimental study (Deuring, 1993)
a) Elevation
b) Cross-section
c) Load vs. deflection diagrams



continuous monitoring of the strengthened bridge indicate its appropriate behaviour in service.

Another well-known example is the Rhine river bridge between Oberriet and Meiningen, Germany (Hollaway and Leeming, 1999). Strengthening of the concrete deck in the transverse direction was required owing to the considerable increase of traffic. Strengthening was carried out by bonding of carbon fibre strips.

Test results by Deuring (1993) give an excellent indication of the influence of strengthening on flexural behaviour (*Fig. 1*). Either prestressed or non-prestressed strengthening strips were bonded to the bottom face of reinforced concrete T beams. In the case of prestressed application, strips were initially stressed and the beams were moved into the layer of glue by a crane and maintained until hardening. (This type of prestressed application is, of course, possible only in laboratories). After hardening of the adhesive the prestress was released by cutting the strips at the ends. *Fig. 1* shows the possible comparison of load deflection responses of un-strengthened and strengthened beams (either with prestressed or non-prestressed strips). The comparative diagram allows the following conclusions:

- strengthening increased the failure load
- increases in the load bearing capacity were practically the same for prestressed and non-prestressed applications of carbon fibre strips of equal cross-sectional area
- presstressed application of strengthening strips led to a considerable decrease of deflections.

In addition to the above mentioned examples, there have been several hundreds strengthening applications carried out world-wide with externally bonded carbon fibre reinforcement. Many applications were, for example, realised in Japan after the Kobe earthquake. In several research laboratories (including Budapest University of Technology) tests are running to determine the properties of strengthening materials, favourable application methods as well as design rules. It means that this strengthening method is already embedded in practice, albeit that further experimental evidence and design models are still needed. Widely accepted design rules are not yet codified and the situation remains similar to that of fibre reinforced concrete.

The International Federation for Structural Concrete (*fib*) has formed a task group entitled "Fibre reinforced polymers as reinforcing, prestressing or strenghtening materials for concrete structures". This committee will be soon ready with an international design guide for strengthening with externally bonded reinforcement.

3. MATERIALS FOR CARBON FIBRE STRENGTHENINGS

3.1 Strip or fabric

Carbon fibres can be applied in two different forms:

1) The first possibility is that the parallel *fibres are embedded in resin by a pultrusion process*. The product is a thin strip of unlimited length having unidirectional fibres. These prefabricated strips are glued to the surface of the member to be strengthened. The glue is responsible only for the contact between the carbon strip and the surface of the member.

   Figs. 3 to 8 give examples to this type of strengthening.

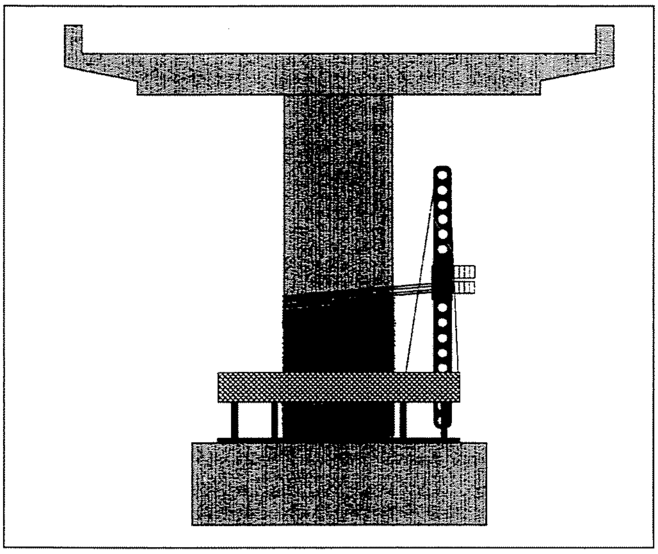


Fig. 2 Automatic wrapping (fib, 2000)

2) The other form is that first a resin layer is applied on the surface of the member and the fibres are put into this resin layer in-situ in the form of fabrics. The fabrics are covered again with a resin layer. This procedure can be repeated several times. The successive fabric layers are embedded into the previous resin layer. Fabrics can be unidirectional by running fibres or perpendicular using woven fibres.

Resin plays a double role in these applications: it impregnates the fibres and ensures their contact to the surface of the member.

Special application method for fabrics is indicated in *Fig. 2*. Jacketing of bridge columns is carried out in an automated way. Both the amount of fibres and their pitch is automatically controlled according to the requirements. Embedding of fibres takes place in the machine head.

In general we can say that pultruded strips require less work in situ, but that the strips are not able to follow sharp changes of section.

Fabrics may require a bit more work in situ, however. They provide the possibility for the application of more fabric layers than strips. The tensile modulus of fabrics after hardening is generally lower than that of pultruded strips owing to manual embedding of fibres into the resin. Sections with small radii or with sharp edges can be strengthened only with fabrics.

3.2 Carbon fibres

Carbon fibres (used for structural engineering applications) are generally produced from polyacrilnitril (PAN) at 1300 °C to 3000 °C. Strength and modulii of fibres as well as density and electric conductivity of fibres can be controlled by the production temperature. The higher the required modulus of fibres the lower the strength and the failure strain and the higher the price of the fibre. The modulus of elasticity can be higher, equal or lower than that of steel. Designers choose the fibres with appropriate modulus based on statical and economic considerations.

The most well-known carbon fibre is called "Toray" and is produced in Japan. Toray T700 SC fulfils the following requirements:

axial tensile strength	≥ 4 800 N/mm^2
axial modulus of elasticity	≥ 200 000 N/mm^2
axial failure strain	> 2%

Property	Sika Carbo Dur S (low modulus)	Sika Carbo Dur M (medium modulus)	Sika Carbo Dur H (high modulus)
Modulus of elasticity N/mm^2	,> 165 000	> 210 000	> 300 000
Axial tensile strength (characteristic value), N/mm^2	> 2 800	> 2 400	> 1 300
Axial tensile strength (mean value,) N/mm^2	> 3 050	> 2 900	> 1 450
Ultimate strain, %	> 1,9	> 1,4	> 0,8
Width, mm	50, 60, 80, 100, 120	60, 90, 120,	50
Thickness, mm	1,2 and 1,4	1,4	1,4

Table 1 The mechanical and geometrical properties of Sika CarboDur strips pultruded from Toray carbon fibres

Production of carbon fibres has also been started in Hungary. PANEX 30 fibres are produced by the firm ZOLTEK with the following characteristics (ZOLTEK, 1999):

axial tensile strength	3 800 N/mm^2
axial modulus of elasticity	228 000 N/mm^2
axial failure strain	1,5%
density	18 kN/m^3
carbon content	94%

An example in *Table 1* summaries the mechanical and geometrical properties of Sika CarboDur strips pultruded from Toray carbon fibres.

3.3 Matrix and glue

The embedded fibre matrix is responsible not only for keeping the fibres together but also to protect them against environmental and localised effects. The resin of the matrix can be polyester, urethane, vinilester or epoxy. In most cases epoxy resin is used.

Fibres are embedded in resin before application in case of pultruded strips and during application in case of fabrics.

The most important property of carbon fibre strengthening is the *bond* between the strengthening material and the structural element to be strengthened. The resin plays the double role of embedding and gluing of fabrics. In case of strips, embedding of fibres is ensured during pultruding while gluing of strips is carried out by a separate resin (which is also generally epoxy based).

The main characteristics of resins are: modulus of elasticity failure strain, shrinkage, glass transition point, coefficient of thermal expansion. The hardening rate of resins depend on their temperature. The higher the temperature the higher the hardening rate.

4. APPLICATIONS IN HUNGARY

The first project in Hungary using carbon fibre strengthening was carried out in 1996. It was then used on a wide range of structures and structural elements:

- Slabs,
- Beams,
- Silos,
- Bridge girders,
- Slab openings,
- truss girders lightly affected by fire.

In the following, short descriptions are given of the Hun-

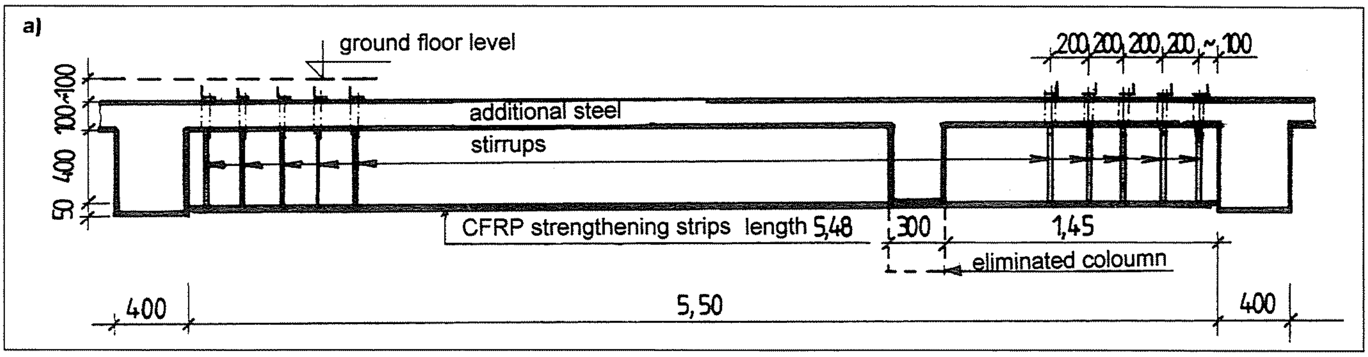


Fig. 3 Strengthening of the slab of a music school in Budapest, Erzsébet street 32. (Date: June 1996. Strengthening strips: Sika CarboDur S512-70 m Designer: Dr. György L. Balázs. Exec: Schöck and Co.)

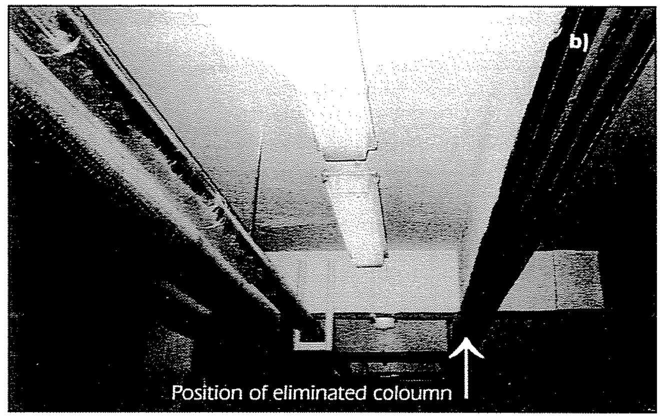


Fig. 4 Strengthening of a reinforced concrete beam owing to the elimination of an intermediate column in the Stollwerck Chocholate Fabric in Budapest (Date: July 1996. Strengthening strips: Sika CarboDur S512-44 m. Designers: Dr. György L. Balázs, Dr. István Hamza, Dr. György Visnovitz. Exec: ISOPLAN Ltd.
a) Elevation
b) Photo after strengthening

garian strengthening projects using carbon fibre products between 1996 and 1999.

4.1 Strengthening of a slab

In 1996, concrete pieces were falling down from the ceiling of a music school in a building that was constructed at the beginning of the 20th century in downtown Budapest.

The investigations indicated that the load bearing elements of the ceiling were I 160 steel girders. The spaces between the girders were filled with slag concrete. The most probable reason of the deterioration was a broken water-conduit running above the ceiling which also produced light corrosion of the steel girders. Strengthening was carried out by gluing CFRP strips to the bottom surface of the steel girders (*Fig. 3.*). The strength of concrete was not adequate for applying strips. A fibreglass grid was also fixed to the bottom of steel girders to avoid further loosening of concrete pieces.

4.2 Strengthening a beam by eliminating a column

The modification of production procedures in the Stollwerck Chocolate Factory in Budapest required that an intermediate column of a reinforced concrete T beam be eliminated. The beam was cast together with the slab.

Analysis of the inner forces of the *modified structural system* indicated that flexural strengthening could be reached by bonding of CFRP strips. However, additional shear strengthening was also required.

Adequate shear strengthening was reached by applying U-shaped external stirrups consisted of steel strips. Anchorage of the external stirrups was provided on the top of the slab using screws within the layers of the industrial floor.

The elevation and illustration of strengthened beam are shown in *Fig. 4*. (The position of eliminated column is indicated by an arrow. External steel stirrups are already covered in the picture.)

4.3 Strengthening of balcony slabs

Several cracks appeared in the natural stone balcony slabs of a four-storey dwelling house in Budapest. These slabs were running all around the internal core of the rectangular building at every level and served as the connection between the flats. In some places the cracks already separated pieces of the slab but were still in original positions.

The structure consisted of a system of simply supported slabs 2,2 m in length and 1,0 m width. The slabs were transversally supported by natural stone cantilevers. The internal edge of the slab was also supported by a wall.

Continuous strengthening strips were applied at the external edge and in the middle of the slabs (*Fig. 5.*). A final plaster covers the bottom of the slabs which masks the strengthening application.

4.4 Strengthening the circular edge beam of a grain silo

The grain silos in Berettyóújfalu were constructed with the so-called Bentall-Simplex method. The cylindrical steel component of these silos is supported by a concrete filler made of prefabricated segmental elements of 12 pieces (*Fig. 6*). The segments were circularly prestressed with four external prestressing strands. The strands had a protecting paint covering

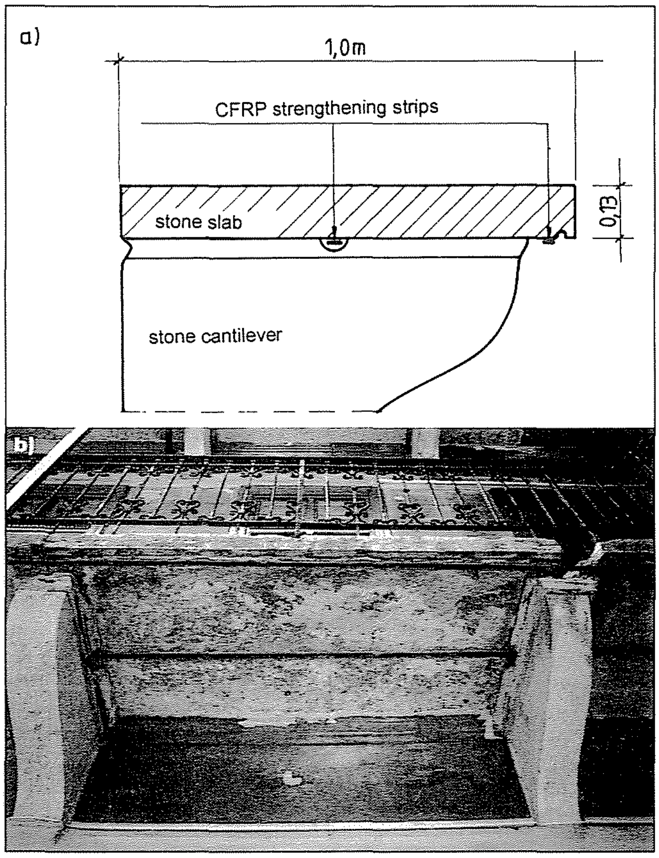


Fig. 5 Strengthening of natural stone balcony slabs in Budapest, Péterffy street 31. (Date: May 1997. Strengthening strips: Sika CarboDur s512-240 m. Designer: Dr. György L. Balázs. Exec: Schöck and Co.)
a) Cross-section
b) Photo after strengthening

but no other protection layers. One of the four strands corroded and failed after ten years service. The silo had to be strengthened owing to the decreased confining effect and the possible failure of the other strands.

The four prestressing tendons were substituted by four Sika CarboDur strips *(Fig. 6.e)*. After the silo was emptied, the strengthening strips were successively applied: first by cutting a strand then gluing a strip (except in the first case when the strand had already been broken). The design of the strengthening had a deformation criteria, being that grain should not be able to penetrate between the prefabricated segments. Strengthening strips were anchored by overlapping and without use of any specific device *(Fig. 6.e)*.

4.5 Strengthening of precast pre-tensioned bridge girders

There are already two cases to show.

4.5.1 Approach span of Petőfi bridge on the Danube in Budapest

The approach span of Petőfi bridge in Budapest was constructed of precast prestressed pre-tensioned concrete bridge girders with an additional in situ reinforced concrete deck *(Fig. 7.a)*. The first girder suffered seriously from corrosion, i.e., five prestressing strands girder situated under a tram line running through the bridge.

Several ways for strengthening were analysed and finally the decision was taken to glue 5 pieces of 28 m long Sika CarboDur strips. This operation was performed during night after the tram traffic had stopped *(Fig. 7.b and d)*. Strengthening strips were of medium modulus equal to the modulus of steel.

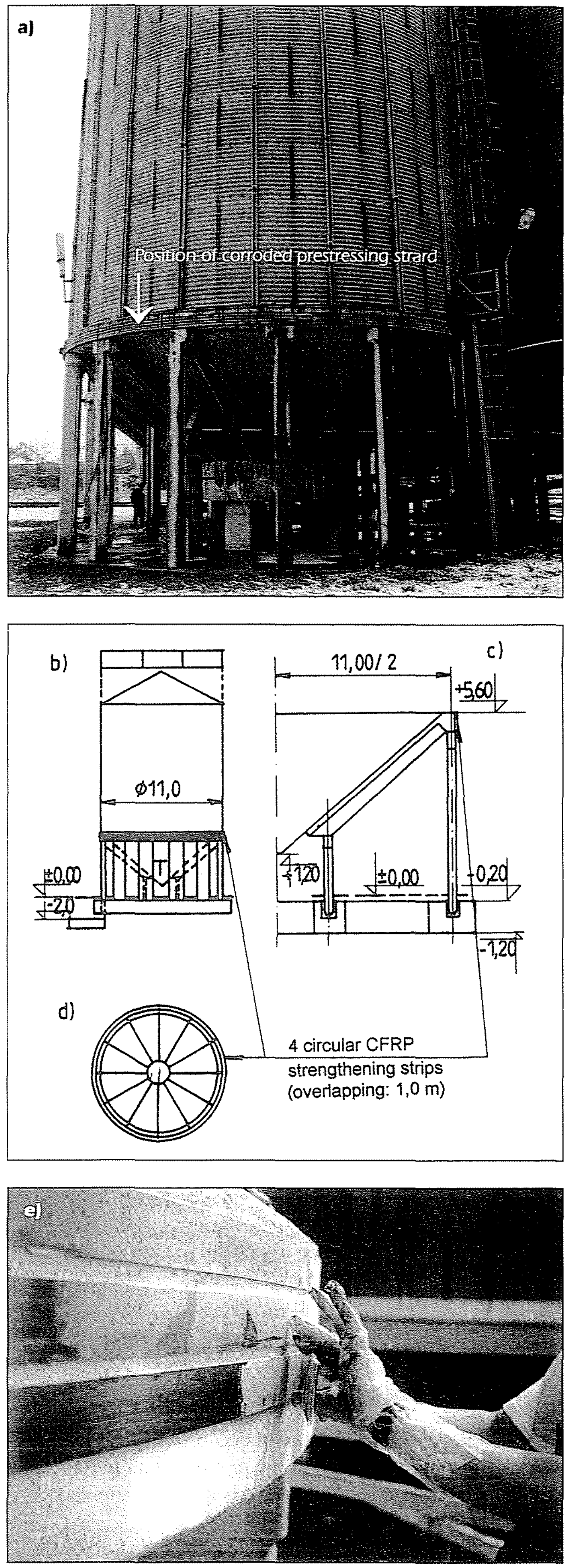


Fig. 6 Strengthening of a grain silo in Berettyóújfalu, Hungary, owing to the corrosion of circular prestressing tendons (Date: June 1997. Strengthening strips: Sika CarboDur S612-152 m. Designer: Dr. György L. Balázs. Exec: Isobau Rt.)
a) Photo of the silo
b) Elevation
c) Vertical section
d) Bottom view (indicating the segmental prefabricated elements)
e) Application of strengthening strips

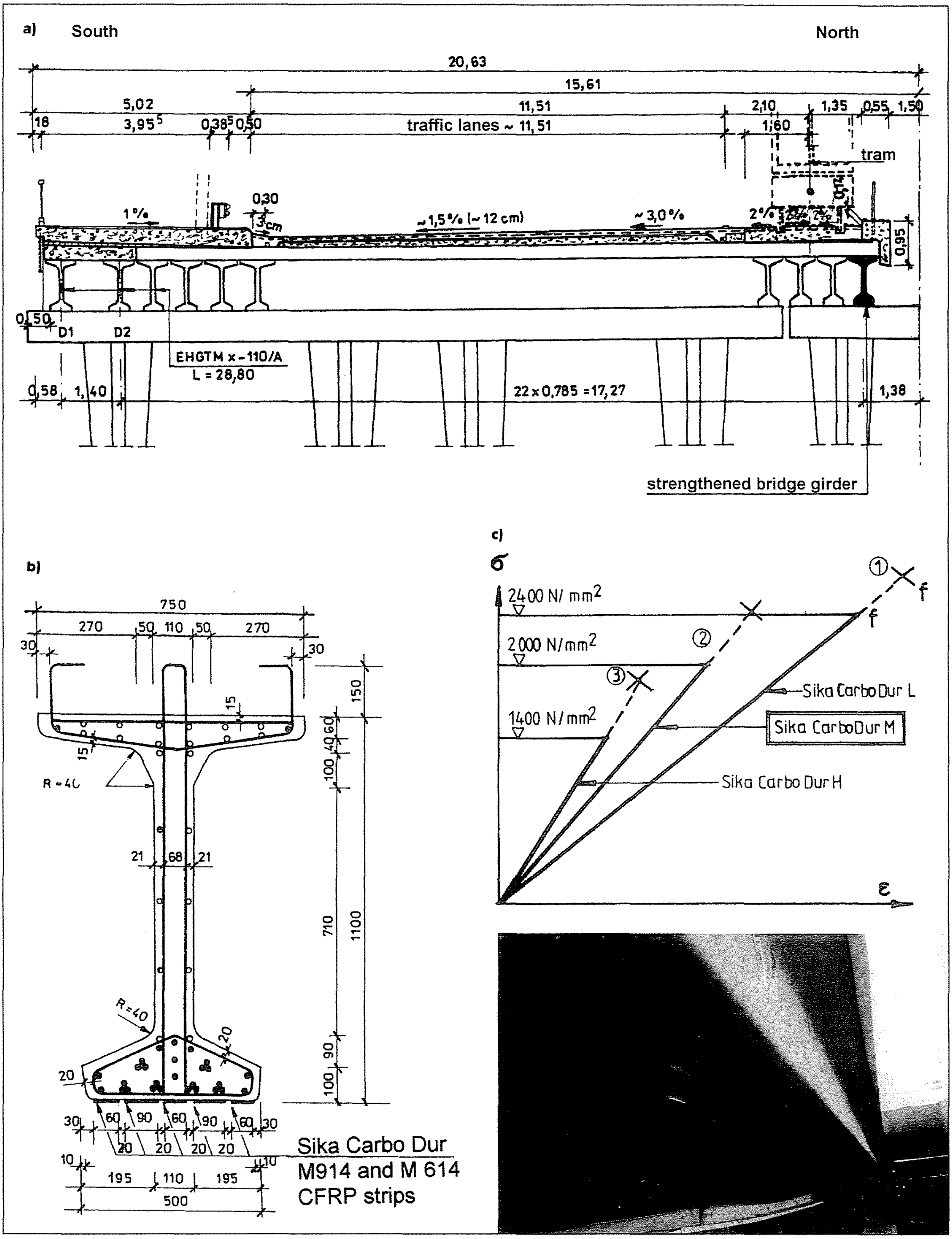


Fig. 7 Strengthening of a prefabricated prestressed pre-tensioned concrete bridge girder in Budapest, Petőfi bridge owing to the failure of some prestressing strands due to corrosion. (Date: Sept. 1997, Strengthening strips: Sika CarboDur H914-56 m and M614-84 m. Designer: Budapest University of Technology: Dr. György L. Balázs and Dr. György Farkas. Exec: Pannon Freyssinet Ltd.)
a) Cross-section of the bridge
b) Cross-section of strengthened bridge girder
c) Stress-strain curve of strengthening strips

The whole bottom flange of the girder was finally covered by a protecting layer against further ingress of de-icing salts into the concrete. Unfortunately, the chloride content of concrete before strengthening was already relatively high and therefore further failure of prestressing tendons is not excluded. Regular deflection and strain measurements are carried out for control.

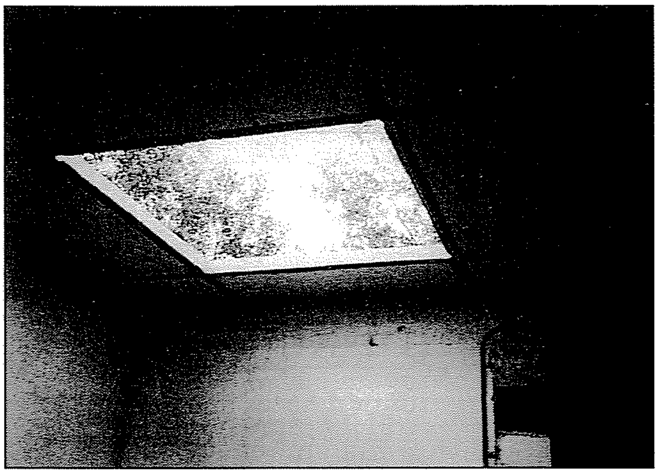


Fig. 8 Strengthening around an opening in a reinforced concrete slab at ZOLTEK Rt. in Nyergesújfalu, Hungary. (Date: Aug. 1998. Strengthening strips: Sika CarboDur M1214-25 m. Designer: Dr. Rita Kiss and Ákos Sapkás. Exec: Fischer-bau Ltd.)

4.5.2. Crossing of M5 and M0 freeways

A truck on the M5 freeway in Hungary hit the prefabricated prestressed concrete girders of the M0 freeway close to Budapest.

The first and the third girders had to be strengthened. Strengthening was carried out by gluing carbon fibre strips. The road was not completely closed during the strengthening work and was restricted to one lane out of two.

4.6 Strengthening of a slab after making an opening

A 3.5x3.5 m opening was required in a 5.8x6.8 m reinforced concrete roof slab in the laboratory of ZOLTEK Rt., Nyergesújfalu, Hungary. The slab was strengthened with 12 mm wide Sika CarboDur strips at the four sides of the opening (*Fig. 8.*, Kiss-Sapkás, 1999).

4.7 Strengthening of a reinforced concrete truss girder

In one of the production halls of Eletrolux in Jászberény, Hungary, some of the reinforced concrete main truss girders suffered from fire damage. The fire attack did not produce serious damage to the girder as it was restricted to the surface. The opportunity was also taken to increase the load bearing capacity of the trusses.

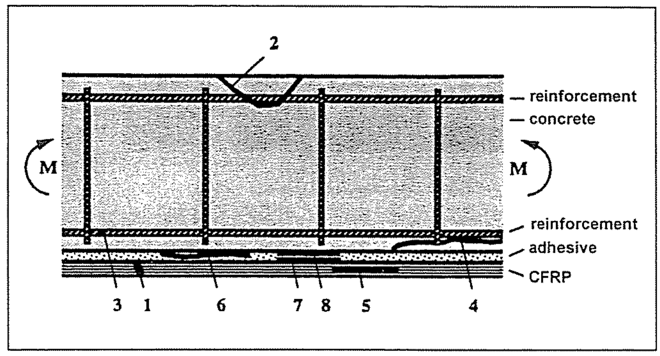


Fig. 10 Possible failure modes of a reinforced concrete member strengthened by carbon fibres (Deuring, 1993)
1 Failure of carbon fibres
2 Crushing of concrete
3 Failure of reinforcing bars
4 Peeling off of the concrete cover
5 Cracking in the strengthening layer
6 Cracking in the glue layer
7 Debonding of glue and strengthening material
8 Debonding of glue and concrete

Strengthening was carried out by applying several layers of CFRP fabrics in March 1999.

5. DESIGN PRINCIPLES

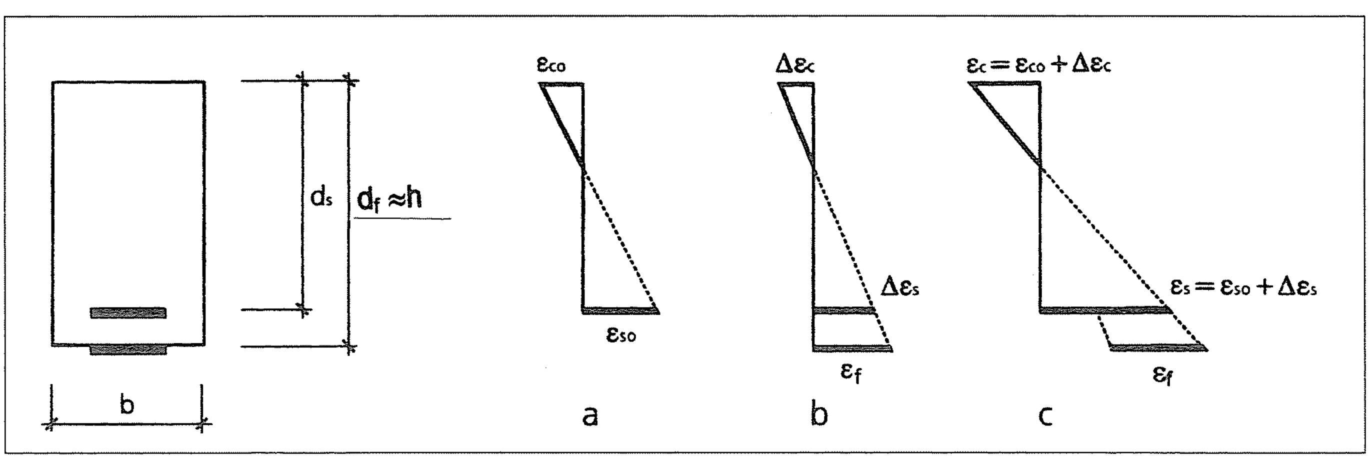
Detailed discussion of design steps was not the purpose of this paper (see Refs. Neubauer and Rostásy, 1997; Triantafillou, 1998; *fib*, 1999). The subject will be covered in a future one. However it is intended to draw attention to the design principles of CFRP strengthening wherein:

- The design of strengthenings should be based on the stresses and deformations of the structure before strengthening (*Fig. 9*).
- Strengthening elements should be stressed only from loads applied after the strengthening procedure.
- Design of strengthenings should include the analysis of all possible failure mechanisms.
- Flexural strengthening is not always enough. Shear strengthening may also be required.

Possible failure modes are the following (*Fig. 10*, Deuring, 1993):

- Failure of strengthening strips or embedded reinforcing bars (or both).
- Crushing of compressed concrete.
- Debonding of glued surfaces.
- Cracking within the glue layer.
- Peeling-off in the anchorage zone.

Fig. 9 Stress-strain diagrams before and after strengthening
a) Strains before strengthening b) Strains from loads above strengthening c) Strains of strengthened member



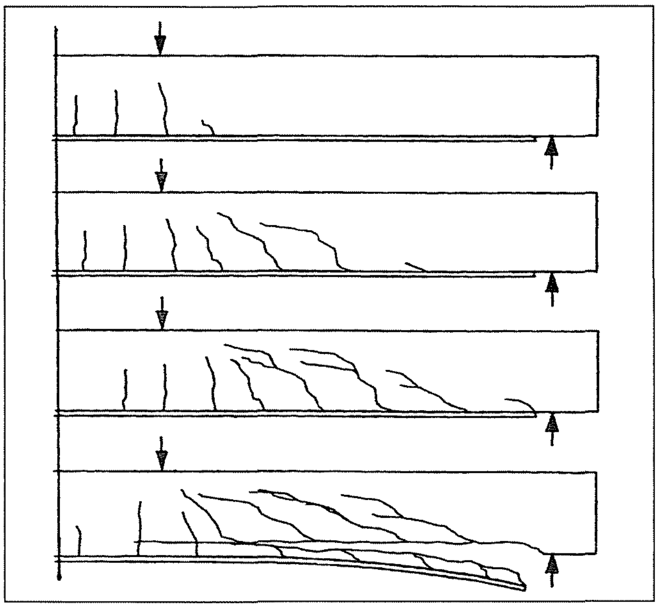


Fig. 11 Anchorage failure (peeling off) (Hollaway and Leeming, 1999)

The last failure mode (which is considered the most dangerous failure mode) is indicated in *Fig. 11* by a test result from Hollaway and Leeming (1999). Flexural and shear cracks were developed at the beginning of loading. However, the failure finally occurred by the peeling-off of the strengthening strips together with a thin concrete layer from the concrete cover.

An additional factor to be considered by design is that the transverse strength of carbon fibres is considerably lower than their high longitudinal strength. Therefore, high stresses transverse to the axis of the strengthening fibres are not allowed to be applied.

6. EXECUTION

Points 1 to 5 and 8 to 10 below are valid both for application of CFRP strips and fabrics. Points 6 and 7 are related mainly to strips. The execution of strengthening should be carried out according to the following steps. Special care should always be taken to ensure the high quality of work.

1) *Tensile bond strength* of concrete is to be measured. It is generally accepted that it should reach at least 1.5 N/mm^2.
2) *Preparation of concrete surface*
   - Mechanical cleaning of the surface should be carried out (e.g. by sand blasting). Best bond is obtained if the surface is not completely smooth but has a roughness: 0.5 to 1 mm).
   - Deep surface holes are to be levelled.
3) *Measuring the water content* of the concrete surface before gluing.
   Water content of the concrete is not allowed to exceed 4% (owing to the appropriate bonding of the glue).
4) *Cleaning* the concrete surface from dust.
5) *Marking* the location of strengthening strips or fabrics on the concrete surface (if necessary).
6) *Cutting of strengthening strips or fabrics* to their design length.
7) *Cleaning strengthening strips of dust.*
   The surface of the strengthening strips to be glued is to be cleaned until a white cloth remains uncontaminated. The purpose of this cleaning is to remove carbon and other dusts to provide better bonding.
8) *Mixing of the two-component glue.*
   Two-component resins must be mixed according to the description given by the producer.
9) *Application of glue as well as carbon strips or fabrics.*
   The application procedure for glue and strips or fabrics was given in Sections 3.1. Support of strengthening strips or fabrics during the strengthening procedure is not required owing to their low weight.
10) *Protecting layers.*
   Protecting layers can be applied for aesthetic reasons, for corrosion protection of embedded reinforcement (carbon fibres are non-corrosive), for fire protection or for UV protection.

7. CONCLUSIONS

The new materials for strengthening of structural elements - fibre reinforced polymers – have appeared in the last decades. Amongst carbon, aramid and glass fibres, carbon fibres seem to provide the most advantageous properties and therefore their extended use can be prognosticated. Carbon fibres are applied in fabric or pultruded strip forms.

Though experimental investigations and preparation of design guides are not yet finished, results of applications are promising.

The advantages of carbon fibres and strengthening methods with carbon fibres: Non-sensitivity to electrolytic corrosion, high strength (both for short-term and long-term or cyclic loads), low weight (low transportation cost, easy application even in small spaces, no need for scaffolding to support the strengthening materials during hardening of adhesive), no limitation in length, no reduction of height below the strengthened member (owing to the very small thickness of strip or fabric), easy to cover and economic (considering the overall strengthening costs).

Drawbacks: Carbon fibres and carbon fibre products are nowadays relatively expensive, protection must be provided in places where vandalism is not excluded, more sensitive to mechanical attacks than steel, surrounding temperature can not exceed approximately 50 °C if no heat or fire protection is used (because of the relatively low glass transition point of the resin).

Reasons for strengthening can include: Decrease in load bearing capacity, increase of inner forces (owing to the increase in loads or modification of the structural system).

Material of strengthened member can be: Concrete, reinforced concrete, prestressed concrete, natural stone, masonry, timber or steel. Most of the applications were carried out on concrete structures.

First application of carbon fibres for strengthening *in Hungary was carried out in 1996.* Since then the method has been used on a wide range of structures or structural elements.

8. ACKNOW LEDGEMENTS

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Extension and Upgrade of South-Pest Wastewater Treatment Plant

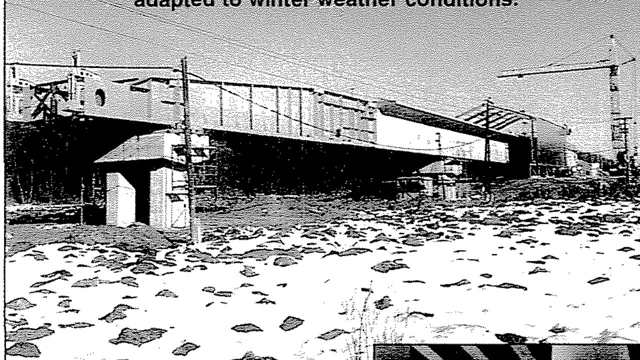
The State-owned Hídépítő Company, the professional forerunner of Hídépítő Részvénytársaság, was established in year 1949 by nationalising and merging private firms with long professional past. Among the professional predecessors has to be mentioned the distinguished Zsigmondy Rt. that participated, inter alia, in the construction of Franz Joseph Danube-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 participating in it were awarded the State Prize. By this technology were constructed five bridges in the region of the rivers Körösök and this was applied at the flyover of Marx square (today Nyugati square) in Budapest, still the most up-to-date two-level crossing in the capital requiring the minimum maintenance costs.

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 on the Mosoni branch of the Danube, for the road bridge at Csongrád over river Tisza, and for the bridge on the Motorway M0 over the Soroksári branch of the Danube.

TÜV CERT DIN EN ISO 9001

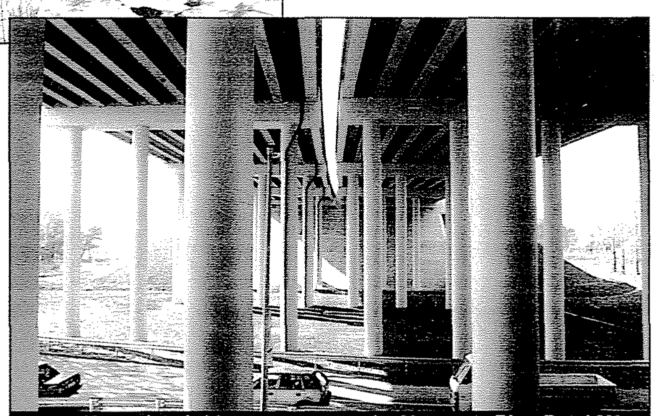
Because of the strong quantitative growth, the use of prefabricated bridge elements for the construction, seemed at that time to be a very great development, and the "Hídépítők (Bridge Builders)" undertook a significant piece of the work both in development and in introduction. A number of up-to-date, monolithic bridges with spare hollows were constructed on the section by-passing the city of Győr of the motorway M1. An important result of the technologic development was the introduction of the so-called incremental launching method. In the period from 1989 up to now yet 22 bridges were constructed by this method, mainly on the base of the own designs prepared by the Company's Technical Department, and even nowadays is in progress the construction of the country's longest bridge (1400 m), a stressed bridge made of reinforced concrete on the Hungarian-Slovenian International Railway Line.



NThe 1400 m long viaduct on the Hungarian-Slovenian International Railway Line in the course of construction, adapted to winter weather conditions.

Important results were achieved by the "Hídépítők (Bridge Builders)" in the field of foundation's technological development as well Among others they have made general the bored piles with large diameter and making CFA (Continuous Flight Auger) piles as well as the procedure of ground strengthening in columns (JET Grouting). A new method has been developed for constructing bridge piers in living water by means of prefabricated elements made of reinforced concrete. Further, the Company did a bulk of the construction of the Budapest Metro network (section of metro line M3 under the Váci road, the elongation of the so-called "Millennium" metro line, etc.).

At these days sight has been taken at winding up the country's arrears in infrastructure (e. g. construction of the Drinking Water Treatment Plant at Csepel, the Extension and Upgrade of South-Pest Wastewater Treatment Plant, the sewerage and construction of wastewater treatment plants in the region of Várpalota, sewerage in the villages Budakalász, Vecsés and the town Szeged, etc.) The Company makes every effort to inspiring confidence in the client by quality work. Therefore it has introduced and has been operating a Quality Assurance System in conformity with the requirements of standard ISO 9001:1994, which fact is also justified by an international certificate.



Up-to date bridge structure on the Motorway Ring-Road M0 around Budapest

Hídépítő Rt. is hopefully expecting new tasks, to be weighed in the field of construction, reconstruction of new bridges and development of infrastructure in order to rise further its reputation.

MÁV BRIDGE CONSTRUCTION CO. LTD

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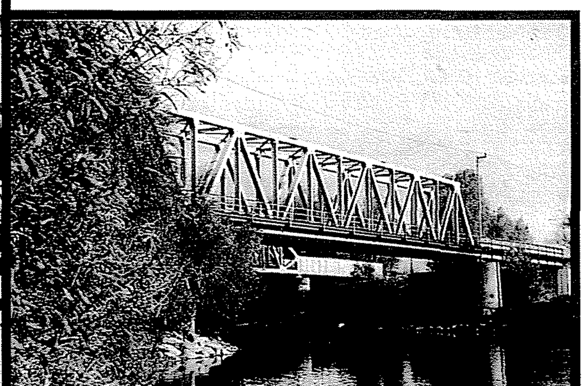
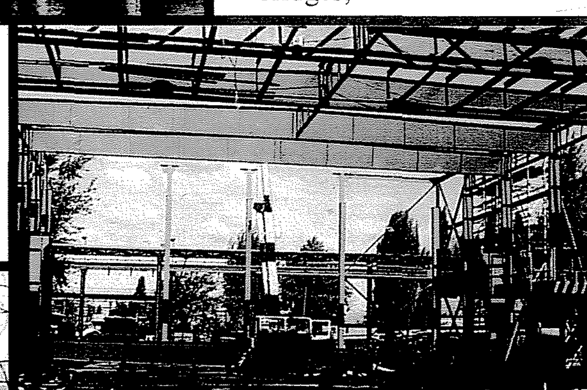
The MÁV Bridge Construction Co. Ltd. was established on the 1st of August 1992, as the legal successor of the former Bridge Construction Directorate of MÁV (Hungarian State Railways).
The Company consists of a staff of specialists having been experienced in structural engineering and construction engineering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

Scope of activity

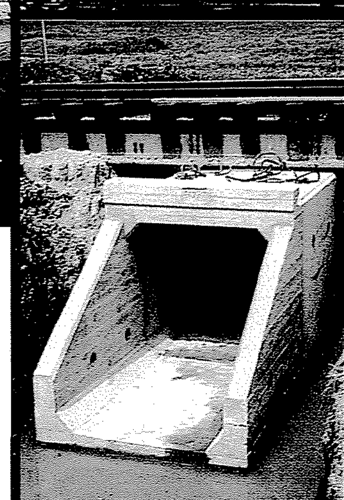
- Production and assembly of welded and riveted bridge structures;
- Production and assembly of industrial steel structures (crane tracks, hall constructions, etc.);



- 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;



- 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



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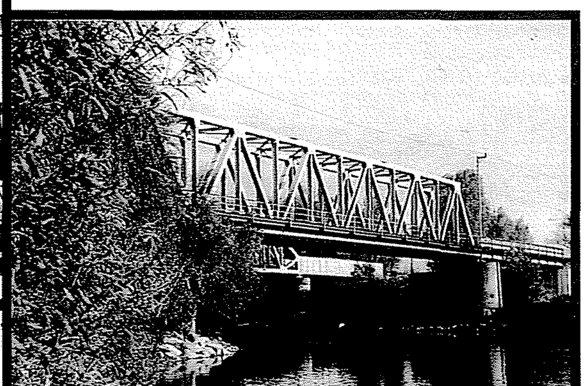

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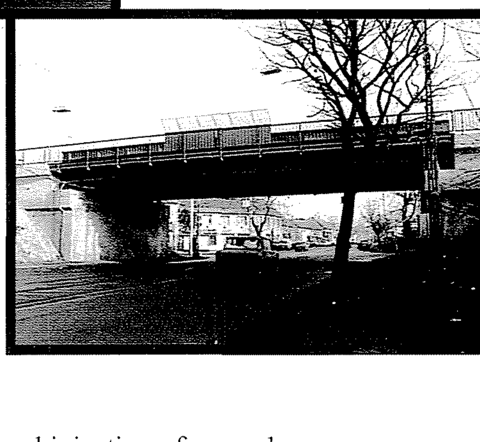

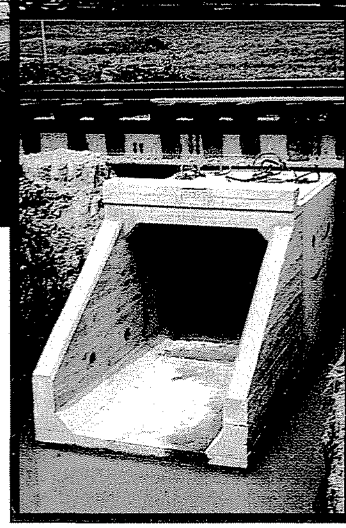

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