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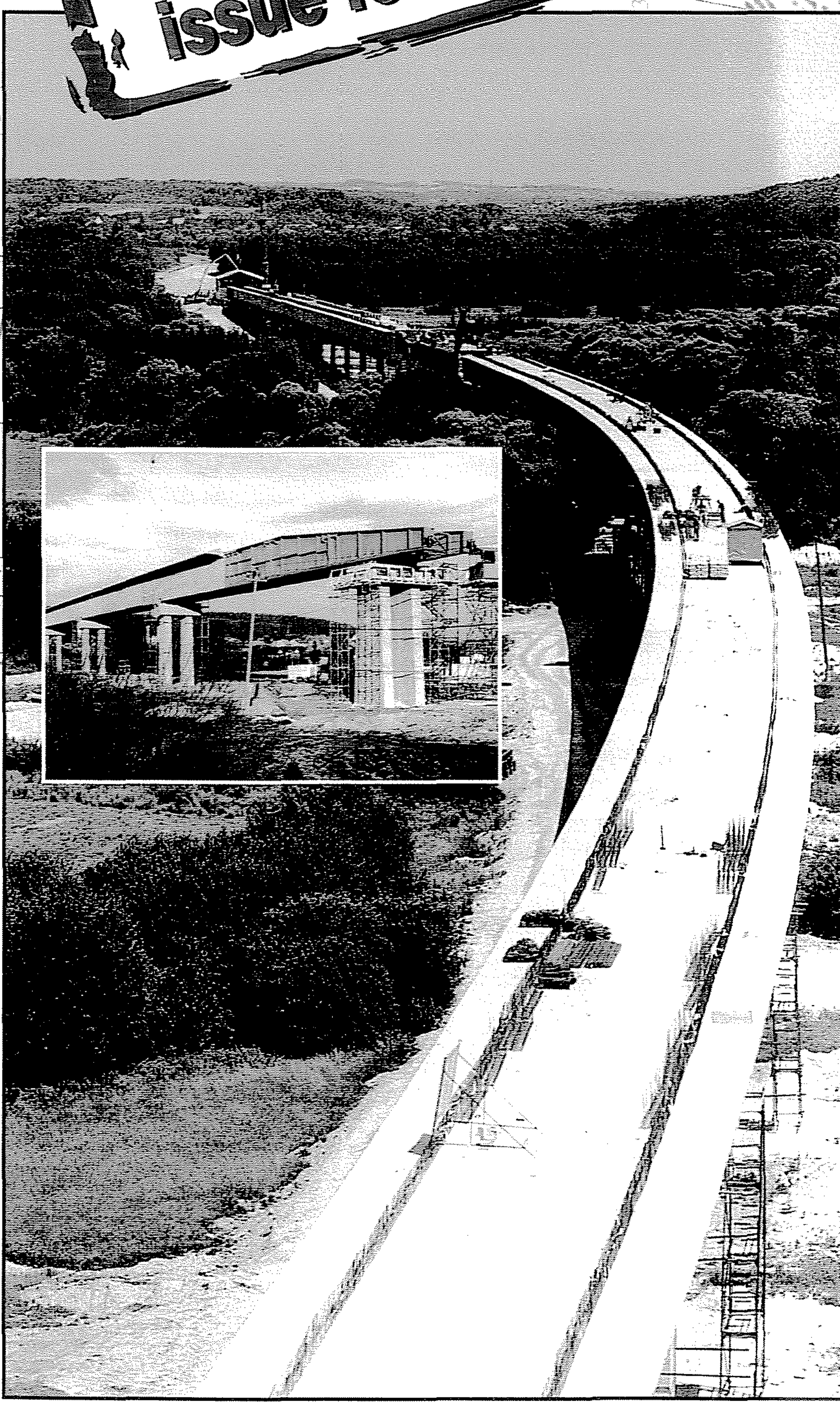
HUNGARIAN GROUP OF *fib*

CONCRETE STRUCTURES

ANNUAL TECHNICAL JOURNAL

Special issue for *fib* 2002 Osaka Congress



Balázs L. G.:
Greetings 2

Tassi:
National reports 3

Wellner–Mihalek:
Railway bridge 7

Horváth–László–Németh:
Precast bridge girder 11

Polgár:
Precast hall construction 14

Almási–Bancsik–Nagy:
Stand structure 16

Pintér–Vörös:
Coach terminal 19

Földvári:
National Theatre 24

Varga–Vigh:
Slipforming 31

Tóth:
Hydraulic structures 35

Józsa–Nemes:
Lightweight concrete 41

Balázs G.–Kopecskó:
Chloride binding 47

Zsigovics–Balázs L. G.:
Self compacting 52

Kovács I.–Balázs L. G.:
FRC beams 57

Kovács T.–Farkas:
Dynamic characteristics 64

Farkas–Kovács T.:
Prestressed slabs 70

Borosnyói–Balázs L. G.:
CFRP prestessing 75

Orbán–Balázs L. G.:
SIFCON 81

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Viaduct unader construction on the Hungarian-Slovenian railway line

CONTENT

2 Prof. György L. Balázs
Hungarian Group of *fib* greets the *fib* Congress Osaka 2002

3 Prof. Géza Tassi
Hungarian concrete structures on previous congresses

7 Péter Wellner – Tamás Mihalek
One of the longest prestressed concrete railway bridges of Europe – executed by incremental launching method in Hungary

11 Adrián Horváth – Viktor László – Tamás Németh
Application of precast prestressed girders in bridge construction

14 László Polgár
Hungarian precast concrete hall construction

16 Dr. József Almási - Csaba Bancsik - János Nagy
Special concrete stand structure

19 Sándor Pintér – Balázs Vörös
Budapest Népliget coach terminal and office building

24 Gábor Földvári
Reinforced concrete structures of the new National Theatre in Budapest

31 Dr. László Varga – István Vígh
Slipformed and other shell structures

35 Dr. László Tóth
The most up-to-date hydraulic engineering projects in Hungary

41 Assoc. Prof. Zsuzsanna Józsa - Rita Nemes
Recycled glass aggregate for lightweight concrete

47 Prof. György Balázs – Katalin Kopecskó
Chloride binding in concrete

52 Dr. István Zsigovics – Prof. György L. Balázs
Self compacting concrete – some Hungarian experiences

57 Imre Kovács – Prof. György L. Balázs
Structural behaviour of steel fibre reinforced concrete

64 Tamás Kovács – Prof. György Farkas
Effects of cracking on the dynamic characteristics of concrete beams

70 Prof. György Farkas - Tamás Kovács
Prestressing in floors using unbonded tendons

75 Adorján Borosnyói – Prof. György L. Balázs
Prestressing provided by CFRP tendons

81 Zoltán Orbán – Prof. György L. Balázs
Concrete with high steel fibre dosages – an effective cementitious material for structural repair

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HUNGARIAN GROUP OF *fib* GREETS THE *fib* CONGRESS OSAKA 2002

DEAR READER, DEAR PARTICIPANT OF THE *fib* CONGRESS,

Concrete is certainly the most important structural material of the 21th century. Reasons are its wide range of applicability and constructibility in addition to its relatively low price.

During the last century we were able to learn what concrete means including its advantages as well as its drawbacks. In the new century we must be able to eliminate all of its drawbacks at least by the new constructions. Our optimism is supported by the improved concrete technology and the ever increasing ideas to reach tailored properties of concretes to various applications.

Structural concrete offers enormous design flexibility and is an economical solution to almost any structure.

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.

In addition to conventional normal-weight aggregate concrete, concrete can be made of light-weight aggregates or fibres can be added to improve its characteristics. Alternatively, new possibilities are offered by non-corrosive, non-metallic reinforcement.

The First fib Congress in Osaka October 2002 is a good example of reviewing fields of interest of concrete engineers like: innovative structures, advanced design, seismic design, new materials, durability, high performance concrete, recycling, safety, management, aesthetics and monitoring. These all contribute to the successful application of concrete.

Present issue intends to give a picture on the application and research in Hungary within the last couple of years from the FIP Congress in Amsterdam.

Merger of former CEB and FIP into *fib* produced a favourable situation in Hungary. All major design offices and construction companies on the field of concrete engineering became members of the Hungarian Group of *fib*.

Our Journal of *CONCRETE STRUCTURES* was founded three years ago with the principle objectives to publicise the most recent technical developments in Hungary in the fields of concrete, reinforced concrete and prestressed concrete structures and research. Our aim was therefore to give a forum to practical and theoretical papers related to concrete as a structural material and the scope of the contributions would range into all disciplines connected to realisation, i.e. design, construction, prefabrication, constituent materials, quality management, research and codification.

The title of the journal, *CONCRETE STRUCTURES*, intends to reflect all the interconnected fields. Papers can be submitted by anyone who finds concrete structures important and who wishes to share their experiences. 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 is not only 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* was 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 on the first page. We would therefore like to take this opportunity to express our gratitude to the sponsors.

Finally, I hope that you will find the contents of this volume of the journal both informative as well as useful and that it provides interesting insights into the technological developments in Hungary.

The Hungarian Group of *fib* wishes much success to the organizers and participants of *fib* Congress Osaka 2002.

Balázs

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

HUNGARIAN CONCRETE STRUCTURES ON PREVIOUS CONGRESSES

Prof. Géza Tassi

The construction industry in Hungary has a very rich history. In the 19th century, a short time after the beginning of European Portland cement production, noteworthy plain concrete structures were built. Later, when reinforced concrete was invented, the Hungarian building industry made use of the new technology very early. The first decade of 20th century brought outstanding achievements in concrete construction and in spite of wars and other difficulties, the development was continued. The Hungarian engineers started to work in FIP in 1962. After the establishment of the Hungarian FIP Group, the development of structural concrete in the country was regularly presented at the FIP Congresses.

Keywords: early structures, development of structural concrete, FIP Congresses

1. INTRODUCTION

To show the real value of the building industry of a country, it is worthwhile to study the roots, the ancient relics and the early achievements in new technology. The national reports at the FIP Congresses always provided a good survey of recent development of concrete structures.

2. ONE THOUSAND YEARS OF CONSTRUCTION AND EARLY CONCRETE STRUCTURES

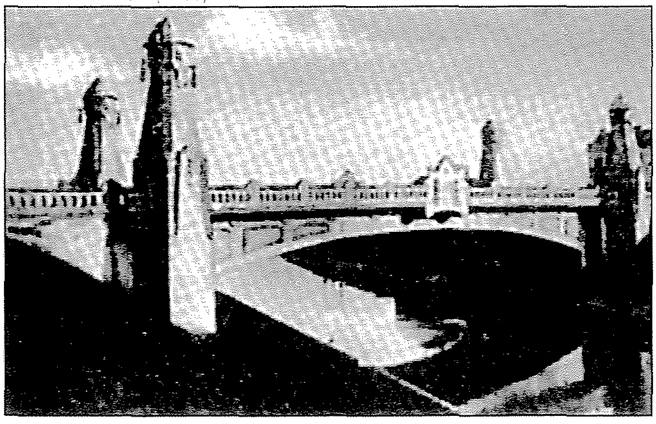
Hungarians arrived to the Carpathian Basin, in the middle of Europe, more than one thousand and one hundred years ago. Following the establishment of the Hungarian state, very keen construction activity started. Beginning from the 11th century, significant churches, monasteries, castles and forts were erected. For instance, the church in Ják, built in 1256. The country survived stormy centuries, had to suffer invasions, occupations, civil strives, grievous disasters and epidemics. However, the Hungarian professional knowledge kept in step with international development.

Consequently, the Hungarian industry has produced splendid examples of buildings in the last millennium. Based on a good tradition and the talent of Hungarian building masters, engineers and workers – among them some of the first carpenters – concrete construction underwent a boom in the second part of the 19th century, when the technology was spreading throughout Europe.

Roman cement had already been in use for centuries. The first important structure in Hungary using Portland cement was completed in 1854. This was the plain concrete lock of a canal between the two big Hungarian rivers (Danube and Tisza), with a concrete volume of 19 000 cubic m. Another example is the construction of the Budapest Parliament started in 1885. For its overall 2 m deep footing slab, 58 000 cubic metes of Portland cement concrete was cast.

The first Hungarian reinforced concrete small bridge was completed in 1890. To illustrate further the quick development of the industry, it should be mentioned that the retaining walls and the floor slabs cast between steel beams of the first underground line in the continental Europe were made of reinforced concrete in the Hungarian capital in 1894.

Fig. 1 RC girder bridge - the largest span in the world at the time of erection (1908)



3. THE FIRST BOOM IN APPLICATION OF REINFORCED CONCRETE AND OUTSTANDING STRUCTURES BEFORE WORLD WAR II

In 1908, Hungary could be proud of the longest – approx. 40 m – span reinforced concrete bridge of the world that time (*Fig. 1*). In the same year the reinforced concrete arch for a railway bridge was also a "world champion" with its 60 m span.

There were many other achievements until the end of World War I. The dictated peace caused severe difficulties. An example being that the majority of the cement factories were situated beyond the new borders of the country. However, the development did not stop completely. Between World War I and II large span bus garages, industrial buildings, indoor swimming pools, grain storehouses, silos, hydraulic structures, bridges and many reinforced concrete skeletons for residential and public buildings were built. The early achievements of shell structures –

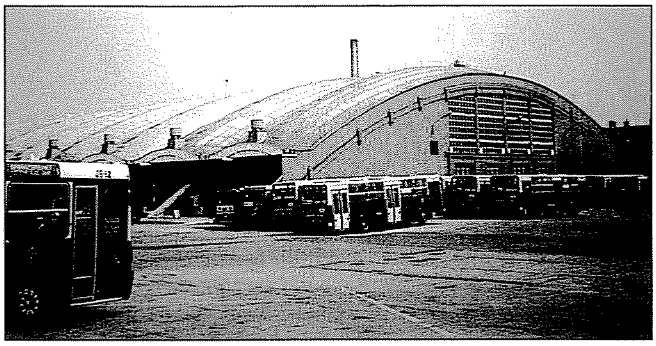


Fig. 2 Bus garage with arch and shell roof

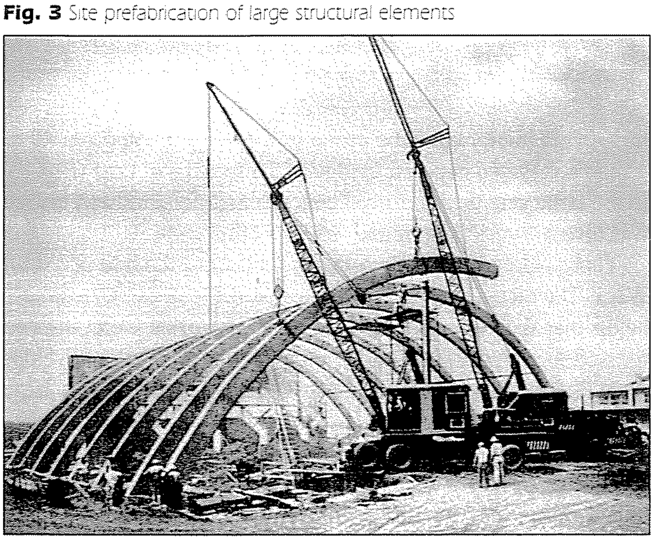
among others the bus garage in Buda shown in *Fig. 2* and the first application of prestressing belong to this period.

4. THE POST-WAR SHORTAGE AND THE BUILDINGS FOR THE QUICK MARCH TOWARDS INDUSTRIALISATION

During World War II Hungarian concrete structures suffered considerably. After the war the reconstruction of destroyed and damaged structures provided many tasks for the building industry. Later on, forced industrialisation provoked a boom in concrete construction. Many power stations, buildings for the chemical industry, metallurgy, machinery, storage and other buildings for the agricultural co-operatives were constructed. Furthermore, many military structures were completed. The forced construction works and the shortage of timber for scaffolds and formwork resulted in a situation where Hungary pioneered prefabrication of large RC elements (see e. g. *Fig. 3*). The experience gained in this field was widely used abroad. When the cold war ended a wide range of residential buildings started. For this purpose large panel housing factories were created. The mass production of factory-made – in great part prestressed concrete – members was characteristic. Besides floor beams, floor panels and other members, load bearing structures for the industrial and agricultural buildings were produced in large quantities. Piles, high voltage masts, lampposts, tubes were developed. In concrete railway sleepers, Hungary became a major supplier. (E. g. later on, in 1980, the volume of factory made concrete elements was 1.2 million cu. m.)

– N. B.: The population of Hungary is about ten million.

Fig. 3 Site prefabrication of large structural elements



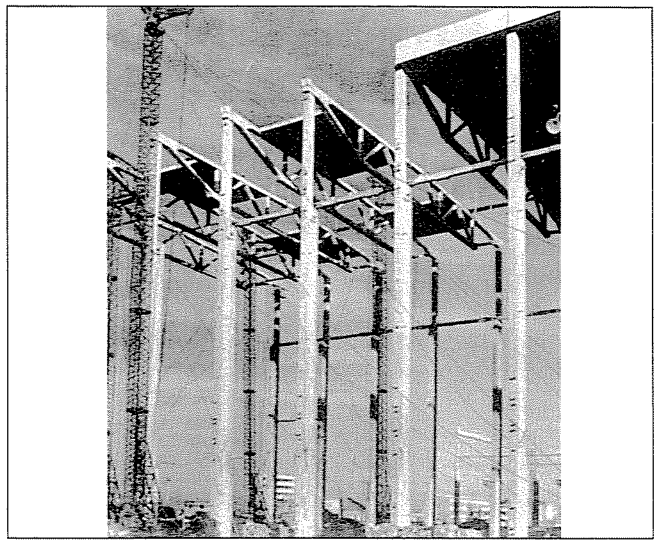


Fig. 4 Large span PC trusses for industrial hall

5. HUNGARIAN ACTIVITY IN FIP AND NOTEWORTHY STRUCTURES

Hungarian engineers used to pay attention to the work of FIP even after the international federation was founded. Hungary was first represented at the *IV Congress of FIP in Rome and Naples (1962).* Presentations were given on a ten thousand cu. m capacity prestressed concrete water reservoir built in South Hungary and on the PC railway sleeper production. Two other studies were published in the proceedings and one in a periodical offered to the Congress by another FIP Group.

In Paris, 1966 at the *V Congress of the FIP*, large span prestressed concrete trusses (*Fig. 4*) for industrial halls and other industrial buildings were shown to the participants.

The FIP VI Congress in Prague (1970) was the first FIP meeting where Hungarian delegates were present in a large number, just after the Hungarian FIP group was founded. Hungary distributed an issue of the periodical "Hungarian Building Industry" with an English text. Seven printed papers were distributed and presented in different sections. At each "Outstanding Structures" session, Hungarian reports were delivered. At the session for buildings, among others, the new type beams and 'TT' panels were presented. The first segmental (not by balanced cantilevering) bridge structures in Hungary,

Fig. 5 Hyperboloid of revolution shaped water tower

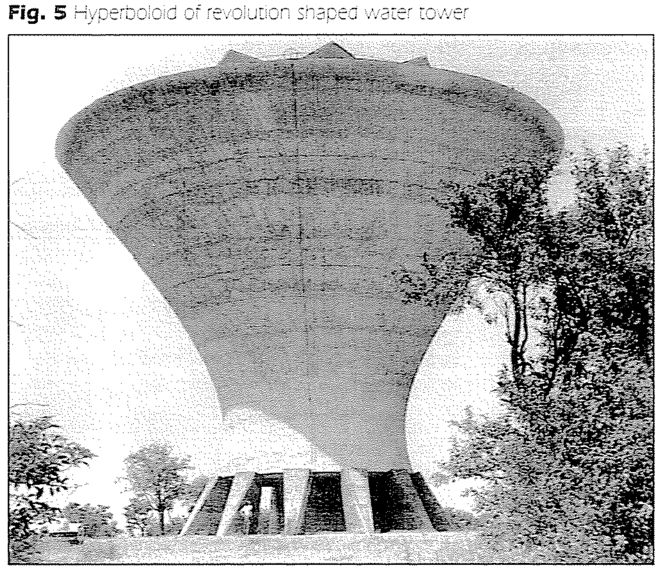


Fig. 6 Precast PC girders for a bridge in horizontal carvature

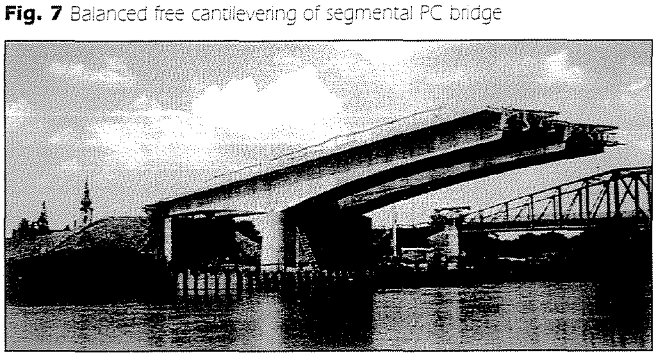
as well as site and factory made PC bridge girders were shown. The Hungarian structures for the "Other structures" session were the following: a 200 m high RC chimney with a PC shell foundation, vertical-flow water cleaning basins with shell roofs made of precast elements with prestressed concrete foot rings, a 2000 cu. m hyperboloid of revolution prestressed concrete water tower (*Fig. 5*).

At the *FIP VII Congress in New York (1974)*, there were six Hungarian presentations. All these were distributed among the delegates. At the "Buildings" session of "Outstanding Structures" series the application of (at that time) new large span 'T' roof panels were displayed. At the bridge session the chief article was the improvement of factory made pre-tensioned, as well as site precast post-tensioned bridge girder systems. It was shown how continuous girder behaviour can be used and how the technologically troublesome intermediate cross beam can be avoided. The different solutions for bridges laying in a curvature in ground plan, were also discussed (*Fig. 6*).

At the *FIP VIII Congress in London, 1978*, the Hungarian FIP Group distributed five papers, which were discussed at different sessions. At the Outstanding Structures sessions, among other bridges, the use of balanced free cantilevering technology was the focus of the national report (*Fig 7*). From among the outstanding "other" structures, let us mention the following three: A robust, bi-directional prestressed foundation block for a 160 000 metric ton capacity clinker silo. It was worthwhile to mention the construction of the cut-and-cover method of the Budapest No. 3 underground line. PC soil anchors were applied for the diaphragm walls and the top floors were made of different PC elements. The most attractive engineering object of that period was the 80 000 cu. m water reservoir of the St. Gellért mount in Budapest. The structure was made according to the Dywidag system, in two parts, each with a piano-form ground plane, with various prestressed and precast structural elements (*Fig. 8*).

In Stockholm, 1982, at the FIP IX Congress, there were only four presentations – also in a written form – but there was no opportunity to show new technical achievements.

Fig. 7 Balanced free cantilevering of segmental PC bridge



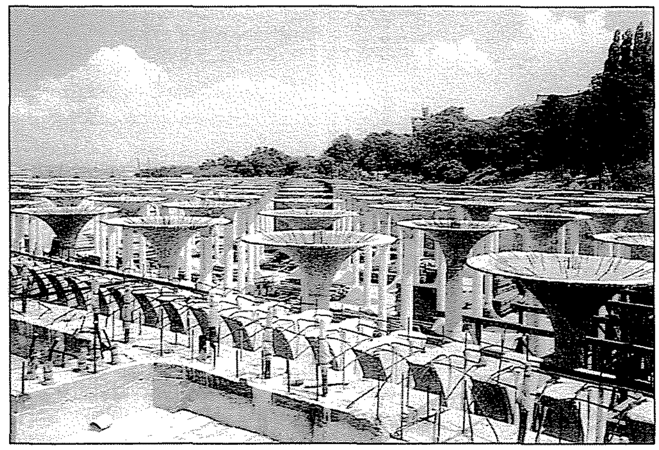


Fig. 8 80,000 cu. m capacity PC water reservoir

At the *FIP X Congress (New Delhi, 1986)*, three Hungarian papers and two posters were present. At the "Outstanding and Innovative Structures" sessions, a lot of interesting Hungarian objects were introduced to the audience. The majority of the works were slip-formed tower-like structures. Some water towers were made such that the conical shaft formed the reservoir itself . A 3 000 cu. m capacity water-tower was proven to be a very good solution. The stem is slip-formed. The reservoir is then assembled on the ground. The conical outer cover consists of factory made precast segments which serve as a formwork for the internal parts. When the complete reservoir was ready, it was lifted to the top of the stem (*Fig. 9*).

The FIP XI Congress in 1990 was held in Hamburg. There were only two Hungarian presentations but the Hungarian FIP Group also published a colour booklet containing 13 articles. And in addition to this a wide range of recent Hungarian achievements were shown at the National Reports session. Besides new pre-tensioned concrete members (e. g. Span-Deck floor elements) the new type of U-form girders and bridges made of them were shown. Referring to the previous successful application of monolithic balanced free cantilevering, the Danube-branch bridge of the M0 motorway circular around the capital was *discussed (Fig. 10)*, as well as the PC deck slab of the composite girder bridge across the main branch of the river. The first Hungarian application of the incremental launching technology was mentioned as well. Therefore, many interesting structures could be seen at the exhibition booth of Hungarian firms.

Fig. 9 Slip-formed stem and precast tank for water tower

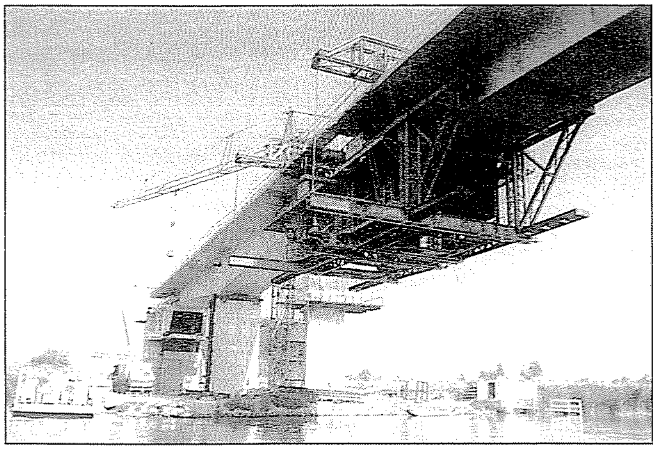


Fig. 10 Balanced free cantilevering of monolithic PC bridge

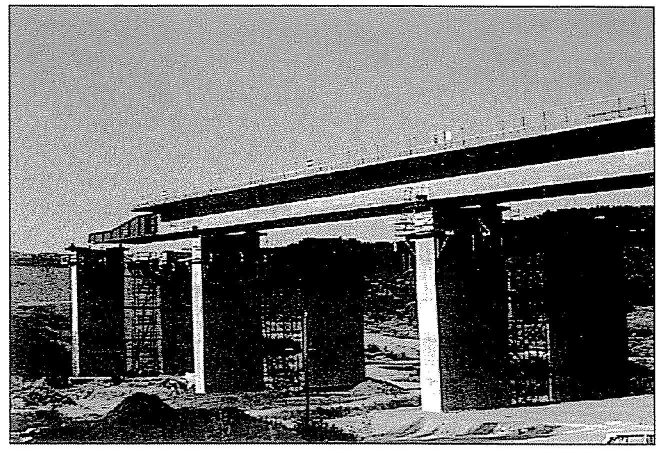


Fig. 11 Incremental launching system with Hungarian specialities

The 1994 FIP XII Congress took place in Washington. The Hungarian building industry was represented at both the exhibition and at the poster sessions. – The National Report focused first on the new type of light PC structures for industrial and commercial halls. In the field of bridge building, the innovative Hungarian system for incremental launching was shown (see *Fig. 11*). St. Steven bridge across the Tisza river with a full length of approximately 700 m was discussed. The river bays are bridged by a twin box girders constructed using balanced cantilevering (maximum span 120 m), while the flood area bays used incremental launching. In this way the full 700 m long bridge is one monolithic unit.

The last *FIP Congress, the XIII, was held in Amsterdam (1998).* Besides four scientific papers submitted by Hungarian delegates, the National Report gave an account of some novelties in Hungary. These included new types of light roof elements. In the field of bridge construction an interesting arrangement of a city flyover was shown as well as a special pipeline bridge. A multi-track railway bridge was erected by a unique technology which can be called 'transverse launching'. In the centre of the presentation, large slip-formed structures were introduced. Hydraulic structures, very tall columns and piers, giant cooling towers, chimneys were illustrated. As an example, we would refer to a tower-like building for a malt factory. It is also to be noted that Hungarian engineers contributed to dozens of huge slip-formed structures abroad, a good example being the 200 m tall clover-shaped chimneys in Yokohama (*Fig. 13*).

6. OTHER WORKS OF THE HUNGARIAN FIP GROUP

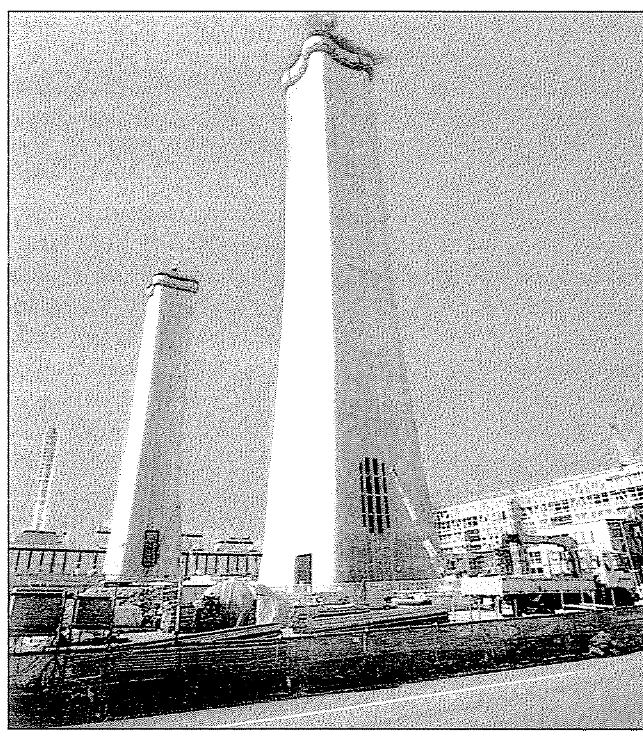
Hungarian experts carried out many important studies in various commissions of FIP. The Hungarian contribution to the Commission for 'Prestressing Steel and Prestressing Systems', as well as 'Prefabrication' was extremely important. – This space is not enough to enumerate the Hungarian papers presented at eight symposia. The FIP Symposium '92 in Budapest was a summit of the activity of the Hungarian Group of FIP which not only gave a possibility to speak about theoretical and practical results but also to show the Hungarian achievements of structural concrete industry in the reality.

7. CLOSING

The Hungarian concrete industry can draw strength from its past. Hopefully, under much better conditions than some historical periods, the future may bring new and rich results in research and development as well as in practical construction in Hungary. The Hungarian Group of *fib* will do its best to report about the achievements at the future *fib* meetings.

Géza Tassi (*1925*), Civil Engineer, Cand. of Tech. Sc., Dr.-techn. (1960), Doctor of Tech. Sc. (1976), Engineer in different positions of building industry. He has been teaching at the Tech. Univ. of Budapest covering almost all subjects in concrete construction. He was Head of Laboratory for concrete structures (1974-1991), has been a full Professor since 1976 and is semi-retired since 1994. He participated in ten congresses and many other meetings, symposia of FIP, plenary sessions and other conventions of CEB and other international professional associations. As an author of more than 220 publications, he has lectured in 38 countries on five continents. He is also Honorary Lifetime President of Hungarian Group of *fib*. FIP Medallist (1992).

Fig. 13 Chimneys in Japan constructed in cooperation with Hungarian enginers



ONE OF THE LONGEST PRESTRESSED CONCRETE RAILWAY BRIDGES OF EUROPE –

executed by incremental launching method in Hungary


Péter Wellner – Tamás Mihalek

An agreement was concluded between the two neighbouring countries, Hungary and Slovenia, for restoring the previous railway connection. For this new railway line The Hungarian Railway Co. (MÁV) issued an invitation for an international tender for the design and construction of a 1,400 m long railway bridge, with ballast and 160 km/h planned velocity for the trains.

Keywords: incremental launching, prestressing tendon, pile, pier, construction deck, lifting-pushing jack.

Type of structure:	multispan, continuous, one-cell prestressed concrete box girder
Bridge length:	1400 m
Width:	8.10 m
Designer:	Technical Department of HÍDÉPÍTŐ Co.
Contractor:	HÍDÉPÍTŐ Co.
Construction period:	May 1999 – June 2000

1. INTRODUCTION

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

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

Last but not least a very important respect was the short construction time of 11 months. If the construction proceeded from both ends of the bridge, parallel work was achievable and time could be saved.

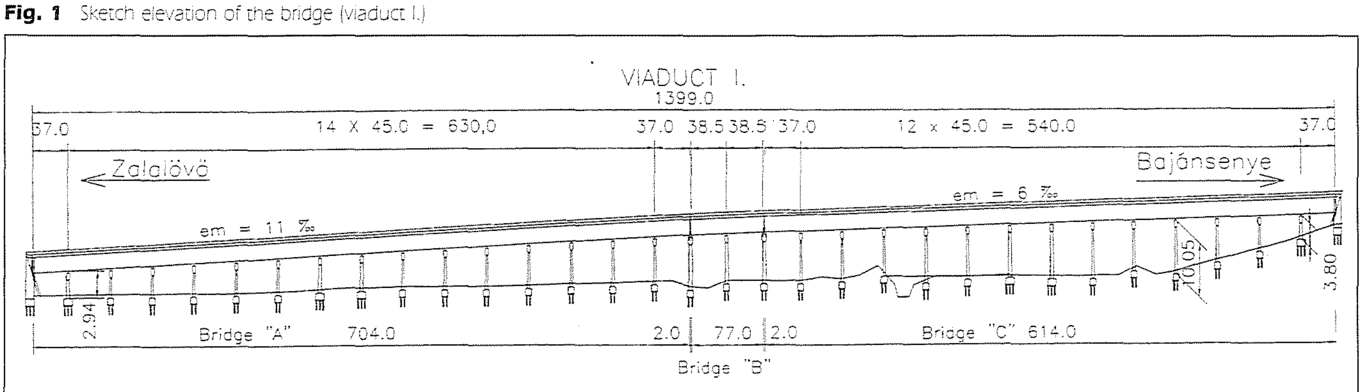
The solution was as follows:
A 704 m long straight viaduct with an 11 permill ascend from the direction of Zalalövő (designated part "A") and a 614 m long curved viaduct (with 2400 m radius) with a 6 permill descend from the direction of Bajánsenye (part "C") were built (see the cover). These viaducts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

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

2. SUBSTRUCTURES

The viaduct I has 31 piers and 2 abutments (*Fig. 1*), they have deep foundation with bored piles of large diameter using SOIL-

Fig. 1 Sketch elevation of the bridge (viaduct I.)



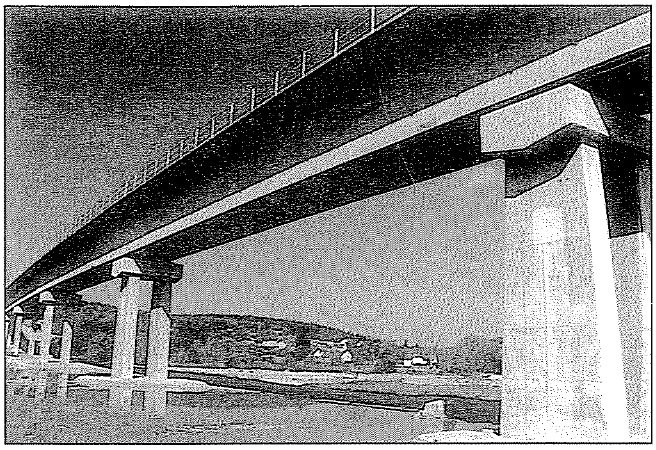


Fig. 2 Side view of pier and box girder

MEC technology Their diameter is 1,200 mm, their length varied between 18 and 31 meters.

The piers consist of two 0.9 m thick rectangular pier walls (*Fig. 2*) which were placed directly under the bearings. Looking at their side view the pier walls have a 1:20 slope. The neck of the pier-walls, which connects to the structural beam is 2.00 m wide at the normal piers, and 3.00 m at the fixed, pushing and common piers. Following the change of the terrain level, the heights of the pier-walls vary between 3.80 m and 10.05 m.

The caps were constructed on the top of the pier-walls. The pier-walls are directly under the axis of the .bearings, the structural beams suffer no bending moment from the vertical loads. Their sizes were determined mainly by the demands of the launching technology of the superstructure. We designed bearing-stools on the caps. During the construction the sliding equipment was placed on them. When the superstructure was completed, the final bearings were placed on them. Beside the bearing-stools a suitable place was ensured which is enough for hydraulic lifting jacks to be placed there at any time during construction, if needed; when placing the final bearings, and later in case of changing the bearings. On the piers two different bearings were placed: one, which allows all direction movements, and another allowing only the longitudinal movements, but which resists the perpendicular forces. Both types were MAURER bearings with PTFE sliding surfaces.

3. SUPERSTRUCTURE

To lead the one-track railway line through, a one-cell prestressed concrete box girder was chosen for the superstructure. The cross-setion of superstructure is shown in *Fig. 3*.

The segment-lengths, mostly of half span, 22.50m, ensured the most elements of the same type. Generally we had to construct two types of segments: above the supports and between them.

The bottom and top slabs are 250-260 mm thick.

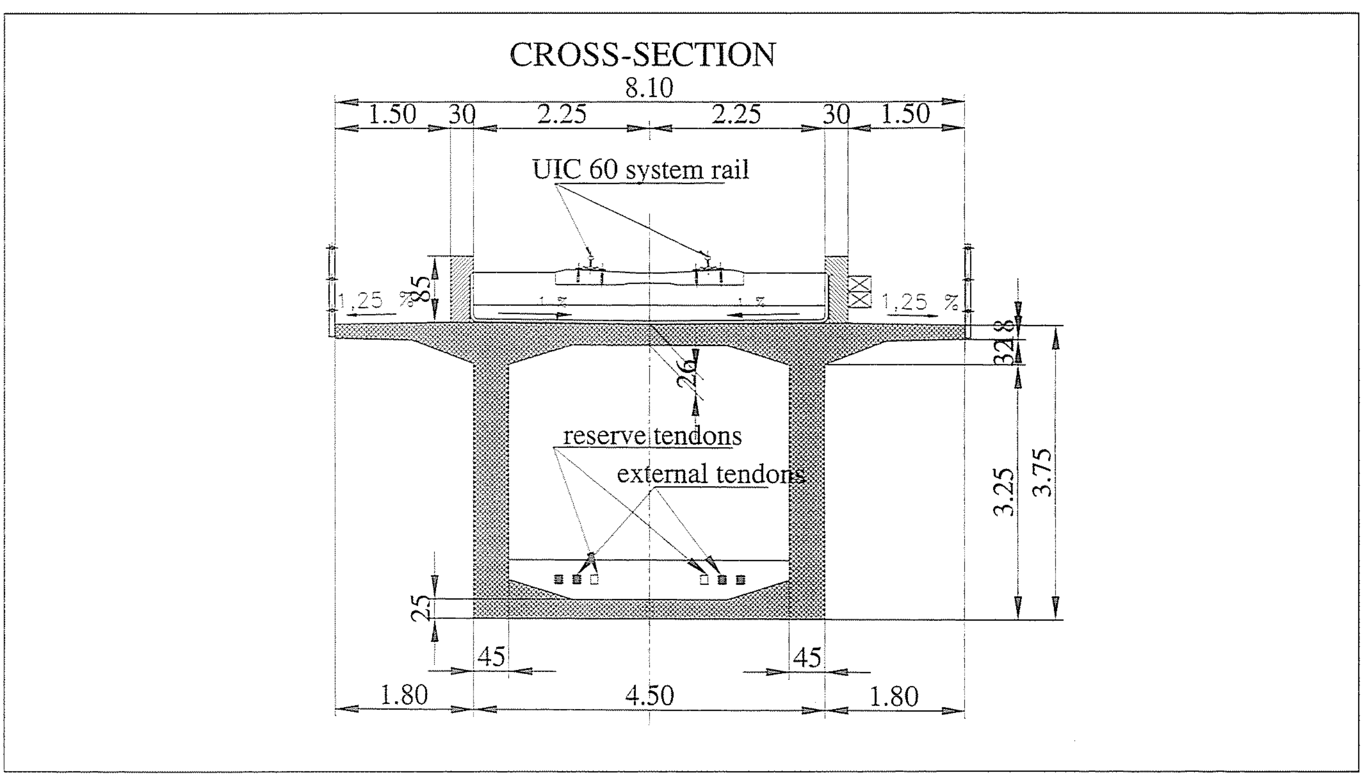
The designed strenght class of the concrete of the superstructure was C35/45-24/semiplastic.

The segments were produced in construction beds behind the abutments. When the concrete reached its required strength, they were connected to the completed part of the viaduct with prestressing tendons. The superstructure was moved forward parallel to the axis by lifting-pushing jacks placed on the top of the pushing piers (two for each bridge part). On the top of the other piers the superstructure slid on built-in slipping equipment, on PTFE sheets fed by hand.

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

Beside these straight tendons, curved tendons were placed in vertical alignment and straight tendons were designed into the webs as well. Each of these tendons consists of 15 prestressing strands of 0.6" diameter and are to St 1630/1860 grade. The tendons were anchored into DYWIDAG MA 6815 (DSI, 1998) type anchoring heads with ribbed outside surface.

Fig. 3 Cross section of superstructure



Beside the two curved tendons in both webs, two straight tendons were also placed in the middle section of the viaduct parts at the height of the centre of gravity of the cross section. The purpose of these is to take the braking forces of the railway vehicles.

To carry the live load of the railway, "external cables" are used inside the box, led through bottom and top deviators.

These cables were composed of four pieces of double protected VT-CMM 04-050 D tendons (each containing four strands), a product of the company VORSPANNTECHNIK. Therefore, each tendon consists of 4×4 pieces St 1570/1770 strands. The low frictional resistance of the strands is ensured by the grease layer mixed with graphite inside the inner protecting plastic tube. They were anchored in VT-CMM 16×150 type anchoring heads.

Construction state

The condition of prestressing the superstructure is to achieve at least 26 MPa compressive strength of the test cubes. During the preparatory works of the prestressing this value increased up to 28 MPa, which meant C25/30 grade concrete. While the segments were concreted in two phases (1st phase: bottom slab and webs, 2nd phase: top slab), this compressive strength value refers to the 2nd phase that was cast later. In the statica calculations we checked the stresses originated from the following loads:

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

The condition of suitability of a prestressed structure is that the stress in the concrete should not reach the permissible stress. In case of incremental launching the superstructure is not homogenous because of the contact surfaces between the segments. According to the design practice at these joints half of the value of the permissible stress of the homogenous concrete can be taken into consideration. It means $\sigma_p = 0.5 \times 1.6$ MPa $= 0.8$ MPa (considering C25/30 younger concrete) permissible stress in the concrete in the tensile part of a flexural girder in the construction state. This condition was satisfied in our structure in all phases.

Launching with lifting-pushing jacks

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

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

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

Launching nose

To reduce the cantilever moments of the concrete girder a steel nose is applied to the front of it when using the incremental launching technology. 6+6 Ø36 mm DYWIDAG prestressing rods in the top slab ensured the connection between the superstructure and the nose and 4+4 prestressing tendons (12×0.6" strands each) in the bottom slab. The shear force is taken by steel boxes (shearing teeth) welded on the end face of the nose and concreted to the first segment.

The length of the nose is 32m, which is about 70% of the longest span. The height at the end connecting to the superstructure is the same as that of the box: 3.75m. Its weight is cca. 1,000 kN. A nose consists of two main girders and cross binding truss between them. A main girder consists of three transportable parts. These parts are connected to one another by prestressing rods. The deflection of the front of the nose is cca. 8cm, when it reaches the next pier. This deflection is lifted back by a hydraulic jack on the front of the nose.

Lateral guidance

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

In the construction state of the curved part the radial forces at the supports had to be taken into account as well. For this purpose lateral guidance with vertical PTFE plates were fixed to the outer side of the sliding equipment on the construction beams of the piers.

The maximum lateral force became $F_{l,max}$=96.6 kN (at pier KP21). The load bearing capacity of the lateral guidance fixed to steel frames constructed around the bearing stools on the top of the caps, were $F_{l,c}$=564 kN, at each support.

Construction bed

As part of the incremental launching technology, the superstructures of the viaducts were manufactured in the construction beds (Fig. 4 background). The construction decks of parts "A" and "C" were behind their abutments (H1 and H33). While bridge "C" is a curved structure, its construction bed had the same curved shape.

A construction bed consisted of the following parts:

- foundation
- reinforced concrete grid,
- bottom formwork,
- outside formwork,
- inside formwork.

The deep foundation of the construction bed consisted of 6×4=24 dia 600mm Franki piles (depth=16m), with 800mm RC pile caps on them. It was designed in such a way, that in case of any occasional settlement it could be re-adjusted to the original, designed position. In case of settlement the grid was lifted back with hydraulic jacks, and could be fixed again by the re-adjustment of the wedges. (We would mention, that the settlement of the construction bed took place during the manufacturing of the first two segments of the superstructure following which no re-adjustment was needed.)

The bottom formwork was supported on the cross beams with 500 kN load capacity steel wedges, and could be sunk

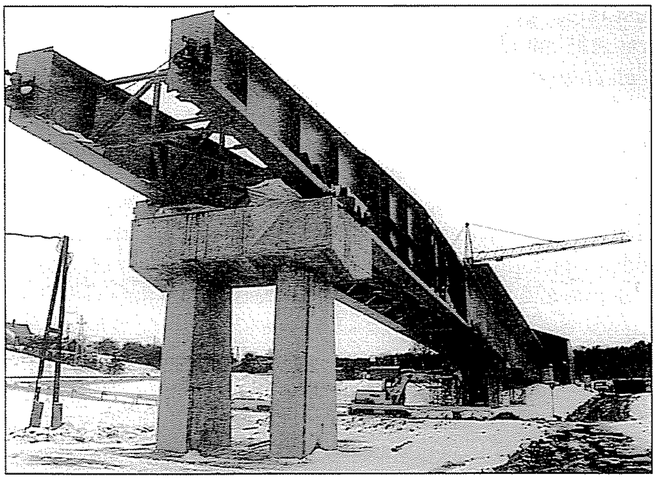


Fig. 4 Construction deck of Viaduct I

down and lifted up by hydraulic jacks. This procedure was needed because of the following. To spare time the reinforcement of the first concreting phase was assembled completely behind the construction bed, and was pushed into its final place on steel U-section rails on Hünnebeck rollers. When the rails and the rollers were pushed back, the bottom formwork was needed to be sunk down. (During this time the reinforcement was supported by the longitudinal beams.)

The structure of the outside formwork was determined by the dismantling method. The common used tilting method could not be used here because of the curvature of bridge "C", we had to pull out the formwork parallelly with the help of hydraulic equipment. The outside formwork consisted of 2×10 parts, each made of 6mm thick steel plates stiffened by cold drawn sections. We left 4mm gaps between them. The curvature of the outside formwork of part "C" was created so that these gaps were closed to 2mm on the inner side of the curve and were opened to 6mm on the outer side. The gaps were filled with rubber strips. The formwork plate of the cantilever of the box was connected to the vertical plate with a hinge. Both plates were supported by adjustable skew rods with screw shafts.

The formwork of the second concreting phase (the floor slab) was created so that when dismantling all parts could be easily carried out of the box by hand. It consisted of lightweight girders and 22mm thick wooden plates. The girders were placed at every metre, supported by steel structures fixed on the completed webs of the superstructure.

The construction period for one 22.5 m long segment was 6 days, the launching process was on the 7^{th} day.

4. ANALYSES FOR DIFFERENT LOAD CASES

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

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

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

According to the Hungarian Standards the ultimate forces on a bridge depend on the length of the bridge, but has a maximum of 6,000 kN. So in our case, the maximum forces to be resisted are 6,000 kN in case of bridges "A" and "C" (that is why we applied two fix piers at each parts, sharing the force between them), 1,580 kN at bridge "B", and 4,000 kN in case of Viaduct II.

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

5. CONCLUSIONS

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

The viaducts were designed from tender design to shop-drawings by the Technical Department of Hídépítő Co. with the help of STABIL PLAN GmbH. During the design works, not only the dimensioning of the structure was to be considered, but also the construction technology and the applied equipment. As a consequence the data of the railway was slightly modified according to the requirements of the construction technology.

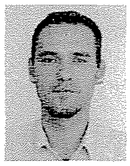
The structure was analysed not only during the construction stages but also in its final state. Beside the general dimensioning we had a special task with the examination of the curved viaduct part during the launching and when finally in place. We had to determine the influence of the lateral forces,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

Péter WELLNER (1933), M. Eng. is Head of Technical Department at Hídépítő Co. Designing of prestressed 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 cantilevered monolitic bridges. He also took part construction of cast in situ concreting balanced in Hungary. The incremental launching technology was initiated in Hungary under his direction. Such structures are now continuously used. He is a member of the Hungarian Group of *fib*, he was awarded the Palotás prize.

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

APPLICATION OF PRECAST PRESTRESSED GIRDERS IN BRIDGE CONSTRUCTION





Adrián Horváth – Viktor László – Tamás Németh

A new type of precast prestressed concrete girder has been developed and applied in motorway bridges in Hungary. Since really continuous structures can be built using them, this signifies a considerable advancement in the area of bridge construction.

Keywords: precast, prestressed, girders, bridge, continuous, multi-span

Type of structure:	New developed **ITG** precast prestressed bridge girder
Typical dimensions (*Fig. 1*):	
Cross-section:	"I" shaped
Height of girder:	900 mm
Width of upper flange:	612 mm
Depth of upper flange:	283 mm
Width of lower flange:	364 mm
Depth of lower flange:	265 mm
Width of web:	160 mm
Max. length:	28.80 m
Designed/ Developed by:	FŐMTERV Co.
Manufacturer:	VSTR-HUNGÁRIA Reinforced Concrete Manufacturing Ltd.
Development period:	2001-2002
Data of applications:	see Chapter 5

1. INTRODUCTION

Much experience has been gained from the application of precast prestressed bridge girders over the last decades in Hungary thanks to their frequent use, mainly in overpasses, all around the country. FÖMTERV Co. has developed a new type of bridge girder that eliminates the disadvantages of former designs.

2. APPLICATION OF ITG GIRDERS COMPOSITE STRUCTURE

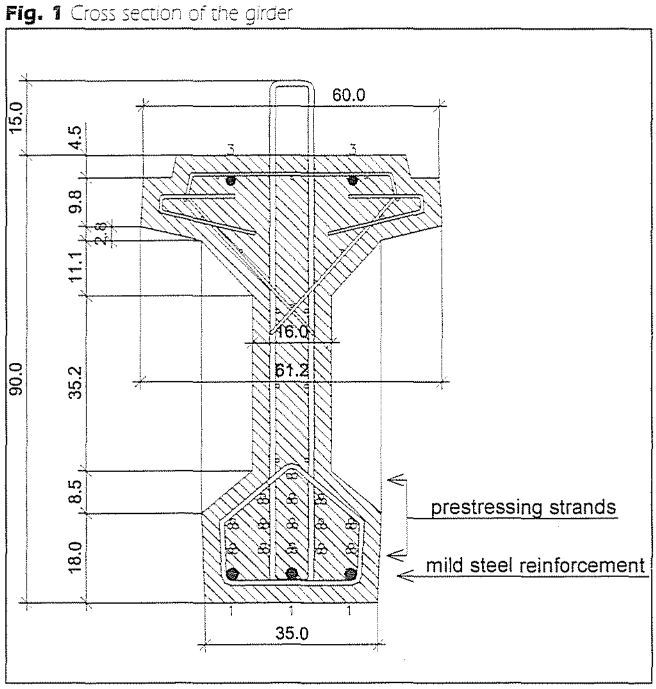
ITG girders can be applied both in simply supported and continuous multi-span structures, as well as in both straight and skew bridges. Their total depth is 90 cm, and they make a concrete-concrete composite structure with an at least 20 cm thick cast in-situ concrete slab (*Fig. 4*). Its important economic advantage is in simply supported bridge structures where as much as 28.80 m span-length can be achieved, which is longer than any other current girder with the same depth.

3. ADVANTAGES OF ITG GIRDERS

The **ITG** bridge girder – thanks to its mixed reinforcement - has significant technical advantages: in other girders there are prestressing strands in the lowest row of reinforcement but in the **ITG** girder there is only mild steel reinforcement. Hence the lowest steel bars that are at the most corrosion-sensitive place of the girder have much lower stress-levels. Calculations show that crack width is also favourable and is far under limit value. On the basis of these factors **ITG** girders are expected to have longer lifetime characteristics.

At tensioning, less prestressing force is needed in tendons because the mild steel bars also take part in transfering tension

Fig. 1 Cross section of the girder



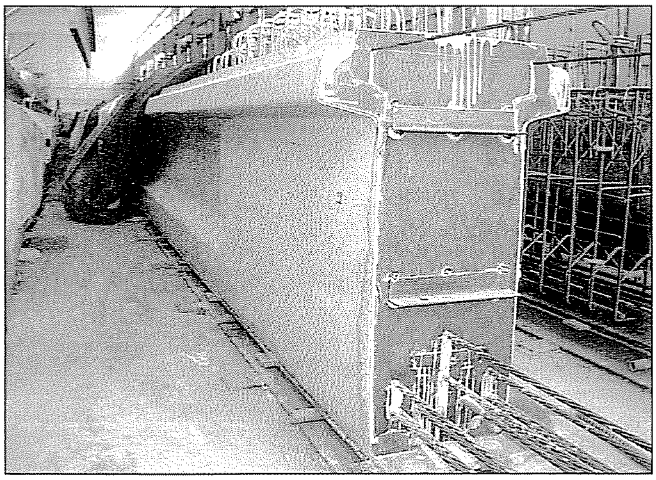


Fig. 2 Manufacture of ITG girders

force of the main reinforcement in lower flange of the girder. Therefore the camber of the girder decreases both in vertical and in horizontal directions. Thanks to less prestressing force, the girder behaves favourably towards splitting at it's ends.

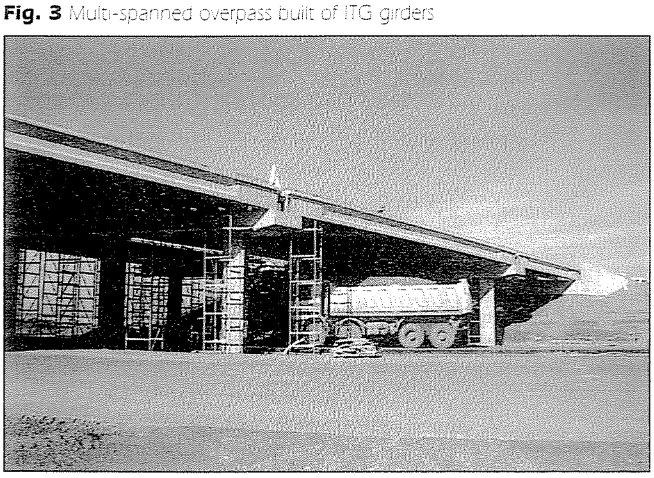
None of the girders available on the domestic market at present, except the **ITG,** can be applied as a continuous multi-span structure for total live loads (details see in Chapter 4 section 2). However **ITG** girders are also suitable for: principal stresses near supports which satisfy the regulations of Hungarian Standard, contrary to other girders. Moreover, the lower flange of the girder can bear compression caused by bending moments above the support. The girder at the prefabrication stand is seen in *Fig. 2*.

4. ECONOMIC EFFICIENCY

Comparing **ITG** girders to other built-in girders in simply supported structures, the use of materials of the **ITG** is more favourable: the area of cross-section and hence demand of concrete is as much as 5-8% less. Moreover **ITG** girders are made with fewer prestressing strands than other girders, although it needs some extra normal reinforcement.

For dead load, both previous and **ITG** girders work as simply supported structures. But in case of other (not continuous) girders, reinforcement of interacting slab is calculated only for service values of live load (and not for total live load) above supports because girders can not bear greater compression in the lower flange. The service value of live load is only 25-40% of total live load depending on the length of bridge. But in reality structures can get greater loads than service load. In this case the concrete slab can crack above supports and this

Fig. 3 Multi-spanned overpass built of ITG girders



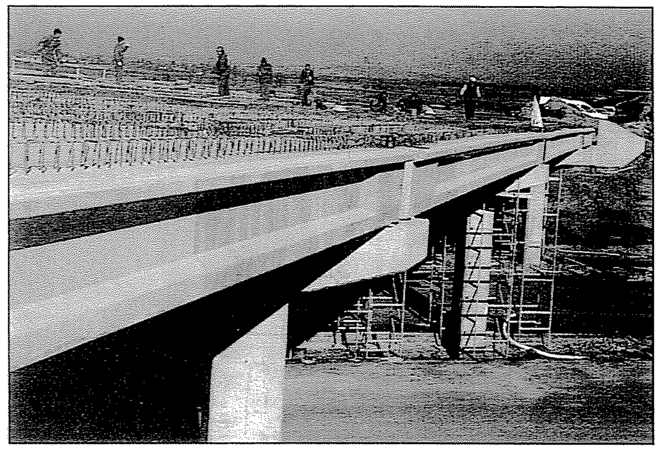


Fig. 4 Reinforcing of in-situ slab

can damage waterproofing, which is essential to duralitity of the structure. Extension joints above supports can be applied to prevent damage to waterproofing but it has a high cost both for investment and during maintenance.

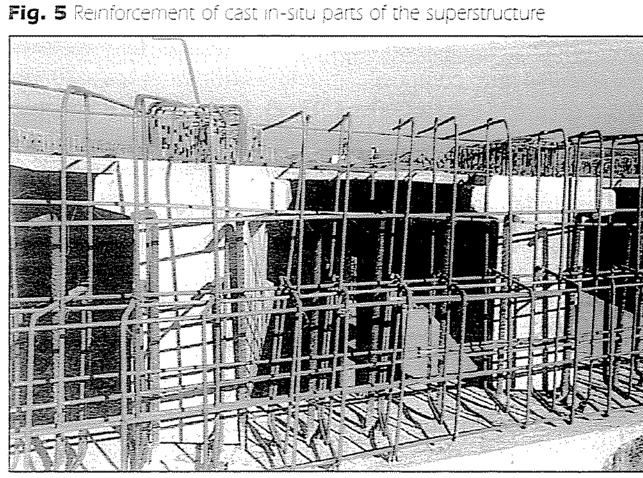
Since **ITG** girders can be calculated and constructed as continuous structures of full value for total live load there is no demand to built-in extension joints and costs can be spared during maintenance as well.

Further advantage of **ITG** girders derives from their greater length because in certain cases intermediate supports can be spared decreasing the cost of the substructure.

5. APPLICATIONS AT STRUCTURES ALREADY CONSTRUCTED AND AT BRIDGES UNDER CONSTRUCTION

Project 1:	M30 highway I. section. No. M30/1 overpass (*Fig. 3*) 2×21.30 + 2×15.10 m long continuous, totally 34.10 m wide overpass
ITG girders applied:	4×21 = 84 pieces
Period of construction:	2001
Client:	ÉKMA Co.
Contractor:	MAHÍD 2000. Co.
Manufacturer:	VSTR-HUNGÁRIA Vasbetongyártó Ltd.
Project 2:	M3 motorway, HB35 overpass

Fig. 5 Reinforcement of cast in-situ parts of the superstructure



	2×18.80 m long continuous, totally 29.8 m wide overpass
ITG girders applied:	4×12 = 48 pieces
Period of construction:	2001
Client:	Magyar Hídépítő Konzorcium
Engineer:	FŐMTERV Co.
Contractor:	MAHÍD 2000. Co.
Manufacturer:	VSTR-HUNGÁRIA Vasbetongyártó Ltd.

6. CONCLUSIONS

The newly developed **ITG** girder is the first and only type in Hungary that can be used in a continuous structure of full value of loads.

ITG girders are
- tougher,
- more durable,
- more economic in use of materials,
- smaller horizontal camber and
- lower in standard deviation of vertical camber

than any other type of bridge girder available in Hungary at present.

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Adrián Horváth (1954) graduated from Budapest University of Technology as a civil engineer in 1979, and completed a two-year postgraduate course in engineering mathematics in 1987. He has worked for FŐMTERV Civil Engineering Consultancy Ltd. since then. His main activity has focused on the engineering of structures for transportation e.g. bridges and underpasses, retaining walls and lately on building and warehouse structures as well. He has extensive experience in directing major complex designing projects. He is the head of Bridge and Structural Engineering Department of FŐMTERV and is the Director of International Relations. He has elaborated and introduced design of cost effective bridge structures and bridge refurbishment technologies in Hungary. As Technical Director of DUNEC Duna Engineering Contracting Ltd. he has also experience in main contracting.

Viktor László (1973) graduated from Budapest University of Technology Structural Branch as a civil engineer in 1996 and has been working for FŐMTERV Co. since then. He has taken part in the engineering and supervision of more than 40 road and railway bridges. He has also acted as structural engineer for many precast warehouse buildings. He played major role in the development of a series of precast reinforced concrete bridge girders.

Tamás Németh (1966) graduated from Budapest University of Technology Structural Branch as a civil engineer in 1990. He has worked for FŐMTERV Civil Engineering Consultancy Ltd. since then. His main activity has focused on the engineering and supervision of structures for transportation and lately on building and warehouse structures as well. Besides this he has experiences in other fields of structural engineering such as aerials and retaining walls. At present he is senior engineer and works as Head of Team in the Department for Bridges and Structures.

HUNGARIAN PRECAST CONCRETE HALL CONSTRUCTION

László Polgár

As a result of need on short construction time, high quality, slender structural elements and favourable price, the market share of prefabricated concrete structures increased during the past four years in Hungary.

Keywords: reinforced concrete, prestressed concrete, prefabrication, Eurocode, globalisation

1. INTRODUCTION

The main features of the development of Hungarian precast concrete structures in building construction over the past 4 years includes:

- the use of precast structures in building projects,
- technical, technological characteristics,
- the effects of globalisation.

2. BUILDING PROJECTS

Precast concrete structures are generally used in the construction of shopping centres, warehouses and industrial manufacturing plants. In Hungary, concrete – especially prestressed concrete – structures are cheaper than those made of steel mainly because in Hungary human resources are still relatively cheap. The majority of the owners are foreign entities – indeed, capital investors from all over the world are willing to come to Hungary. Construction schedules are becoming ever tighter and can only be met by means of using pre-manufactured structures, see *Auchan (Figs. 1 and 2).*

3. TECHNICAL CHARACTERISTICS, DEVELOPMENT TENDENCIES

The demand for greater spans has been increasing. The main limit to the longevity of buildings today is their amortisation.

Fig. 1 AUCHAN, Budaörs

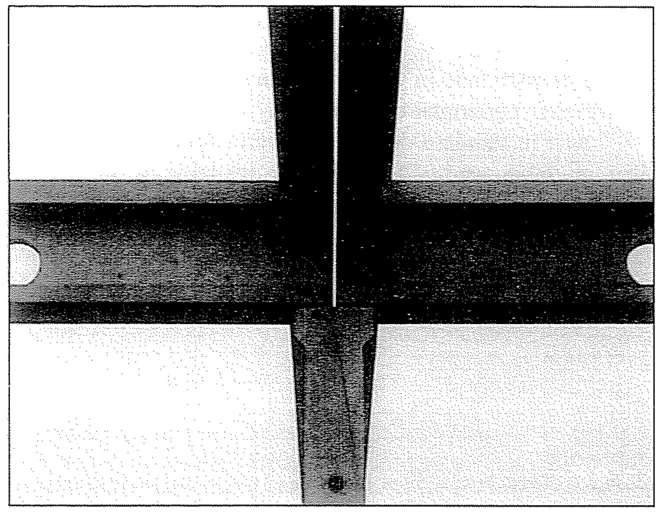


Fig. 2 AUCHAN, Budaörs

Functional demands are changing every 4–6 years. Load-bearing structures with a lifetime of 50 years serve several purposes over this time. In industrial manufacturing plants technology is renewed every 3–5 years and the operating companies are also changing – a commercial structure may be converted into a manufacturing plant then into a warehouse. The larger the span, the more varied the use of the building. A mere 4 years ago most owners required spans between 18–24 m; nowadays spans of 24–34 m are in high demand as *Leoni, Arad (Fig. 3).*

As a result of the increase in span, stronger types of concrete are used. While 4 years ago the most frequent types were C 30/37–C 40/50, today C 50/60 and even C 60/75 are also widely used. The main data of single-storey halls are given in Table 1.

Table 1 Recent commercial – industrial halls

Figure	Name	Town	Grid
1.	AUCHAN	Budaörs	18×26
2.	METRO	Kecskemét	14×21
3.	PRAKTIKER	Budaörs	18×24
4.	ASIA CENTER	Budapest	8×16
5.	LEONI	Arad (Ro)	6×30
6.	VELUX	Fertőd	6×32
7.	HIRTENBERGER	Pápa	6×34
8.	CLARION	Nagykáta	10×27
9.	SANYO	Dorog	7.5×30
10.	ALPINE	Biatorbágy	12×24

Fig. 3 LEONI, Arad

4. THE EFFECTS OF GLOBALISATION

Even though the Eurocode standards have been used by one of the largest Hungarian planning/manufacturing entities since 1993, the EU-standards have not broken through yet.

Eurocode is mainly used by companies with international clients and foreign investments. ASA-PLAN 31 companies operate in Hungary, Romania, Bulgaria, Croatia, the Ukraine and Russia. After the change of political system the construction boom began earlier in Hungary than in her eastern neighbours, so that today ASA-PLAN 31 is in a position to introduce the achievements of the Western construction industry in these countries.

Reinforced concrete structures are exported today just like any other commercial goods. Reinforced concrete elements manufactured in different countries (mainly in Hungary and Rumania) fit together as if they were made in the same manufacturing plant. As a result of Eurocode, national standards are becoming more and more compatible in Europe but gradually a set of world-wide standards will also be developed. There is a high demand for this because of the great number of investors from the USA, Japan and even China.

Let us mention just a few from Asia:
- Takenaka construction company: Clarion, Sanyo, Mitsubishi, Alpine, Suzuki plants,
- ASIA Center was built by Chinese developers,
- Samsung of South Korea.

Today there are practically no language problems – Hungarian and foreign companies working together communicate in English. Differences in the techno-cultural background are more serious. Communication is getting ever easier – thanks to the Internet. Designers can work in any part of the world – structural designers do not have to be stationary any more.

Prefabricated concrete structures are almost identical, e.g. Metro Department Stores are all the same in Hungary, Romania, Bulgaria and Russia.

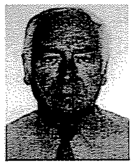
5. CONCLUSION

The balance of the prefabricated structures is getting higher nowadays. This growth is supported by the following:
- fast building erection
- use of high strength concrete
- globalisation, international co-operation
- good fire resistance
- competitive price.

László POLGÁR, MSc (1943), civil engineer; Technical University of Budapest, Faculty of Engineering; 1966 – Foreman in Hódmezővásárhely, 31. sz. ÁÉV (State Building Company No. 31.); 1970–71 Structural Designer, IPARTERV; 1971 – Product developing engineer, chief process engineer, head of the Technical Main Department, 31. ÁÉV; 1992 – managing director, PLAN 31 Mérnök Ltd., managing technical director, ASA Építőipari Ltd.
Activities: design and construction of prefabricated concrete structures and industrial floors. Chairman of the Concrete Section of the Hungarian Building Material Association; member of the Hungarian Group of *fib*.

SPECIAL CONCRETE STAND STRUCTURE





Dr. József Almási - Csaba Bancsik - János Nagy

For a coking plant, a tower-like steel structure had to be built the top of which is at a height of 100m. The paper describes the reinforced concrete structure which supports the steel structure. It is a spatial frame having special foundation and joints. Both the design and the construction was meaning unusual tasks for the responsible engineer, because the extraordinary forces acting on the foundation and the superstructurer.

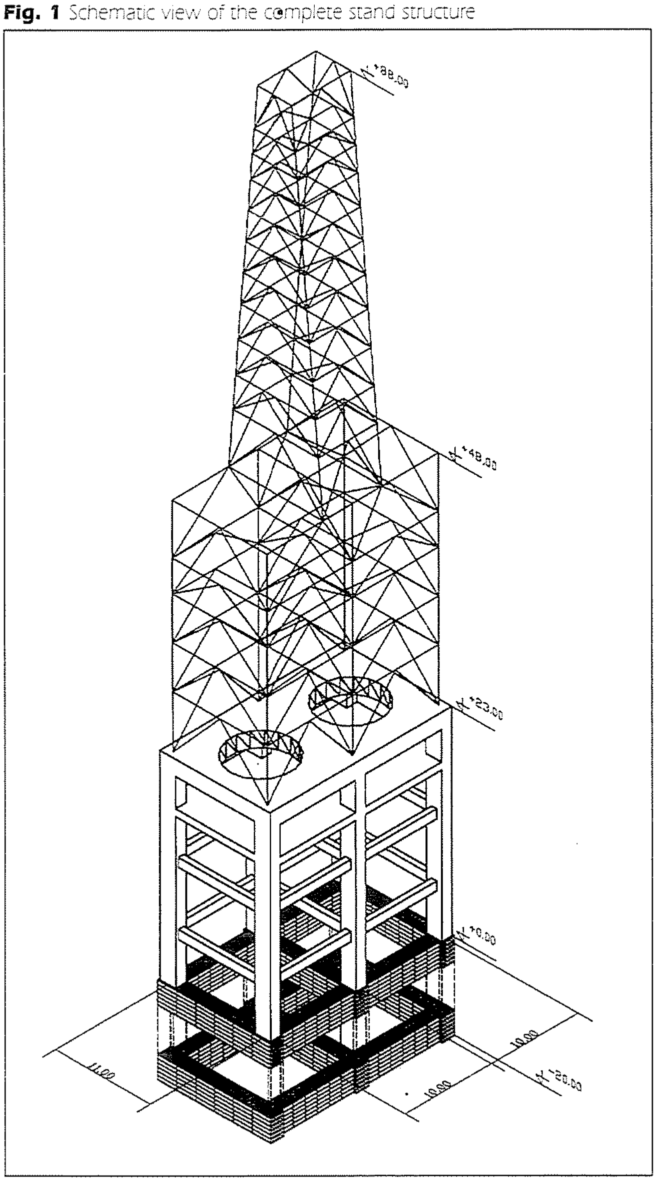
Keywords: Coking plant, special RC stand, joints

1. INTRODUCTION

MOL Inc., Danube Refinery (MOL Co. Dunai Finomító) erected a delayed coking plant for processing by-pass oil.

For placing the complete equipment, a 23.00 m high reinforced concrete stand structure was necessary upon which a 70.00 m high steel structure tower was erected with the process equipment fixed to it.

Fig. 1 Schematic view of the complete stand structure



The conceptual plans for the required deep foundation, reinforced concrete and steel structures were prepared by FOSTER WHEELER IBERIA in co-operation with the process designer.

The detailed design of the reinforced concrete structures were performed by CAEC Ltd. (Cronauer Almási Engineering Consulting Ltd.) under the leadership of OLAJTERV Co. The contractor was VÍZÉP Ltd. who implemented the plans.

Structural solution

Fig. 1 shows the schematic of the complete stand structure. *Three main parts of the structure can be distinguished:*

- *The foundation* is a diaphragm wall system supporting the reinforced concrete frame of box-like design bearing down the terrain level for 20.00 m. The thickness of diaphragm walls in the cross-direction is 1.20 m and that of the long direction is 0.8 m. This 1.2 m thick diaphragm wall was constructed for the first time in Hungary. The type of the dipper was STEIN 810/1200 with 18 t weight.

Welded steel reinforcement was put in place with balances manufactured for this purpose enabling both lifting and tilting.

The ready made diaphragm walls are kept together by a reinforced concrete girder lattice of 2.00 m structural height. The stirrups of the girder lattice permitted mounting of the longitudinal reinforcement from above. Assembly was performed by workers moving within the large girder lattices after which the open stirrups were closed. The precise position of starter bars for columns was given by rigid steel lacing bonds and geodesic checking.

The built-in formwork of the reinforced concrete girder lattice was performed by cleaned slot guide-beams.

Concreting was performed in a single stage without a working joint, in layers according to the "fresh on fresh" principle, with low setting heat cement.

Post treatment consisted of water flooding.

- *The middle part of the structure is a spatial reinforced concrete frame connected to the foundation.* This is located between the site level and +23.00 m level with its two upper levels closed by 80 cm thick reinforced concrete plates of at the +18.00 m level and of 175 cm thick plates at the +23.00 m level. There are two openings of ∅2.1 m on the mentioned plate at +18.00 m level and two openings of ∅6.1 m on the plate at +23.00 m level.
- *The third part of the structure is a steel structure* situated between +23.00 m and +88.11 m levels.

Up to the +48.00 m level it coincides with the plan view dimensions of the reinforced concrete frame, while the upper section follows a narrowing shape.

In the following we provide information with regard to the special reinforced concrete frame structure forming the middle part.

2. THE LOADS AND EFFECTS

The values of loads and effects on structures are determined by the Hungarian standard MSZ 15021/1.

In addition to the usual wind load and temperature effect, loads originating from the process and dynamic effects caused by the operation of the equipment influence the structure comprising the coking plant.

A considerable part of Hungarian territory does not lie above potential earthquake fields and thus generally the maximal load originating from the wind load comprises sufficient reserve, even for the case of earthquakes of the 4-5 MKS intensity.

In the case of this particular structure, taking in account the very high value of the equipment, it was expedient to consider earthquakes of 7 MKS intensity. This fact considerably affected the choice of dimensions and reinforcement system of the structure.

3. THE ANALYSIS OF THE REINFORCED CONCRETE STAND STRUCTURE

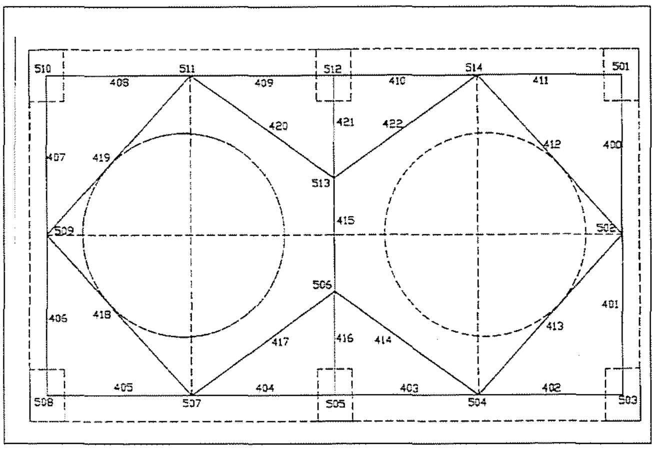
The main elements of the stand leaning on the diaphragm wall system and receiving the loads of the steel structure supporting the equipment are: the frame structure consisting of vertical and horizontal bars and the plates of high thickness situated on the upper two levels.

On these levels the horizontal beams are omitted. Rigid connections of vertical columns' corners and the frame effect are achieved by means of the mentioned 80 cm and respectively 175 cm thick plates.

For the "global" analysis model of the structure in question the substitution with beams is natural. Because of the large openings in the floor plates, it is more useful to substitute "bars" placed in the centres of mass of the remaining "plate bands". *Fig. 2* shows the example of a model used for one of the plate fields.

It is worth mentioning that because of local effects in the area of corner and rend columns, e.g. torsion caused by eccentric connections of beam axes, the problem of introducing reaction forces of the columns into the slab (piercing) required special examination. These models examined the actual design of the structural elements and their compatibility with the armature arrangement.

Fig. 2 *Plate model at +23.00 m level*



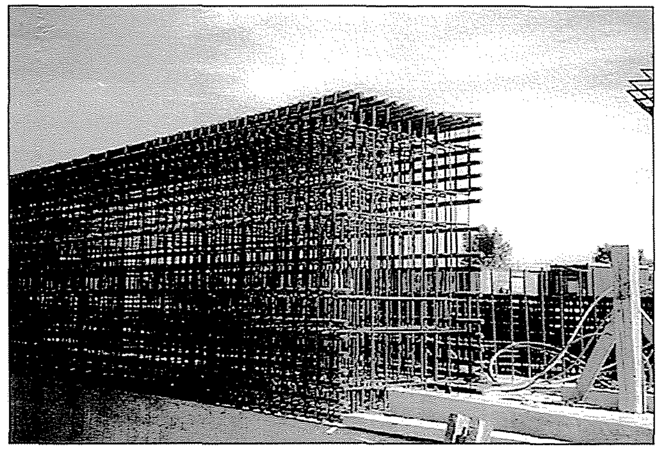


Fig. 3 Reinforcement at the columns

4. METHOD OF CONSTRUCTION

Because of the exceptional large loads and large cross-section dimensions, it became evident that only the in-situ construction method would be cost-effective to meet the structural requirements.

The placing large amounts of reinforcement corresponding to the loads in the beam cross-sections and co-ordinating the concreting units represented a special task (*Fig. 3*).

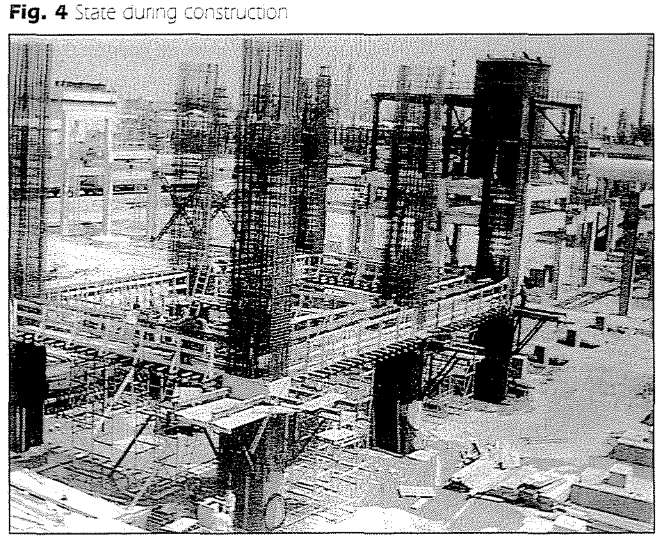
Furthermore, accurate assembly of the armature at the connecting corners and crossings was required.

Accurate forming of the supports and formwork sufficiently rigid to withstand construction together with the selection of the correct camber of the floor formwork was also a distinct task *(Fig. 4)*.

The concreting of a thick floor slab with plastic fibre-reinforced concrete according to a specific technology is also noteworthy. Additionally, the precision positioning of fittings in accordance with the plan was difficult task also. E.g. in the 1.75 m thick top slab (+23.00), fittings had to be positioned within ±3 mm of plan view and vertically that required special fixation systems from the building contractor.

It is also remarkable that the 1.75 m thick plate at 23.00 m was performed in a single step of layered concreting. For the

Fig. 4 State during construction



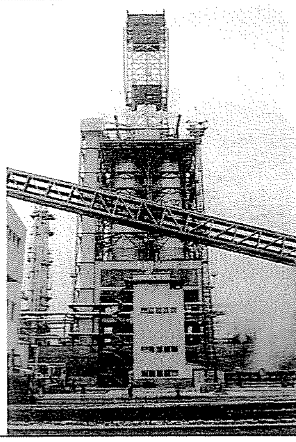


Fig. 5 The completed structures

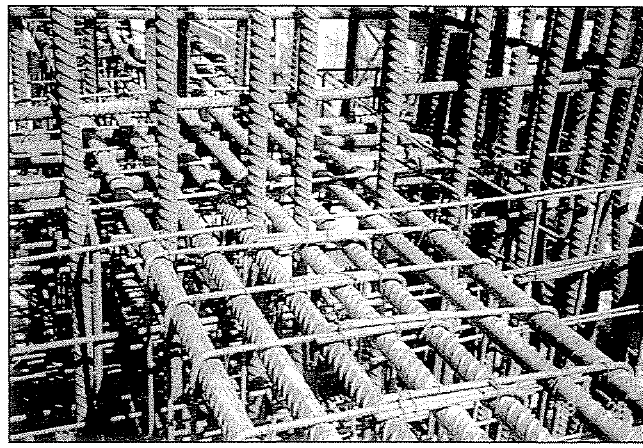


Fig. 6 Connection of the column and the beam

cost-effective construction of the stands required for supporting this large mass, the reinforced concrete frame structure already built was also used for carrying vertical forces *(Fig. 5)*.

5. STRUCTURAL DETAILS

Some structural details of the reinforced concrete stand and some of the solutions are also noteworthy:

- The prefabricated reinforcement of the reinforced concrete columns had to be manufactured on special production benches with steel structure accuracy. This reinforcement also had to meet the requirements of transportation and lifting.
- The lengthwise bars of high columns could not be spliced because of geometric reasons, thus steel bars of exceptional length were ordered, which in turn required special vehicles for their transportation.
- The lifting and fixing of the ready-made column reinforcement required special design considerations in order to avoid deformation. The lifting and positioning was performed under instrumental control.
- Because of the dense reinforcement, the necessary and sufficient concrete coverage required special attention.
- In the given corrosive medium, concrete coverage short of the designed value could not be permitted. At the same time, thicker concrete cover of the reinforcement could lead to cracks and "musseling"-off. Because of this reason, formwork of exceptional accuracy and rigid support were required. In the case of freshly concreted columns tension effects caused by wind and movements had to be restricted.
- Introduction and processing the concrete in high, ready-made column reinforcement without segregation required concrete mixtures with chemical additives. Also the operation of high frequency vibrators in a curtain-like manner were performed for the first time here.
- For protection against thermal damage because of unusual concrete dimensions, low setting heat cement was used to reduce the heat losses of surfaces at night and multi-layer blankets (geotextile, foil) were used.
- The order of assembly of dense columns in the nodes of reinforced concrete pillars, beams and plates was determined in a "sequence instruction" that greatly facilitated methodical material handling and assembly*(Fig. 6)*.

Dr. József Almási (1940) graduated the Civil Engineering Faculty, Technical University of Budapest in 1964, he started his professional activity earlier at Láng Machine Factory, then as an engineer at Mélyépítő Co (1964-66).1967-95 had been teaching concrete structures at the Technical University of Budapest as senior Assist. Professor meanwhile he carried out various design, expert and scientific work. Since 1995 he has been manager of Cronauer-Almási Engineering Consulting (CAEC) Ltd. Member of Hungarian *fib* Group, Palotás Award holder (2002).

János Nagy (1938) graduated the Civil Engineering Faculty, Technical University of Budapest in 1964, in 1992 had a post-graduate education in management (OMEGAGLEN). Honorary Associate Professor at the Technical University of Budapest, from 1964 to 68 design engineer at the Surveying and Geotechnical Engineering Bureau (FTI), since 1968 has been working at Hydraulic Construction Co., presently as general manager of the VÍZÉP Engineering Construction Ltd. Major construction works: Danube port at Baja, Tisza port at Szeged, Port and riverside retaining wall at Óbuda, Tisza Hydraulic Plant Kisköre, Danube Hydraulic Plant Dunakiluti. Member of Hungarian Group of ISSMFE and that of the Hungarian Chamber of Engineers.

Csaba Bancsik (1944) finished his studies as hydraulic engineer at the Baja College of TUB in 1974, has been working mainly as site engineer, major construction works: Sió-canal mouth plant, dwelling estate of Paks nuclear plant, several 3000 cu. m. water towers, Hydraulic plant Dunakiliti, Danube ports at Nagymaros, Gönyű etc. Presently he is active as technical manager of VÍZÉP Engineering Construction Ltd.

BUDAPEST NÉPLIGET COACH TERMINAL AND OFFICE BUILDING

Sándor Pintér – Balázs Vörös

This article outlines the structural design of Budapest Népliget coach terminal and office block. The architectural configuration of the building called for a somewhat unusual solution.

Keywords: uneven settlement, large span trimming, deformation, construction joint, tolerances

1. INTRODUCTION

Construction works for the new coach terminal started in August 2001 following two years of preparatory and planning work. The depot plays a key role in the domestic and international public transportation of Budapest. As the existing bus terminal was situated in the city, in the middle of dense, overloaded communication lines, the concept aimed at transferring it to a more easily accessible site nearer to the outskirts. The new site is at the junction of Könyves Kálmán körút and Üllői út where significant public transport routes meet.

The main purpose of the project is to provide a high standard coach service with an unchecked connection to downtown mass communication. Further, the new building accommodates offices for the transit company. As a result of all this the new facility had to answer rather involved inner functional as well as external relationship demands.

The intricate system posed an extraordinary challenge for the structural design as the placing of office suites above a large space demanding bus and passenger service areas demanded a structural solution unique in Hungary.

2. DESCRIPTION OF THE BUILDING

Three main occupancies are linked within the building and the shape of the building and the supporting structure had to conform to these. While the depot had to be directly linked to the urban bus transport and the underground communication, the building, at the same time, had to meet requirements of an independent administration building. Both occupancies required car parks. On the narrow site, only distributing them on separate levels could satisfy all these functions. These levels are: Basement, ground floor plus four elevations. Attached to and separated by an expansion joint from the building a deep-level garage was built.

The underground part of the building consists of two units. A 2500 m^2 passenger service hall (*Fig. 1*) connecting to undercrossings of the junction, receives departing passengers. Situated here are also the mechanical rooms serving the entire building. A 1500 m^2 deep-level garage adjoins the basement level on the opposite side of the undercrossings.

The basement is entirely sunk underground. Above is the coach station with the parking stands at grade. Part of the waiting hall and the parking stands are situated over the passenger service hall. Above the deep-level garage a steel structure umbrella-roof has been established.

Above the ground floor rises the four storey high office block. The narrow building site and the space demands of the large buses rendered wide-span structure over the coach station for the multi-storey administrative building necessary (*Fig. 2*). A wide-span, column supported frame was designed for the load-bearing structure of the floors, 900 m^2 each.

3. GENERAL DATA

Construction site area	13,998 sq. m
Ground area built in	4,569 sq. m (building+platform roofs)
– Basement level (public traffic + garage)	2,305+2,576 m^2
– Ground floor - public traffic	961 m^2
– First floor – public traffic	957 m^2
– Second floor – office building	664 m^2
– Third floor – office building	654 m^2
– Fourth floor – office building	665 m^2
– Mechanical room	51 m^2
Total useful area in ground plan	7,833 m^2

The developer is Népliget Autóbusz Pályaudvar Ltd. Project Manager István Varga.
General designer: Közlekedés Ltd.
General contractor Hídépítő Co. Project supervisor Balázs Vörös.
Architectural design by A&D Stúdió under the leadership of Prof. Antal Lázár.
Foundation plans by FTV Co. under the leadership of director János Bólya.
Insulation plans were made by consultants "Pataky & Horváth" jointly with FRT RASZTER, architects.
Structural engineering job was carried out by Szigma Stúdió Ltd.
Structural engineer: Sándor Pintér .
Contractor for the supporting frame: Vortex 2000 Ltd. Engineer in Chief: László Michók.

4. REVIEW OF THE STRUCTURE

4.1 Basement structure and foundations

The basement, consisting of two independent units separated by expansion joints is a cast in-situ reinforced concrete structure. Because of the groundwater situation, the building elements had to be waterproofed against ground-water pressure

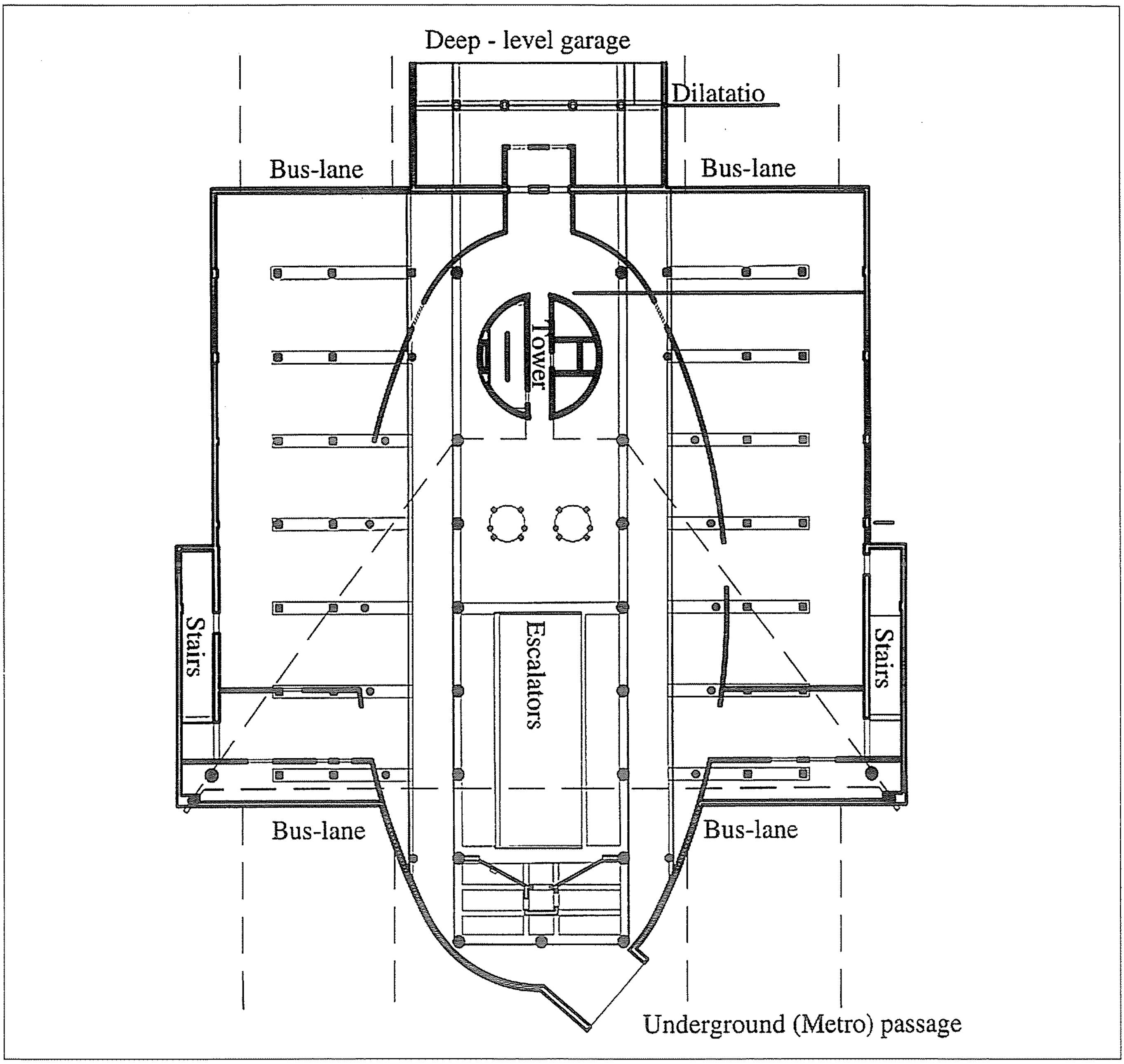


Fig. 1 Layout of the basement of the coach terminal

while the deep-level garage, not being ballasted, had to be provided with negative buoyancy. However, the soil mechanics of the area were advantageous as below 2.5 to 3 m of filled ground, very good, bearing gravel was found.

The single-storey deep level garage received a watertight reinforced concrete slab base and side walls. Its overall dimensions are 85 x 17 m. Because of the pressing completion date, both slab and walls were prepared in stages. The concreting construction joints were made by using SIKA strips. Parallel to the building work of the substructure, dewatering was necessary up to the stage of completion of the floor slab above the basement that provided the needed negative buoyancy.

The 30 cm deep floor structure, made of C20/25 concrete, is multi-point supported inside the perimeter walls with supports spaced at 6 x 6.60 m centres. Part of the deep-level garage is under the bus lanes; here the floor slab is underpinned by more closely spaced pillars at 3 m centres. Above the supporting columns, steel anchoring devices had to be fixed into the floor slab to receive the steel stanchions carrying the umbrella-roof. The impregnable walls were constructed 30 cm thick and the baseplate with 40-45 cm varying depth.

The load-bearing structure of the passenger service part of the basement is a similarly reinforced concrete frame with external reinforced concrete walls. Its overall dimensions are 66×54 m (*Fig. 3*). the principal members of the basement structure are: the circular reinforced concrete columns carrying the structure of the office block above, the slab-and-beam floor with its pillars carrying the bus lanes and the shear tower housing the mechanical service shaft and the staircase. Column spacing is 6.0 m lengthwise, crosswise 5-5.7 m; above, the passenger space span is12.0 m.

Above the large area of the basement is the passenger service area and the bus-course. The heavy loads applied at any point throughout the course (400 kN vehicle load) determined the load-bearing capacity requirement. To meet this, a reinforced concrete multi-span floor slab with 30-40 cm varying depth was made. Cast in-situ reinforced concrete beams carry the slab. The span of every bay of the continuous slab is 6.0 m. The need to control surface runoff determined the gradient of the floor.

Over the passenger service area a 30 cm deep 12 m span, two-way slab was constructed. Situated here and built into the slab covering the basement passenger service space, is the 106

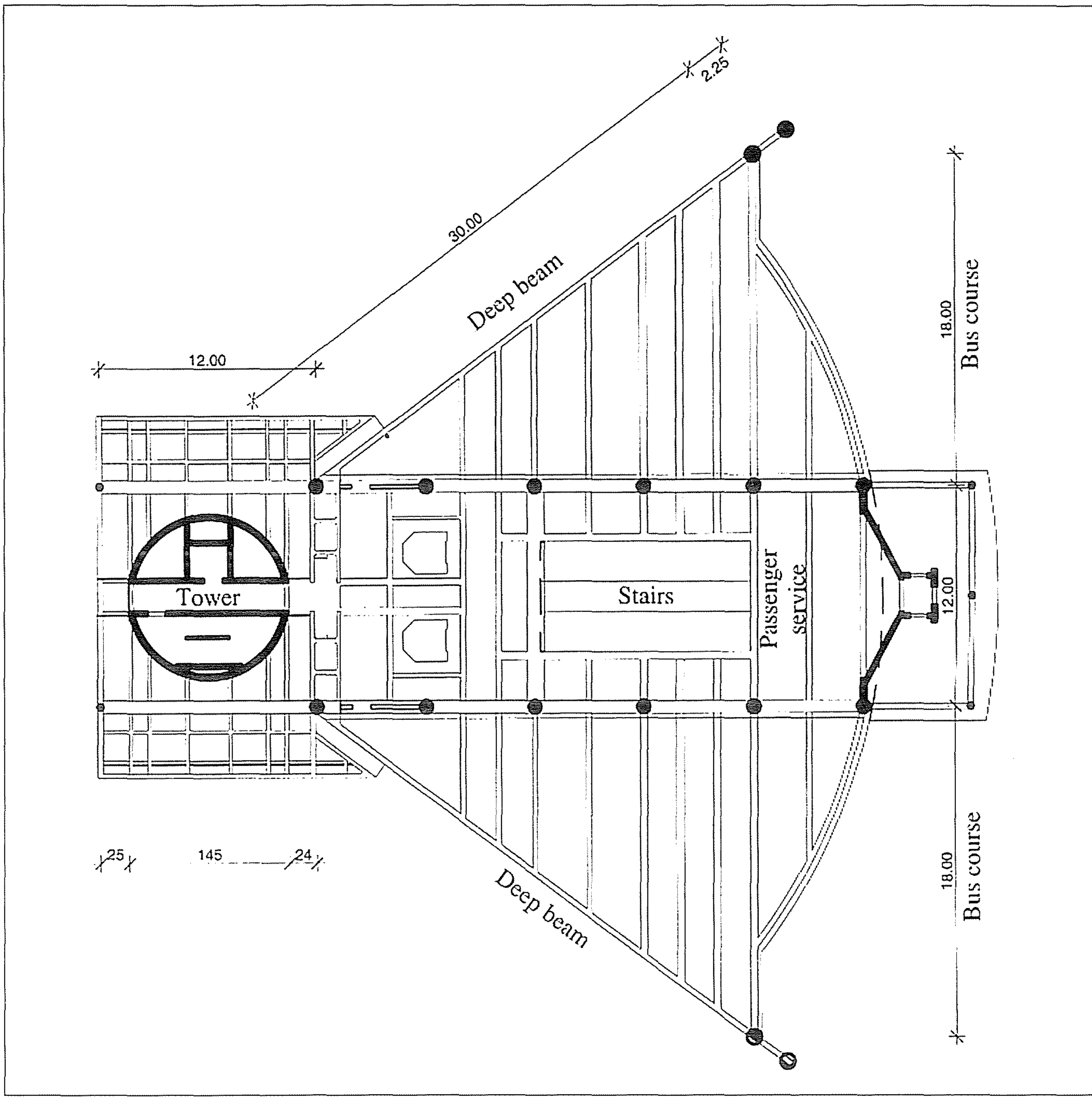
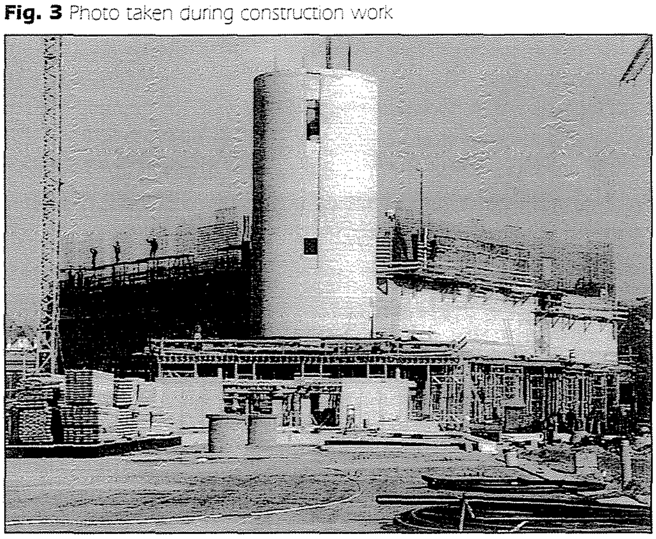


Fig. 2 Structure of the first floor

cm deep, 12.0 m span cast in-situ reinforced concrete beam that carries the rising transversal diaphragm as well as the 18.0 m span main staircase structure. The concrete grade of the slabs is C20/25 and that of the columns is C25/30. The diameter of the columns carrying the load of the administrative building is 70 cm and the pillars under the bus course have a 40×40 cm cross-section.

Fig. 3 Photo taken during construction work



The basement was constructed in the excavated pit, enclosed by the watertight cut-off (diaphragm) walls that were restrained in the subsoil. The occupancy of the basement required strict comfort. Thus the space had to be enveloped with impermeable proofing and in addition the expansion joint connecting to the deep-level garage had to be watertight too. Lateral support of the pit together with the need for dewatering justified the application of cut-off walls. Ground water lowering in the sizeable excavation was – because of the water-permeable soil – reasonably not applicable.

To avoid differential settlement damage to the building frame, bored piles were chosen for the heavily loaded columns while for the foundations of the perimeter walls the diaphragm was used, encircling the pit. Between the foundation piles a 25 cm deep loading slab holds off external hydraulic pressure.

4.2 Superstructure

The architectural configuration of the first (ground) floor determined the superstructure design. The communication zones at ground level and the large area demand of the passenger service space resulted in a structural solution rare in the field of office buildings. We were forced to develop a multi-storey wide-span frame for the office building levels. For economic reasons reinforced concrete ribbed slab skeleton structure was designed.

Serviceable bus traffic demanded an 18.0 m clear span on both sides under the administrative building and the passenger service needed a 12.0 m span space between the two bus-lanes. But the architects, instead of a simple rectangle, conceived a body over a triangular plan gained by chamfering the oblong above the bus-lanes. Besides this the quantity of the office rooms required did not require exploiting the entire floor area upstairs.

Adjusting the structure to the architectural concept, the horizontal bearing structure of the building became a transversally running three-span ribbed slab where the two extreme supports are 30.0 m long, four storey high deep beams (Bölcskei, Orosz 1972; Palotás, 1973), running at an angle. On the flank of the middle 12 m span space, reinforced concrete circular columns stand in 6 files spaced at 6.0 m centres. The central columns support a 50 cm wide main girder. The transversal ribbed slab leans on this girder. The span of this floor slab over the side bus-lane varies along the length of the deep beams which, in turn, are running from an extreme point in an oblique line towards the inside column file.

Side fields of the wide-span ribbed slab have varying spans along the length of the building. Therefore, the main concern was to restrict the diverse deflections generated by the disproportionate spans. The cross-section of the outside girders with the widest span is larger than that of the inside girders with narrower span, with the former more closely spaced than the latter.

With respect to the stiffness the difference had to be equalised also between the various levels above each other of the maximum span front deep beam, because the load of the different floors and geometry of the beams were dissimilar. We considered the maximum deflections – instead of the usual 1/200 - to be equal 1/400 of the span. In addition, we specified that the fixing accessories of the façade glass-wall should tolerate the computed ~2,0 cm differences.

Similar problems arose in the middle bay, too, because hogging was produced in the outside spears however, sagging was produced in the tower - side region. Here again we limited the deflection-differences by shifting the rigidity of the beams.

The thickness of the ribbed slab was 15 cm and the hang of the ribs changed from 90 to 118 cm. The applied concrete grade was C20/25. The contractors completed slabs of a pash level in two stages. The face of the construction joints was perpendicular to the principal compression stress directions and with the beams we specified reinforcing stirrups. We required inserting expanded metal lath into all stop-end joints.

For the large-diameter piping, penetrations had to be provided through the beams hanging under the slab. For these penetrations we designed circular holes encircled by subsidiary reinforcement.

The multi-storey deep-beam supported the beam-ends facing the bus-course (*Fig. 2*). This thin-wall beam was concreted one storey at a time at all four levels, applying only horizontal working gaps with continuous underpinning, dimensioned to the self-weight load of the thin-wall beam and the slabs. This propping had to be carried down through the basement ceiling right down to the ground plate. The thickness of the wall is 30 cm and the concrete design grade C20/25 is identical to that of slabs and beams. The strutting could be removed only after the full hardening of the deep-beam concrete and therefore for the uppermost level we prescribed a stronger C25/30 grade of concrete because of the closeness of the completion date and in order to enable earlier striking of the formwork. It was further justified by the fact that in the final phase of the work the structural builders were obliged to prepare for winter-work. Concreting had to be carried out below freezing point, applying suitable concreting process.

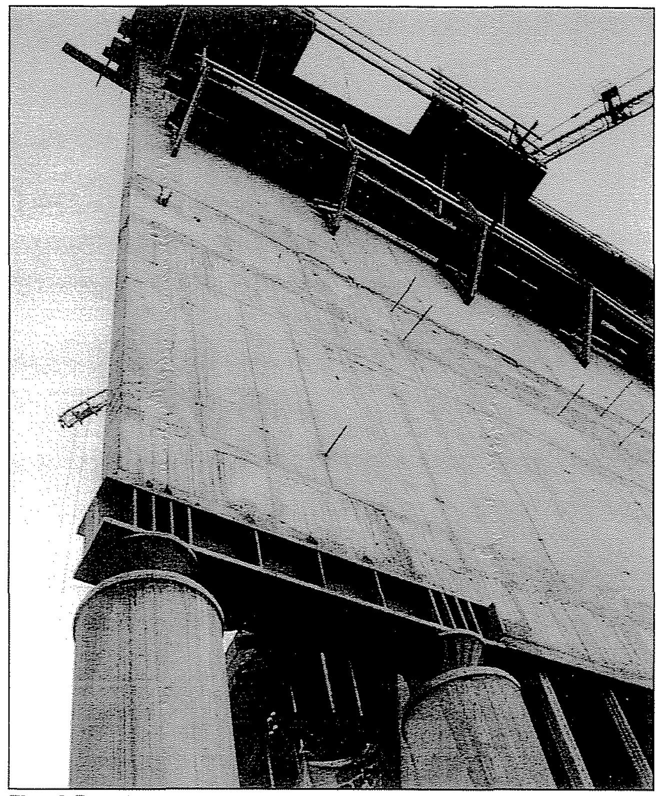


Fig. 4 Deep beam

Support for the facade end of the deep-beam was achieved with a built-in reinforced concrete pier-pair. At the point of concentrated loads, to receive the reaction forces, we designed individual architecturally-shaped steel column crowns with steel load-distributing girder framing. The columns, supporting the deep-beams (*Fig. 4*) are 80 cm diameter and fixed in the basement floor slab. The grade of concrete used for the columns is C30/35 and the main bar diameter is 40 mm. The steel column capital had to be welded to the steel cap embedded in the concrete with a one mm positioning clearance and in accordance with the architectural concept. Accurate line and level setting out of the columns was a basic requirement.

The deep-beams were supported by C30/35 cast-in-situ coloumns at every level.

The 900 m^2 office superstructure is supported at the centre by 7 pairs of columns. These columns have a 70 cm diameter at foundation while at the upper levels the diameter decreases with the diminishing load.

4.3 Stiffening the building

We solved the issue of stiffening of the structure by using reinforced concrete shear walls. As a consequence of the different occupancies above each other, none of the reinforced concrete walls could be built uninterrupted right down to the foundations. Generally their load had to be discharged to columns at some places from beams. Floor slabs transmitted lateral forces between walls, thus all bracing walls functioned also as deep-beams. On the main front levels, beside the shear walls, the frame (constituted by the columns and the ribs of the floor slabs) also played a role in the transverse resistance because of the eccentricity of the stiffness centroid. For the acceptance of heavy loads the columns received double course reinforcement in a number of cases.

The plumbing services tower that is linked to the office block by an access corridor at every level played a decisive role in longitudinal stiffening. The tower accommodated the staircase and the sanitary shaft. This circular tower was erected with a 25 cm thick perimeter wall and the dividing plane walls inside are 20 cm thick. The staircase and the access floors provide local stiffening for the walls. The height of the reinforced concrete tower wall is 27.0 m from the basement level.

The tower structure was built using the slip-form construction method in the first phase of construction. For the associated floors and the staircase to be subsequently built in concavations were made which included reinforcement for connection later. The simultaneous erection of the solid newel, weakened by the mortices (concavations), was unusual. As this was not connected to the other walls of the tower its stability had to be secured by putting in temporary steel braces until the completion of the staircase structure some weeks later. The staircase tower provided stiffness for the building throughout construction.

5. CONSTRUCTION MATTERS

The building-complex of several parts was completed within a short deadline. The building was structurally completed in the period from the beginning of July 2001 to mid-December 2001. During this period numerous construction problems required solutions. The major concerns were: bracing matters related to the deep-beam construction. Arrangement of working joints was adjusted to the working phases with special attention to the imperbeality of wide-span beams. In these cases we designed special connecting pieces and subsidiary reinforcement. The cast in-situ concrete floor slabs joined steel structures with inferior tolerance, while the requirement was millimetre scale precision. Due to the type of formwork used and accurate setting out, the contractor was able to meet this requirement.

The early winter weather caused another difficulty. The concreting process had to be redesigned and it became necessary to use setting and hardening accelerator admixtures while curing had to be carried out under heat-insulated covers. The extended formwork stripping time had to be taken into consideration as well.

6. CONCLUSIONS

The spatial model and strength analysis of the structural components was computed using the Axis 3D finite elements software system. Strength design was carried out in accordance with the

Fig. 5 Perspective view of the completed building

specifications of the Hungarian Standards. With the strength design of the deep-beam and linked beams we also considered the individual loading circumstances arising or happening in use. Such cases are: deformation differentials of the connecting structural members and their back actions on the stresses.

The working drawings had to be prepared simultaneously with the architectural drawings, and with only 6 weeks at our disposal. The building was built to high standards from materials universally used locally (carcassing material C20 – C30 grade concrete and B 60.50 grade reinforcing steel with cast-in-place technology). It was completed and handed over according to schedule.

Since completion no detrimental deformations or cracks have appeared, although utilisation experiences will be known only in after years. This, in our opinion, is owing to the fact that the designed structure met more exacting requirements than is specified for conventional structures. The complete building is shown in *Fig. 5.*

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Sándor Pintér (1970) Architect. Graduated from Budapesti Műszaki Egyetem (Technical University of Budapest.) Faculty of Architecture in 1993. Leading design engineer of Szigma Stúdió Ltd. Participated in the planning of numerous office buildings and dwelling houses.

Balázs Vörös (1963) graduated from the Technical University of Budapest, Faculty of Civil Engineering. He joined the Hidépítő (Bridge Construction) Co. and participated in various significant works from their foundation to full completion. From 1995, he had been head of construction works, later as responsible technical leader for structural investments. Had been heading construction works by incremental launching method and assembly of bridges with precast girders. He was leading the construction of railway overpass using the method which can be called transverse launching. Later, he was involved in construction of commercial and office buildings as well as of the building of the bus station discussed in this paper.

REINFORCED CONCRETE STRUCTURES OF THE NEW NATIONAL THEATRE IN BUDAPEST

Gábor Földvári

The article presents structural composition of the new National Theatre (Fig. 1). Briefly outlines precedents of implementation, represents complexity of the planning process, characterises intricacies and volume of the project, communicates aspects of choosing structural solutions to the reader, then goes on introducing main loadbearing components of the building in detail. Describes the building site, the soil- and subsoil situation, subgrade structural solutions, such as the two different pit enclosures suited to diverse basement depth, the combination watertight bore pile foundation.

Among the superstructures, first the stage block will be shown in conjunction with state of the art stage engineering, then come the varied frames of the floor sections encircling the stage block, involving the flat slab office and changing rooms, the reinforced concrete floors, placed on lost formwork which, in turn, is supported by 14.0 m to 15.0 m span trusses, and the purely steel floors suspended from steel trusses. Major part of the service wing's walls was raised by using slipforming, observing exacting accuracy requirements.

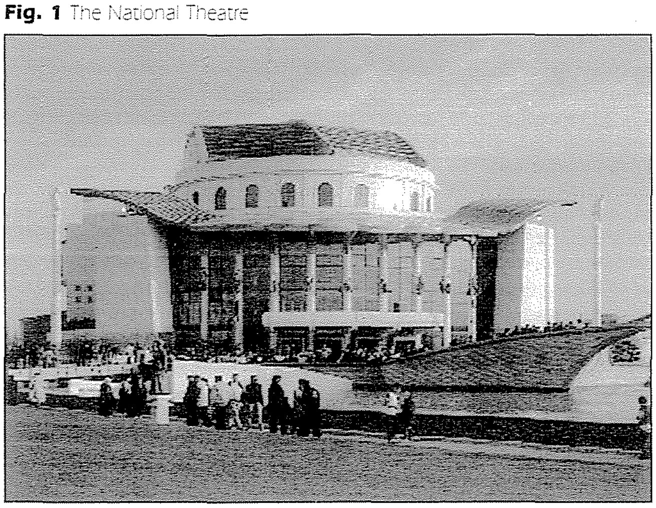
Main parts of the loadbearing structures in the public use wing are curved cast in situ reinforced concrete walls, flat and stepped monolithic floors, and reinforced concrete floors cast into formwork-panels, wide span deep-beams, and the glass canopy/roof, engineered from curved girders. Finally a few notes regarding some interesting individual structures in the surroundings of the theatre.

Keywords: theatre, watertight RC raft, bored pile, slipforming, steel truss supported floor, deep beam

1. PRELIMINARIES

The first National Theatre built in 1837 became obsolete by early XX. century, and was demolished. The company moved into the "Népszínház" (folk theatre) building in the Blaha Lujza square junction and operated there till 1964. Then – in conjunction with the construction of the Metro (Underground) – the building was demolished by blasting. The company of the National Theatre moved into a temporary site once more. There were several attempts made over the following decades to build a permanent home for the National Theatre, but neither of those was accomplished. At last on the 26th August 1999 governmental decree decided building of the National Theatre. György Schwajda, a well-known theatrical personage was entrusted with the implementation of the project and, at the same time, date of the opening premiere was set to the 15th March 2002. *(Fig 1).*

Fig. 1 The National Theatre



2. BRIEF AND DESIGNERS

György Schwajda asked Mária Siklós to prepare the project concept. Mária Siklós was the architect of several successful theatre reconstructions, and thus can be considered to have the most experience in theatre designing in Hungary.

Mária Siklós undertook the project and called in fellow designers tested and tried over earlier theatre reconstructions, thus fell the structural engineering job to Földvári Mérnökiroda.

Planning of a theatre always presents a complex and involved job to all participating parties. This is particularly true in the case of a new theatre, but preparing the structural designs for the theatre of the nation was an exciting challenge for us, in spite of owning considerable experience in planning theatre reconstructions.

As a result of the project's circumstances and the pressing completion term, the structural working drawings had to be provided ahead of the architectural drawings in instalments. Following the issuing of the building permit the construction work was immediately begun; consequently planning went parallel with the Work. These conditions heightened the importance of cooperation between the fellow designers. Planning circumstances were further aggravated by the fact that the result of the theatrical engineering tender became known only after the foundation drawings had been issued, therefore checking structural and theatrical engineering plans against each other had to be carried out subsequently in a very short time.

3. THE WORK AND THE CONTRACTORS

Although this article deals with other aspects of the project, we shouldn't skip a brief introduction to the tune of the building. Building plot of the theatre measures 14621 m^2. Built on area is 4073 m^2, gross total floor area 21668 m^2.

The auditorium can accommodate 610 spectators, the studio 150. Various public use areas surround the two auditoriums in a horseshoe fashion. The Coliseum motive, conjuring up the bygone times of playacting, the fretwork stone balustrade of the single flight grand staircase, the steel-frame glazed roof/canopy, the grandiose glass walls and the glorious panoramatic view of Budapest, all contribute to the festive mood of the public use areas.

The stage block comprises the most advanced stage engineering plant, unique in Europe. On the main stage 72 sink platforms with 1×2 m plan, operated by freely programmable computerisation, 60 brail-line and 26 purchase line upper machinery have been installed.

The U-shape service wing, accommodating also 3 rehearsal spaces, girdles the stage block and includes the scene-dock, the costume room, the changing rooms, the office rooms, the actors' club, the maintenance workshops, rooms for the technical personnel and the mechanical rooms.

An arched steel ornamental roof structure connects the lofty stage block and the public use wing. Constructing the building, covered by limestone and ornamented by pieces of art was awarded to ARCADOM Építőipari Részvénytársaság (building comp.). General Manager Péter Bálint, supervisor Lajos Béres. Numerous Hungarian contractors were included in the Work. Main subcontractors in the reinforced concrete profession were the ZÁÉV Rt, MONÉP Ltd. and ATLASBAU Ltd.

Construction work begun in September 2000, and was completed and handed over in January 2002, thus the entire duration of the building construction was 16 months. A further 4 months were needed for the development of the surroundings, according to the designs of Péter Török landscape architect, carried out by VARPEX Ltd, complete with large water-expanses and individual gardening objects.

4. OVERVIEW OF THE LOADBEARING STRUCTURES

This standalone building consists of two main parts in terms of function as well as of structure: public wing and service wing. Overall plan dimensions of the building are about 55.0× 96.0 m. Characteristic basement level related to finished ground level in the area of the service wing is 4.0 m, in the area of public wing 4.5 and 7.5 m, respectively – both are below design water table.

We designed belowground part of the building without expansion joints, as a single, continuous, cast in situ reinforced concrete unit. We divided structural parts above grade into two parts with an expansion joint, running along the auditorium part of the stage. This way we allowed for the dissimilar thermal motion over and below grade, at the same time avoided the delicate and expensive job of creating expansion joints under groundwater.

The level of the stage designates the ± 0.00 basis level of the building's own level system. This is also the level of the first floor: 110.10 m Baltic sea-level altitude in an absolute elevation system. The finished ground level varies between –4.00 and –6.00 m

The service wing includes the stage block too, it is built around it. The cornice height of the stage block is + 28.5 m, rises approximately 32.5 m to 34.00 m above grade. The entire area of the service wing is vaulted, floor level of basement is –8.0 m. The frontal plane of the sixth floor is checked back, resulting in a reduced footing area, and the appearance of a terrace. The overhouse constructions encircle the stage block on three sides, and thus create a seventh storey, but with an even more restricted floor area.

The public wing has a centralising plan, arranged lobe-shape around the auditorium. This wing is also vaulted. Clear height of the basement varies according to the needs of the various occupancies, floor levels therefore fluctuate between –8.63 m and –11.45 m.

Public wing, enclosing the theatre hall has essentially 6 floors over the ground floor with a variety of floor levels and clearances.

5. STRUCTURAL CONSIDERATIONS

An important aim of the design was economy of space, efficient exploitation of the building body. Consequently the planners endeavoured to adjust structures to various clearances and spans required by the diverse functions, to avoid dead spaces, furthermore to exploit the spaces between the girders by running, for instance, the service conduits there.

The already mentioned complexity and diversity of the building does not lend itself to prefabrication. On the other hand there is already a significant number of contractors, who operate state of the art formwork systems for in situ cast reinforced concrete work. When working on the organisation of the construction work, we had to deal with the considerable labour and cost of the shoring that supports the shuttering under the concrete of the floor slab. We reduced these demands by using steel trusses especially for the major spans high above the floor, and profiled steel sheets as lost formwork.

6. LOCATION, SUBSOIL AND FOUNDATIONS

6.1 Characteristics of the building site

Following deliberations over several siting variants, the National Theatre was built on an almost triangular plot, the no. 9. land parcel, inside the projected Millennium City Centre, to be established between the Petőfi and Lágymányosi bridges in district IX on the Pest side. Filling up this part of the Danube riverside began in the eighteen seventies, bulk of the job was done around the turn of the century. This area, including tracks, workshops, underground tanks, and conduits, belonged to the Ferencvárosi railway station for decades.

The vertical loadbearing structures of the building, standing some 40 m from the riverside are partially walls and in part columns. Major column loads appear in the public use wing (1425-3225 kN), column loads in the service wing are less (500-900 kN). Loads on the walls are also diverse, maximum load is 1124 kN/linewar metre.

Subsoil conditions of the building can be summarised as follows:

Under the original ~104.0 m Baltic sea-level altitude a 3.0 – 5.7 m deep rather mixed stratum can be found with densities ranging from the very loose to medium density. Top layer consists of slag, coal dust (!), slag-detritus-sand mix, railway crushed stone, lower down a zone of sand (and gravel), but we also found remains of old building parts and public utilities. The largest piece was found when building the ziggurat in the garden around the theatre. Purpose of this several metres high masonry "bunker" remained unknown. It was probably used for storaging Danube ice, cut in wintertime. Presence of missiles from the Second World War in the mixed filling could not be excluded either. Rehearsals in the already completed building were in progress, when the bomb-disposal people had to detonate the bomb found during the earthwork of the surrounds.

Below the filling, generally downwards from 99.0 m Baltic sea-level altitude appeared the 7-10 m deep sandy gravel Danube terrace, with significantly varying compactness. Dynamic soundings were made to map the distribution of the varying solidities.

From under 13-14 m belowground depth, that is at 90-91 m Baltic sea-level altitude appeared the upper Oligocene seat, the grey sandy medium clay, very compact bedding, hard fundamental rock that can be considered impermeable.

According to soil mechanics and hydrogeology effect-studies, movement of the non-corrosive soil-water is governed, with a 2-3 days delay, by the Danube flow regime.

Thus maximum groundwater table assessed with a 1% deviation probability is at 102.5 m Baltic sea-level altitude, resulting in a 103.0 m Baltic sea-level altitude design water table. We used cutoff walls partially, only to close the building pit of the deepest basement section. The calculated backwater generated by this is very little: +20 cm.

For our planning work we reckonned with a 5 % highwater frequency at –9.43 – 10.73 m relative level (99.4 – 100.7 Baltic sea-level altitude). It was possible to construct the basements in general with floor levels -8.45, -9.95 m under dry conditions, but the –11.75 m floor level basement had to be protected from flooding by applying impregnable cutoff wall enclosure.

6.2 Excavation enclosure

It was sufficient to secure the open pit of the basements of general floor level by a 4/4 incline. The deep basement received a 55 cm nominal thickness cutoff wall enclosure, encastré 0.7 m deep in the watertight seat, with temporary – in major part single line – anchoring.

The cutoff wall contributes to the transfer of vertical loads, as the loadbearing revetment wall, springing from the baseplate, expands over the head of the diaphragm and bear on the same by a triangular armoured abutment.

6.3 Foundations

The building has a combination watertight reinforced concrete raft and bored pile foundation, designed by Miklós Juhász (Taupe Ltd.). The piles are restrained into the slab base directly, without girder lattice. This, by the piles providing point support, enables deep-wall-like structural behaviour in the reinforced concrete walls, standing in the base plate.

Thickness of the slab base under the service wing is 45 cm and 60 cm under the public wing. Diameter of the piles bored by a continuous spiral is 60, 80, and 90 cm, depending on the load. Under large-area basement rooms we also applied embedment piles to lessen the need for extensively thick loading slab. The construction joints of the base plate were fitted with expanded steel and joint band, the moment armature bars were run continuously through.

Noteworthy is the problem of incorporating the sheath tubes for the working barrel of the spectacle elevators. This demand taxed both designers and contractors. Namely an impermeable casing had to be fitted into a casing adapter reaching down a further 8.0 m under the boss of the elevator arriving at basement level – all this against water pressure and to strict accuracy requirements.

7. BEARING STRUCTURES OF THE SERVICE WING

The service wing includes the stage block and the other service spaces, offices, changing rooms.

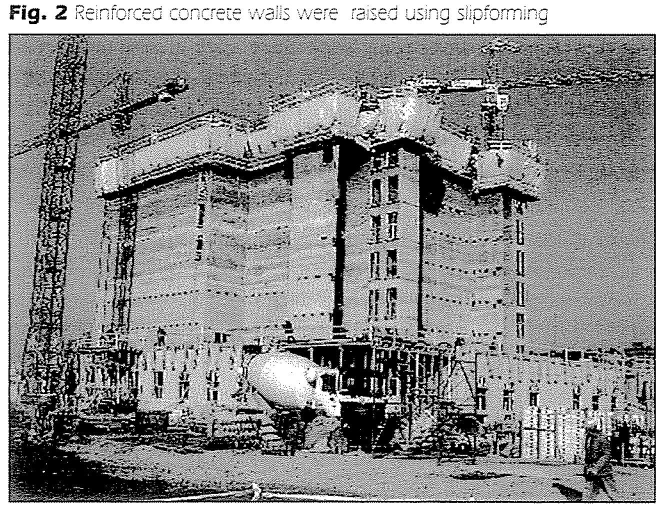
7.1 The stage block

The inside dimensions of the stage block are 18,0×24,0 m. Floor level of the stage is also the reference level of the entire building, the ± 0,00 level. The 12,0×12,0 m central area is occupied by the system of 72 trapdoors, each representing 50 kN vertical load. Fitting the heavy assembled working barrels into their final position posed an interesting organisational problem, as none of the floors along the possible access routes could carry the self-weight of the units. Temporary openings had to be maintained in the top floor of the stage block to enable precision placing of the working barrels by tower cranes. The 5 m long and 70 cm diameter working barrels had to be lowered through the openings 37,0 m above the receiving plates and manoevred to their final destination by using guide ropes.

The acoustical measurements and tests carried out following the completion of the combination slab-and-pile foundation indicated that the traffic on the railway bridge and that of the local HÉV regional railway generate a noise load that requires the placement of an acoustically dimensioned floating reinforced concrete slab with elastic bedding over the watertight foundation slab. This elastically bedded floor slab rests on lost formwork, supported by rubber-cork stubs, spaced to suit load-distribution.

Cast in situ reinforced concrete walls of the stage block were built by the slipforming method *(Fig. 2)*. Intention of applying the slipforming process at first worried the designers, as dimensional accuracy and retention of shape with structures built by slipforming generally lag behind the precision of structures raised by conventional shuttering. Designers of state of the art stage techniques endeavour to satisfy demands

Fig. 2 Reinforced concrete walls were raised using slipforming



of the directors as best as they can, consequently crowd the effective floor area of the stage with brail-lines, purchase lines, movable lighting bridges and other plant. After having consulted the designers of stage engineering, we agreed to the slipforming technology with the provison that deviation of the corner points of the stage block cannot exceed 25 mm from their design position at any point along the entire height.

Subcontractor of the slipforming applied special auxiliary structure to meet accuracy requirements. He built, for example, in the middle of the stage block a temporary reinforced concrete guiding shaft. Timber subsidiary trusses, rigged to this solid core, secured configurational trueness of the perimeter walls of the stage block.

Result surpassed expectations: deviations of the stage block walls remained inside 20 mm.

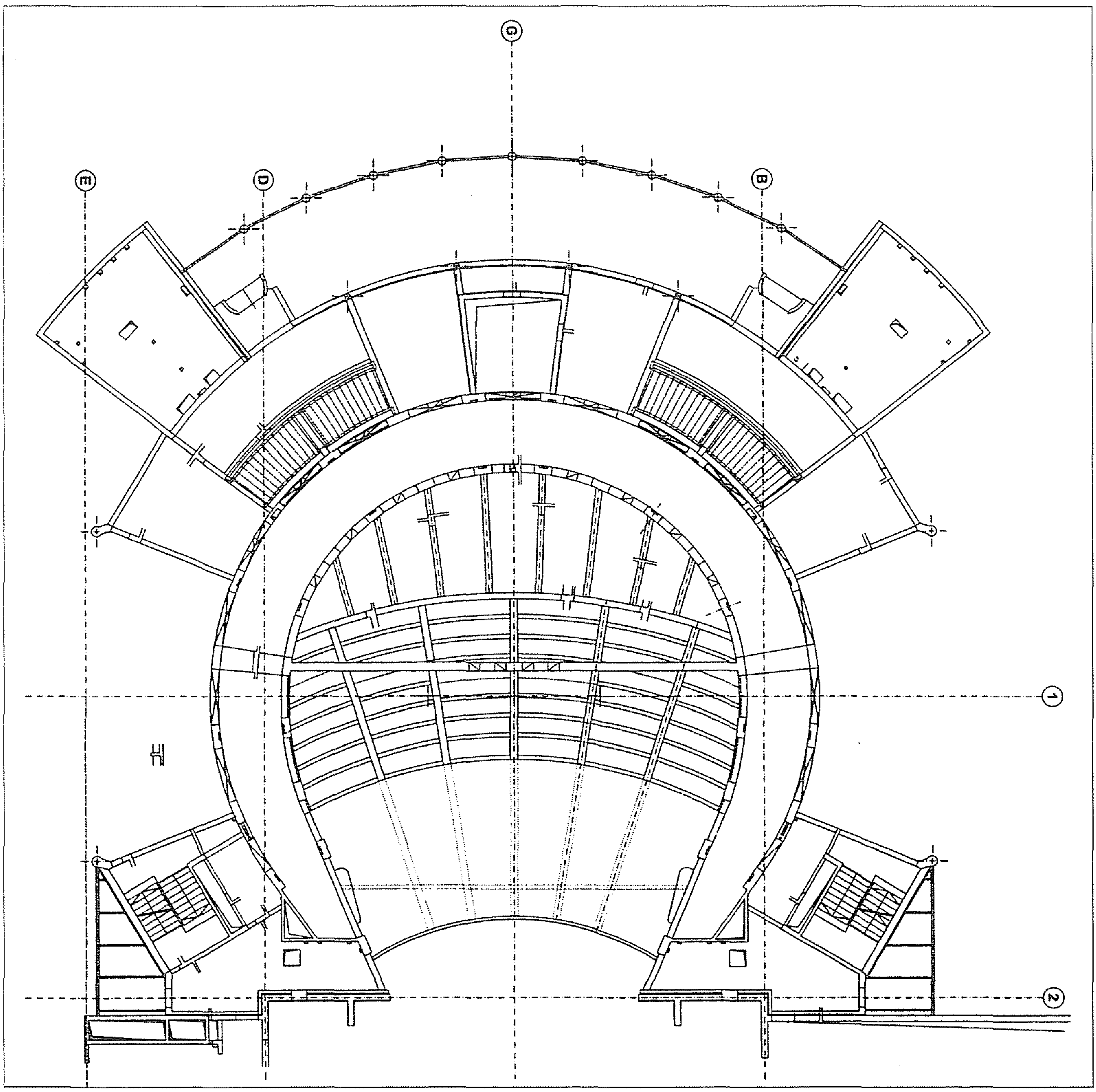
Provisional reinforced concrete structures were used once again with the large openings of the stage walls: with openings of the proscenium, the rear- and side stages. Temporary reinforced concrete columns held the wall-sections above the openings.

A row of storey height trusses positioned at the level of the rigging-loft carry the loads of the top floor, the rigging-loft, the bearing system of stage engineering and that of the suspended five-level service gallery. The heavy load trusses are framed into the cast in situ reinforced concrete - poured into slipforming - by imbedding doubly armoured fixtures and by placing them on supporting brackets, welded by seams with strength equal to that of the girders.

7.2 Floor areas around the stage block

Behind the main stage was built the 14.80×14.80 m plan, 8.70 m headroom rear stage. Its floor was built here again without temporary shoring, by way of lost profiled steel sheet formwork, supported by steel trusses. Over the rear stage, at the fourth and the fifth storey, the two-high costume-room is situated, its structure hung from the steel trusses of the fifth storey. The two-high cubic capacity paint room/maintenance shop occupies the sixth and seventh storey, decked by steel trusses. The trusses carry

Fig. 3 Reinforced concrete structures of the public use wing, second floor



partly reinforced concrete plate floors, partly rounded, assembled roof, combined with glazed skylight.

To the main- and rear stages joins the section on the eastern side that accommodates, among others, the spacious scene-dock, and preparatory rooms, and here can also be found the dressmaker's shop, with its structure held up by interior supports. Characteristic headroom is around 8.0 m; characteristic spans are between 13.0 and 15.0 m in this section. For this area we designed cast in situ reinforced concrete walls, mostly reinforced concrete floor slabs, placed on profiled steel sheet permanent shuttering, which in turn is supported by steel trusses. This scheme enabled raising the walls in advance, using slipforming, the floors were built subsequently. The steel trusses of the floor slabs were framed in by steel fixtures cast into the concrete walls; mortices were retained there during "sliding" for receiving the reinforcing roll bars of the slabs.

The western and eastern floor area of the service wing houses offices and changing rooms, facing the Danube and the Lágymányosi bridge. These areas are characterised by lower clearances, numerous multiform wall-apertures and non-sagging floors. These parts were built by traditional in situ cast reinforced concreting, using large-sheet wall- and floor formwork system. The inside floor sections connect to walls raised by the climbing formwork process. Shear teeth fit into the concavations, created when the walls were concreted.

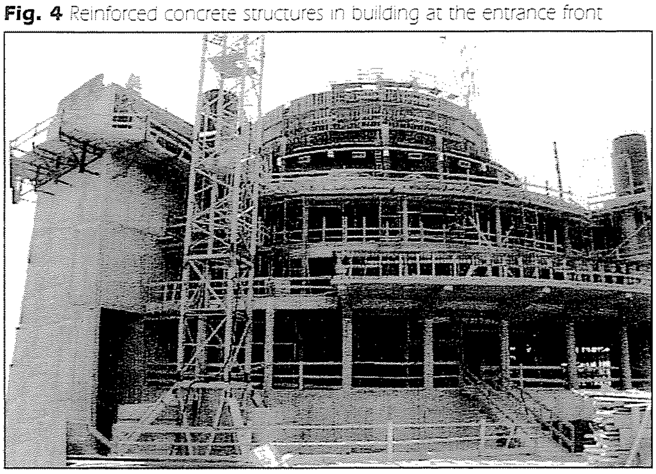
Thanks to the slipforming method, the reinforced concrete walls of the main- and rear stage, also of the eastern floor section were built up from the ±0.00 level upwards to their full height within a month, despite the very cold winter climate. First the related working drawings had to be adapted to the slipforming technique.

8. REINFORCED CONCRETE STRUCTURES OF THE PUBLIC USE WING

The public use wing is arranged around the circular borderline of the auditorium *(Fig. 3)*. A double colonnade surrounds the theatre house, calling forth the famous Coliseum motif. This structure lends adequate rigidity to the entire public use wing. Further rigidity is provided by the battlements – like pylons of the sanitary blocks on both sides of the main entrance *(Fig. 4)*.

The public use wing has rather varied spaces and structures both inside and outside the auditorium block. Many are the arched walls with manifold shape and size openings at the

Fig. 4 Reinforced concrete structures in building at the entrance front



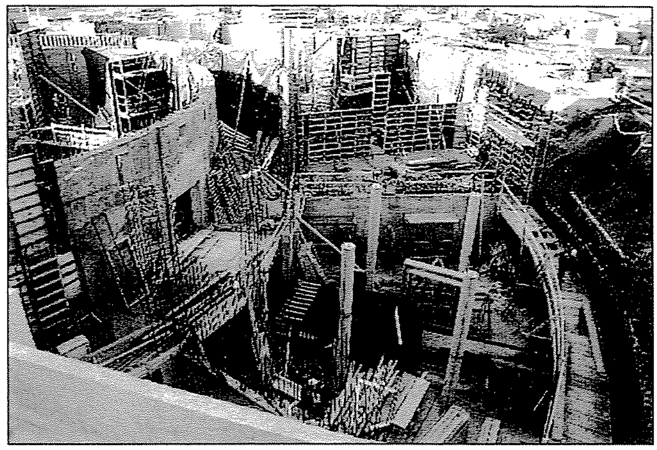


Fig. 5 Varied reinforced concrete structures of the public use wing

various levels *(Fig. 5)*. Builders of this part attempted the almost impossible: they tried to keep pace with the progress of building the service wing walls, in major part by slipforming.

This endeavour charged the structural engineers with more new duties. Configuring construction joints, investigating stability of the punctured walls, running ahead of the floor building by several levels, adjusting the joining reinforcement, supervising detail drawings prepared by the builders, called also for the strenuous cooperation of the contractors.

Main structural components under the auditorium are: the curved reinforced concrete wall encircling the theater space, the 22.0 m span, 3.5 m deep, "almost" deep beam, the sloping beams, arranged fan-fashion, resting on the thin-wall beam and the curved rib-system, adjusted to the stepped tiers.

The loadbearing structures of the theatre had to be constructed without interior supports, as underneath the theatre, at basement level, the studio theatre was established. 16.0–22.0 m span steel trusses support the studio floor. This floor is, at the same time, the floor below the entrance hall and the cloakroom. The 16.0 – 22.0 m depth of these trusses enabled the lighting and work galleries positioning between them.

Over the rear quarter of the auditorium a four-tier gallery was built *(Fig. 6)*. A row of cantilever beams, running parallel to the longitudinal axis of the house, carry the gallery. The rear support of these cantilever beams is the curved wall of the auditorium, the fore support is a 16.0 m span non-uniform girder laid over two points of the curved wall. Overhang of the gallery varies between 4 and 5 m.

Over the airspace of the auditorium three more floors were built. Their common primary beam-couple are the two storey-high reinforced concrete thin-wall beams, dividing the space over the auditorium into three separate compartments on the 6th floor.

The top floor over the 6th storey and that of the 5th storey are similar structures. To avoid the need for tall shoring we designed broad-rolled beams of constant depth, laid between the walls bordering the auditorium and the deep beams as well as between the two deep beams. The cast in situ reinforced concrete slab was placed on lost profiled steel sheet formwork, supported by HEA 400 and HEB 400, HEA 450, HEB 450 sections, respectively.

The floor above the 4th floor decking the auditorium is carried by the steel beams hung from the beams over the 5th storey. For acoustical reasons, material of the entire slab area here had to be reinforced concrete exclusively. To avoid on-site shuttering we specified reinforced concrete formwork panel.

Pivotal points of the three floor structures above the auditorium were the two around 20.50 m span reinforced concrete

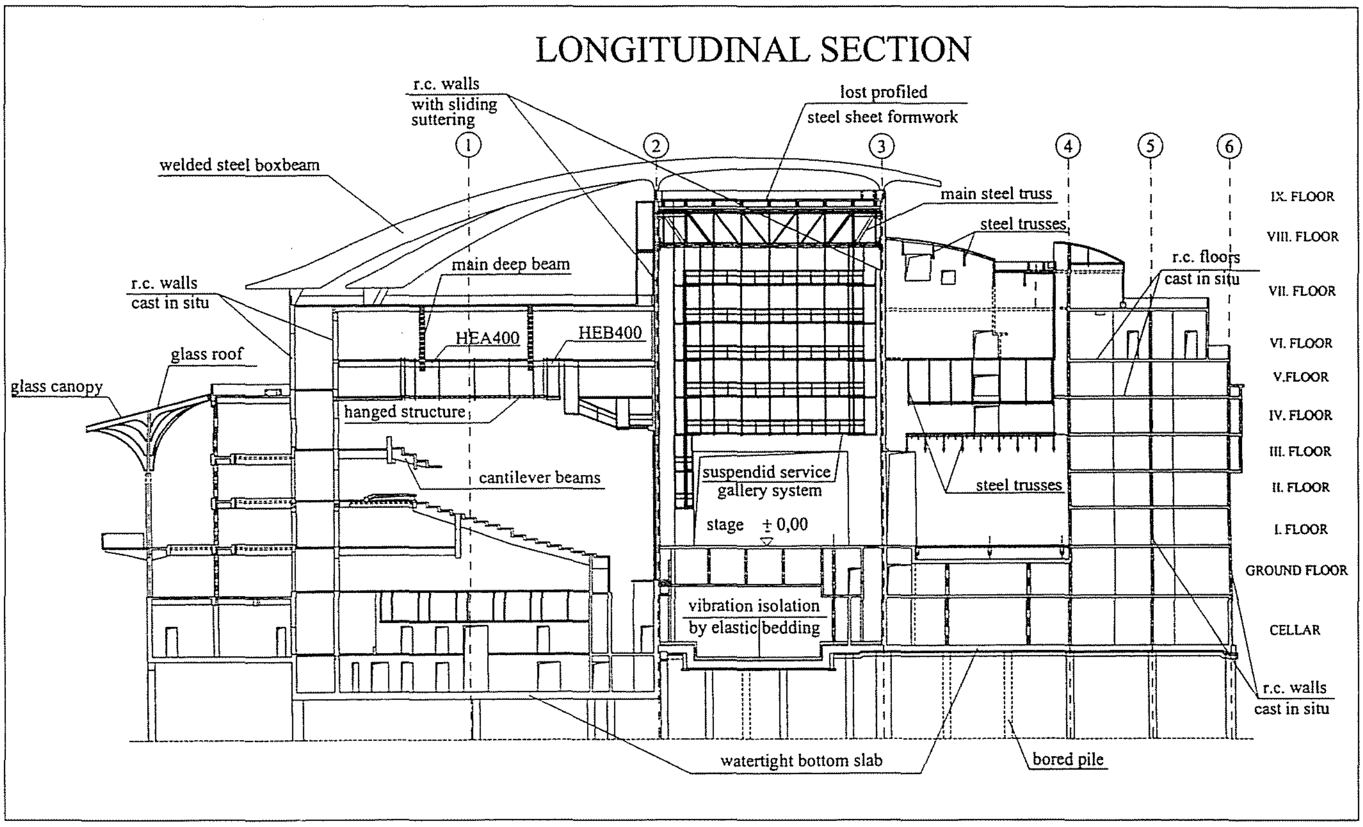


Fig. 6 Longitudinal section

deep beams. Constructing these conventionally by applying shuttering supported by very high shoring, would have arrested work under the deep-beams for a long time. Therefore, upon the iniciative of the general contractor, the designers worked out the method of constructing th deep-beams without scaffolding. First 4.5 m deep steel-lattice girders were hoisted to the position of each thin-wall beam. These trusses carried during construction work individually the weight of the connecting steel structures and the unset concrete of the deep-beams, they further were dimensioned to carry the load of the deep-beam formwork and that of the concreting stands. The trusses participate in the structural behaviour of the reinforced concrete deep-beams as stiff reinforcements.

Top flange of the imbedded steel-lattice girders was 2U200, the bottom one 2U220. Beyond this, 12Æ32 mm steel rods, spliced by sleeve pipes, had to be added for positive moment as auxiliary reinforcement. Total depth of the deep-beam was 5.34 m. BETONTERV Ltd worked out a specification statement for the specific concreting job. C35-16/F-AD self-compacting cocrete with VISCOCRETE additive was worked in, as specified.

Detailed process directive was worked out for harmonising the work of steel erectors, reinforced concrete workers and shuttering erector subcontractors cooperating in building the deep-beam.

I will refer to the steel structures linked to and supplementing the reinforced concrete structures only to outline a relatively comprehensive representation of the building.

An important part of the auditorium roof is the large oval glass dome over the centre. The designs of the dome were worked out in close, minute cooperation with the fellow-designers and specialist constructors. Lengths of the principal axes of the elliptic plan are 11.0 and 19.0 m. Generators of the dome are parabolic, its principal components are the 200 x 150 mm hollow section elliptic lower and upper rings. The hollow section was built up from steel plates, cut to appropriate shapes and welded together. The radially placed eared hollow section parabolic ribs were prepared similarly. The horizontal elliptic ribs are made of bent cellular sections.

Mounting the representative steel canopy over the main entrance came following completion of the service wing's structures. The glass canopy turns into a glass roof over the entrance hall space behind the frontal glass wall. 14.5 m high cylindrical columns support the glass canopy/roof along the crescent of the façade. Overhang of the scale-form units of the canopy is 3.5 – 5.0 m. Elliptically shaped girders with web-plate cross section, support the outward sloping plane of the glass canopy.

Copperplated upturned roof-wings border glazed fields of the canopy on both sides. The V-shape legs of the glazed strips rest along a polygon-approximated curve. The roof extends by an approximately 5.0 m overhang over the reinforced concrete blocks of the plumbing units.

Heights of the public use wing and the stage block are different. The arcuated steel ornamental roof links these into a single entity, as if crowning them. Its principal girders are steel plate welded section two-span Gerber box-beams. A line of three-chord trabeated trusses links the principal girders to each other.

9. STRUCTURES IN THE THEATRE'S SURROUNDS

A garden comprising a number of individual structures surround the theatre. We will mention three of these for their interesting structure: the statue-plinth on both sides of the main entrance, the 80 cm deep large pond and the Ziggurat recalling ancient cultures.

The plinths, consisting from 1 m deep stone blocks are centrally stressed-anchored to piles.

Bottom slab of the pond is a waterproof, altogether 25 cm thick, reinforced concrete slab, without expansion joint, laid on an undersoil, compacted and checked in several layers. The reinforced concrete parapet of the "ambulatory", pushing out into the pond, resembles a big ship. Over suitably shaped armoured concrete blocks, built in the pond, detail of the old National Theatre's main façade has been modelled from fashioned stone, and tippled in the water.

The central reinforced concrete roll shell of the Ziggurat and the radial walls were erected by applying slipforming, the stepped floors, the inside and outside ramps, the screwed mantle-wall were made by using conventional formwork.

10. OBSERVATIONS IN RETROSPECT

Implementing the new National Theatre represented a special undertaking for all participants. To satisfy demands of the work in progress, structural drawings had to be issued first, providing drawings in periodical batches, including at the same time structural requirements of all related special trades. Harmonising role of the practised Architect came once more to the fore. By promoting cooperation between the various designers, she was able to assure timely mutual data communication. The fact that the planning work was executed by familiar designers of various professions, well versed and experienced in theatre designing, definitely proved to be an advantage.

Pressing rate of the Work assumed "mechanising" the planning work. Overwhelming majority of the structural calculations, following load analysis, was made by AXIS VM 5.0 finite elements software-method. Structural plans were drawn up entirely by computer. We prepared formwork plans by applying AUTOCAD R14 and AUTOCAD2000 versions. For the reinforcement drawings we used VB_EXPRESS 2.5 version. Steel structure drawings were partly prepared by STEEL-EXPRESS, and in another part X-STEEL applications. Due to this, we were able to assist preparation of steel frame shop drawings by turning over our computer files.

During the performance of the Work, significance of the expert organisation was – if possible – even more marked. A mere 16 months were at our disposal from the issuance of building permit to the completion and handing over. Coordinating the work of numerous subcontractors, organising materials handling, strict adherence to uncompromising quality standards was the outstanding performance of the professionals of the general contractor.

As shown in the above, every collaborator contributed to the successful performance of the new National Theatre to completing it to the original term and, not in the last place, within the specified budget.

In January 2002, the National Theatre could occupy the building, commissioning could be begun. Finally, on 15. March 2002. curtain rose. On the ceremonial night Imre Madách's "Tragedy of Man" came before the foot-lights.

Public happily occupied the long anticipated new theatre. Tickets were sold out in advance for the whole season. To walk around the new building, to have a cup of coffee on the theatre's terrace and to stroll in the garden among the statues of famous Hungarian actors became a favourite Sunday pastime of the Budapest families.

Gábor Földvári (1944) structural engineer (1968) head designer of Földvári Mérnökiroda Ltd. Of his 33-year's professional career spent 25 years with large state architectural and planning enterprises (Általános Épülettervező Vállalat, Középülettervező Vállalat), where he accumulated extensive experience in designing steel as well as reinforced concrete structures. From 1994 he specialised in planning theatre reconstructions. Simultaneously he worked for 5 more years with HUNGARO-AUSTRO PLAN Ltd. as head structural engineer. In 1978 he graduated with distinction as a specialist designer of steel structures, attended EURO post-graduate engineering specialist training in 1996-97, obtained resident engineer's (clerk of works) certificate in 2002.

SLIPFORMED AND OTHER SHELL STRUCTURES


Dr László Varga – István Vígh

The authors describe three objects of reinforced concrete shell structure executed with various building technologies and which have recently been constructed by their company. At the reception building of the Millennial Park we review the execution of two reinforced concrete adjoining conical shells reversed, which resulted a very attractive external and internal appearance. Secondly, large cooling towers built using slipforming technology and having hyperboloid rotation shell. The exceptionally thin wall thickness compared to the diameter and height may be of considerable interest. Finally, water storage tanks constructed with a prestressed shell structure demanding complete water tightness. The shuttering in this case was corrosion-resistant steel plate needing the application of special technology.

Keywords: slipforming, shell structure, hyperboloid of rotation, prestressing, variable thickness

1. INTRODUCTION

The General Building and Formwork Ltd. No. 31 was founded in 1992. During the privatization of the General Building-Trade, Company No. 31 was acknowledged in the national and international building trade markets alike. The limited company inherited renews continuously technological and technical knowledge, which has secured success for it in the realization of public buildings, industrial and agricultural projects as well as special projects.

Major trends in project activities of the company in Hungary are: public buildings, industrial, agricultural and environmental projects, waste water purification plants, waterworks, silos, reconstruction of silos, execution of vertical, high structures having variable cross section and wall thickness.

In addition to the Hungarian projects, our company has been engaged in export activities for several decades through our Swedish / Hungarian joint venture Bygging Ungern 31. AB which is of 30 years standing. We have built and are still building industrial structures for instance in China, Japan, Singapore, Iran, Egypt and in 24 other countries of the world.

The qualified experts of the company have always been predisposed and interested in the execution of special constructions.

We should like to review three of the structural objects lately executed in Hungary and abroad:

1. Millennial Park, Central Reception Building
2. Large cooling towers having a shell structure with variable diameter and wall thickness
3. Technological water storage tanks of prestressed concrete shell structure for paper factories.

2. BUILT STRUCTURES

2.1 Millennial Park, Central Reception Building

General designer: CÉH Co.
1114 Budapest, Bartók Béla út 57.
Architect: Weber Architect Office Ltd.
József Wéber, Ybl prize-winner graduated architect
General contractor: 31. General Building and Formwork Ltd.
1052 Budapest, Petőfi Sándor utca 7.
Contractor's agent: Csaba Pataki, graduated civil engineer
Period of construction: 2000 – 2001

Principal dimensions of the building:

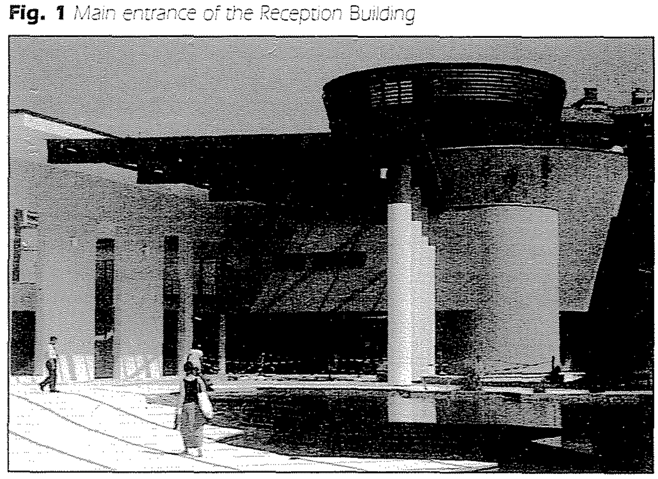
– Height:	16.5 m
– Dimension of the outer conical shell:	
Base diameter:	17.0 m
Top diameter:	21.0 m
Height:	8.8 m
Wall thickness:	0.25 m
– Area of the building:	1 100 m^2
– Number of levels:	3

The building was built for the coronation millennial of St. Stephen, the first king of Hungary.

The Reception Building (*Figs. 1, 2 and 3*) serves as an area for various shows and has an attractive external and internal appearance. The structural solution was one of the great architectural productions of the year and was acknowledged with the Master Prize of the Building Trade Industry in 2001.

The center of the building is a reinforced concrete conical shell rising from the ground floor slab. The conical shell has a wall thickness of 25 cm with a circular symmetrical flexural shell structure inclining outward at 75° and is supported by 4 columns of variable cross section.

Fig. 1 Main entrance of the Reception Building



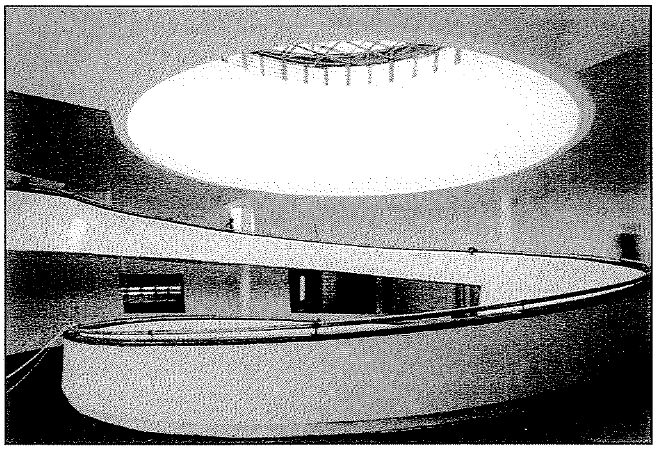


Fig. 2 Hall of the Reception Building

The smallest cross section of the columns above the foundation slab is 50 x 40 cm. The increase corresponds to the 75° angle of slope and the largest cross section is at the ground floor slab. From this level the conical shell leans towards the outer circumference onto the inner side pillar of 40 cm diameter.

Starting at the level of the slab above the ground floor, a spiral ramp for pedestrians is constructed inside the inner part of the conical shell with its structure supported by the conical shell, the inner pillars and balustrade ring.

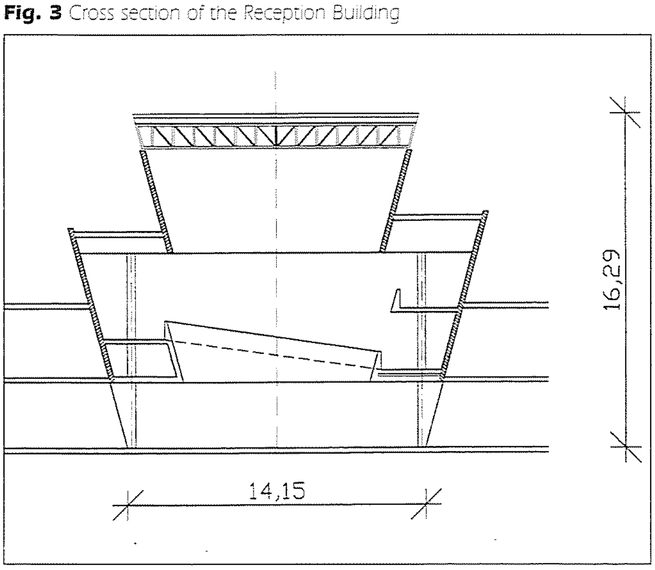
The closure of the space between the two conical shells and the fixing of the inner one are assured by a spiral reinforced concrete plate ramp.

The slab closing the inner center is made using glass fibre concrete formwork. Its structure is a steel, bi-directional, flat lattice girder, loaded on the upper ledge of the horizontal reinforced concrete conical shell. The grade of the applied materials was: C20-30 concrete and B60.50 reinforcing bar.

Due to the winter weather, the construction of the formwork had to respect the tight work schedule as follows: the formwork elements could be pre-assembled; the inner surfaces did not require any plaster work on the various radius and heights; the same formwork units might be re-used with a little conversion and assembly (*Fig. 4*).

Following the investigation of many systems the DOKA TOP 50 system was applied. As it can be seen from *Fig. 1*, the vision of the designers came to fruition and a high quality building was constructed in a popular park in memory of the millennial coronation of St. Stephen, the first king of Hungary.

Fig. 3 Cross section of the Reception Building



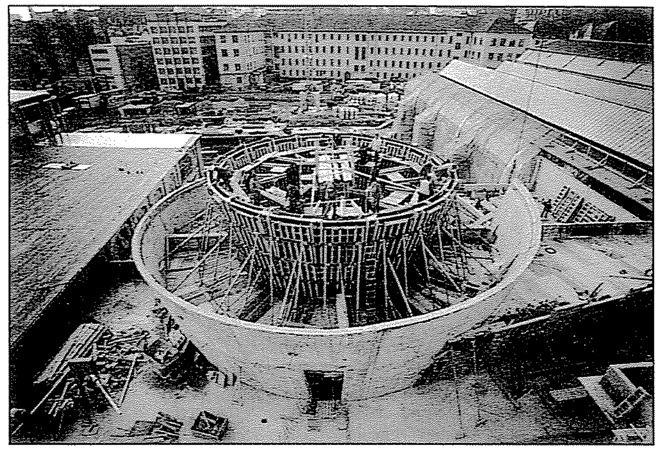


Fig. 4 Formwork of the shell structure

2.2 Large cooling towers with a shell structure of central - symmetrical, variable diameter and wall thickness

The large reinforced concrete cooling towers (*Fig. 5*) were built using slipforming. Having a hyperboloid of rotation shell they may be considered as special reinforced concrete objects. Earlier examples were prepared of steel structure and those built of reinforced concrete were considerably smaller and made using climbing formwork.

For the slipforming developed and produced in Hungary the drive hydraulics and central control device was ensured by the Swedish BYGGING-UDDEMANN AB. With this equipment more than 14 large cooling towers have been built up to now with Hungarian participation, seven of them in Iran.

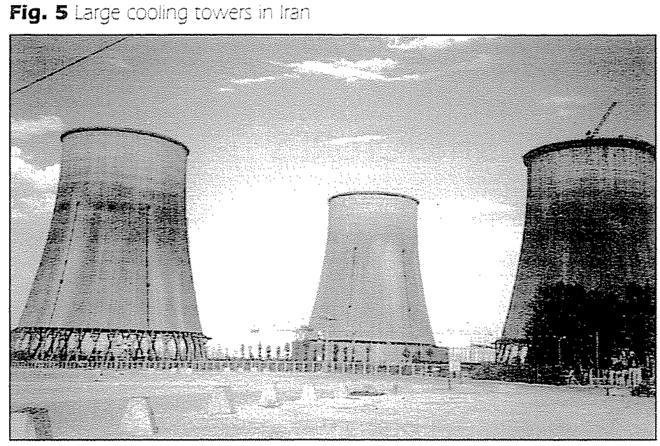
Principal dimension of the shell of the cooling towers:

– Base diameter:	108 m,	wall thickness	120 cm
– Top diameter:	72 m,	wall thickness	24 cm
– Minimum diameter at 86 m:	64 m,	wall thickness	17 cm
– Height:	118 m		

Designers: Both the equipment and the cooling towers were designed by
MÉLYÉPTERV Consulting Engineering Ltd.
1052 Budapest, Bécsi u. 1.

Architect: József Thoma, graduated civil engineer, inventor.
Structural engineer: László Mérei, graduated civil engineer.
Structural designer: István Szatmáry, graduated civil engineer.

Fig. 5 Large cooling towers in Iran



Customer: BYGGING – UDDEMANN AB
SE 11642 Stockholm Sweden
Katarina Bangata 79.
General contractor: 31. General Building and Formwork Ltd.
1052 Budapest, Petőfi Sándor utca 7.

From the centrally controlled unit the equipment, which may be assembled to an arbitrary diameter, is capable of executing objects either with a positive or a negative angle of inclination within a variation of 2 cm/m.

The 3 main movements of the equipment are coordinated centrally. These are respectively the vertical and horizontal movements and the increase or decrease of the wall thickness.

The wall thickness of the cooling tower narrows from 120 cm to 17 cm at the minimum diameter at 86 m, then it diverges continuously to 24 cm, until the achievement of the total height.

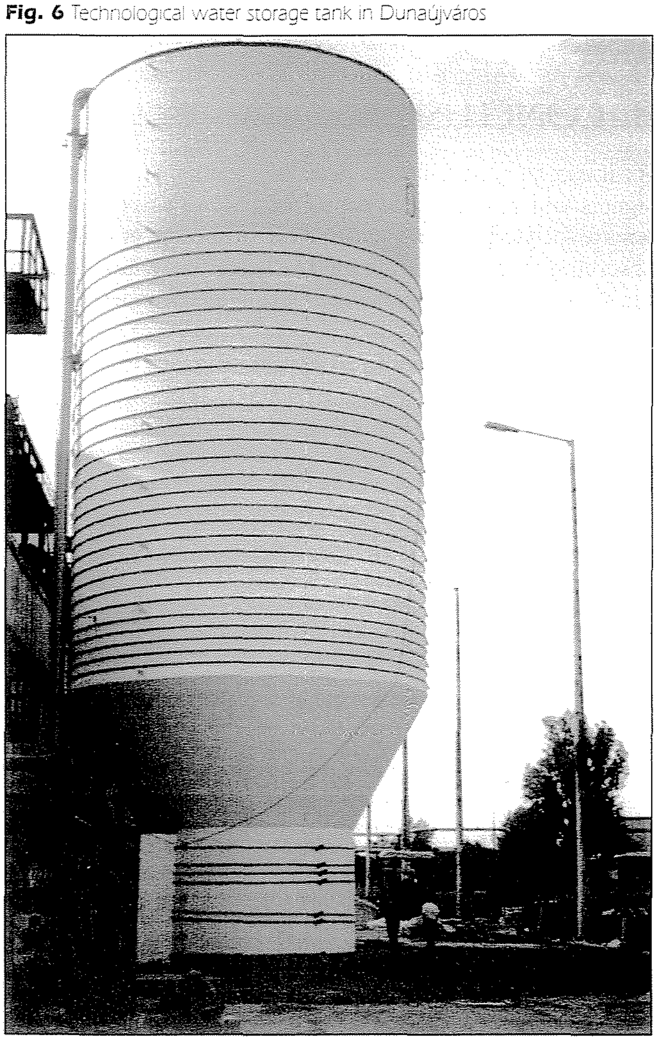
It is characteristic that the thickness of the structure is proportionally less then that of an egg-shell.

2.3 Technological water storage tanks of prestressed shell structure for paper factories

The company executes 5-6 environmental projects every year. Such an object would be the so-called vertical steep box built for a paper factory.

The thin-skinned, cylindrical, 22 m high, 1000 m^3 capacity tank in Dunaújváros (*Fig. 6*) was made of waterproof concrete with unbonded post-tensioning to correspond to water tightness and other technological requirements.

Fig. 6 Technological water storage tank in Dunaújváros



The principal dimensions are:

Lower mixing cylinder:	
– diameter inside:	4.6 m
– height:	3.3 m
– wall thickness:	35 cm
Funnel:	
– base diameter:	4.6 m
– top diameter:	8.6 m
– height:	3.5 m
– wall thickness:	30 cm
Upper steeping part:	
– diameter inside:	8.6 m
– height:	15.5 m
– wall thickness:	26 cm
Total height:	22.3 m

The lower mixing cylinder and the funnel were constructed using normal shuttering, while the upper steeping part was made of waterproof concrete (C30-16/KK-Vz4) using slipforming technology.

It was a basic requirement that the water-retaining test had to be held after tensioning without special coating of the surfaces.

Following the successful water-retaining test the external wall surfaces are painted. This grand steep-box has operated without failure over the last three years.

Following the success in Dunaújváros the paper mill in Csepel ordered a steep box of 800 m^3. But in this case the lower cylindrical part together with the bottom and the funnel had to be coated on the inner side with 5 mm thick KORACÉL plate (corrosion-resistant steel plate) to correspond to the abrasive resistance requirements.

The principal dimensions of the steeping box are:

Lower mixing cylinder:	
– diameter inside:	4.5 m
– height:	3.0 m
– wall thickness:	30 cm
Funnel:	
– basic inner diameter:	4.5 m
– top inner diameter:	9.0 m
– height:	3.5 m
– wall thickness:	30 cm
Upper steeping part:	
– diameter at the bottom and at the top:	9.0 m
– height:	10.5 m
– wall thickness:	30 cm
Total height:	17.0 m

Designer: MÉLYÉPTERV Consulting Engineering Ltd.
1052 Budapest, Bécsi utca 1
Architect: József Thoma, graduated civil engineer, inventor
Structural engineer: László Mérei, graduated civil engineer
Structural designer: István Szatmáry, graduated civil engineer
Gábor Felföldi, engineer
General contractor: 31. General Building and Formwork Ltd.
1052 Budapest, Petőfi Sándor.utca7.
Materials used:
- corrosion-resistant steel plate (KORACÉL) KO 35 Ti, thickness 5 mm, grade of the weld is of second-class, watertight.
- tensioning cables: strand ST 1670/1860 coated with aT15 S plastic A_p= 150 mm^2

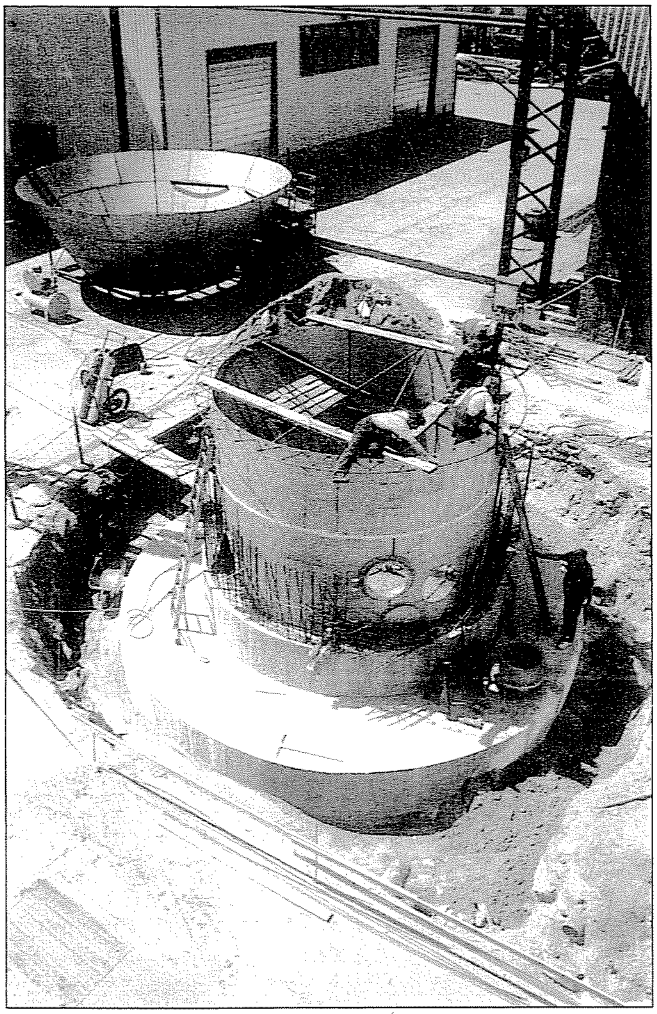


Fig. 7 Technological steep box with KORACÉL internal shuttering in process of construction at Csepel (1)

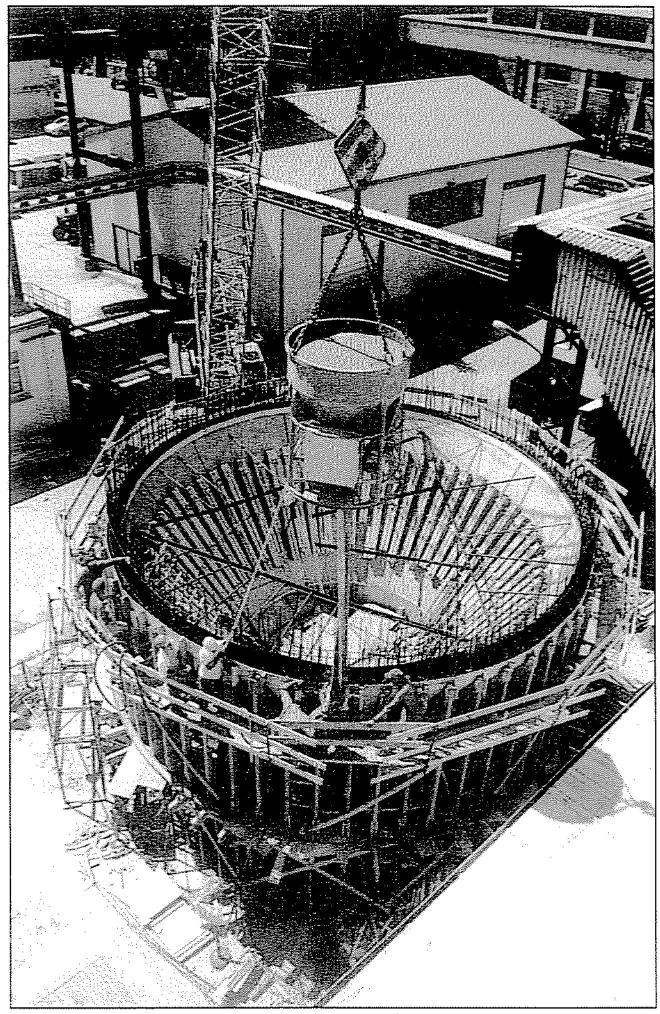


Fig. 8 Technological steep box with KORACÉL internal shuttering in process of construction at Csepel (2)

During the detailed elaboration of the technical solution, the experts of the Contractor decided to use 5 mm thick KORACÉL lining as an internal shuttering.

In the meantime emerged that the placement of the KORACÉL part of funnel as a shutter and the stable anchoring of the funnel caused the biggest problem, because the plate could not be bored through for anchoring.

The following is the technological sequence executed:

- The KORACÉL (corrosion-resistant steel plate) base plate and the inner ring were bonded on the spot,
- mounting of reinforcing steel,
- mounting and fixing of external shuttering,
- concrete placing.

a) funnel

- the internal KORACÉL(stainless steel) plate was mounted and bonded on the spot near to the object,
- shuttering of the outside surface,
- mounting of reinforcing steel,
- on the mounted reinforcement steel radial guide rails were adjusted with a height corresponding to the concrete cover,
- the assembled KORACÉL shell of the funnel was placed by crane,
- it was centered and fixed against displacement,
- it was concreted in from the outer side with special conveyors.

The technological procedure is shown in *Figs. 7 and 8*.

Following this the slipforming equipment was mounted for sliding of the upper part of the steep box.

We cannot report on the final outcome because at the time of writing of the article the building of the object is in progress.

3. CONCLUSIONS

As it can be seen from the described works, Hungarian designers and building contractors are able to solve the problems associated with the construction of special, puzzling and absorbing objects requiring high technical know-how including implementation at high level. Our engineers undertake with pleasure the execution of such objects because the accomplishments of such high quality structures increases their professional pride and ambition.

Dr. László Varga (1956) graduated from the Faculty of Architecture, Technical University Budapest (1982) and received doctor's degree at the University of Economics. He was active at the Public Building Design Company 1982-84 and 1984-92 at 31. General Building Trade Company Export Department and currently Managing Director 31. General Building and Formwork Ltd. Former Secretary then President of Hungarian FIP Group, then Honorary President of Hungarian *fib* Group. Major works: Ministry of Foreign Affairs, special slipformed towers, Dunaújváros, malt factory (incorporation DYWIDAG). Abroad: chimneys, water towers, TV towers, bridge supports, gas tanks, corn and cement silos, cooling towers in Australia, Singapore, Turkey, Iran.

István Vígh (1933) graduated from the Kharkov University of Construction, Faculty of Structural Engineering (1958). 1958-63 resident engineer at 31. General Building Trade Company.1963-69 Comecon, Committee of Building Regulations from 1970 31. General Building Trade Company, head of Export Department from 1992 31. General Building and Formwork Ltd. Leading expert. Major projects: dwelling houses in Budapest, town building and Hospital in Kazincbarcika. Abroad: Direction of 124 objects in 27 countries.

THE MOST UP-TO-DATE HYDRAULIC ENGINEERING PROJECTS IN HUNGARY

Dr. László Tóth

In this article, the author gives a brief assessment of the Hungarian hydraulic engineering project construction practice of recent years, summarises the development tendencies and results by illustrating some significant projects in the fields of wastewater and sludge treatment. The article also presents some details, which are modern or significant in engineering respects.

Keywords: monolithic structures, water tightness, slipforming, construction joints, dilatation, trench, drainage, heat effects

1. INTRODUCTION

In the specialisation of civil engineering associated with the development of wastewater drainage and treatment, there are a significant number of hydraulic engineering projects with different functions with the aim of processing useful volumes. As far as the generally combined shell or box structures are concerned, water tightness of material, durability and inexpensive building are possibly the significant requirements. In most cases projects have to meet these important dominant demands, while environmental impacts and adaptation are disadvantageous.

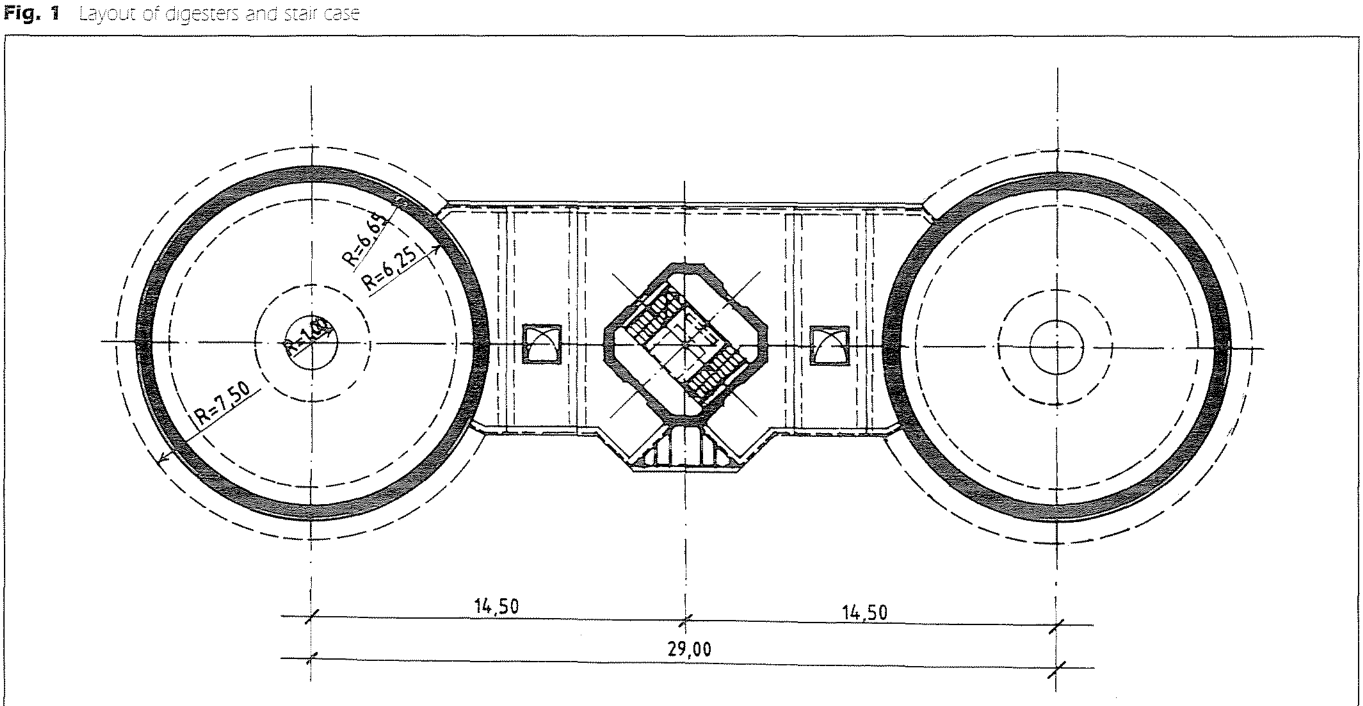
2. RENAISSANCE OF MONOLITHIC REINFORCED CONCRETE STRUCTURES

20-25 years ago in Hungary there was a strong effort within civil engineering practice to construct partially prefabricated structures even for projects with the requirement of water tightness. The elements prefabricated in industrial circumstances, which were generally executed with smaller wall thickness compared to monolithic construction, had been qualified actually watertight considering the material, due to favourable concrete pouring and compacting possibilities. According to e.g. post-tensioning or epoxidizing, the joints of prefabricated elements regularly meet the demands of water tightness in addition that of the load cycles. A number of basin-like constructions with the parameters mentioned have been built either in the field of drinking and industrial water tanks or wastewater treatment buildings.

Low standard of formwork systems of concreting technology explains the considerable frequentation of prefabrication. In recent decades a significant development has been going on in the fields mentioned, and in the years up to the turn of the century the situation has been radically changed. Naturally, political and social transformations, competitive markets and last but not least deliberate development played role in this process.

According to the development of formwork and concreting technology systems in super-structural engineering, a

Fig. 1 Layout of digesters and stair case



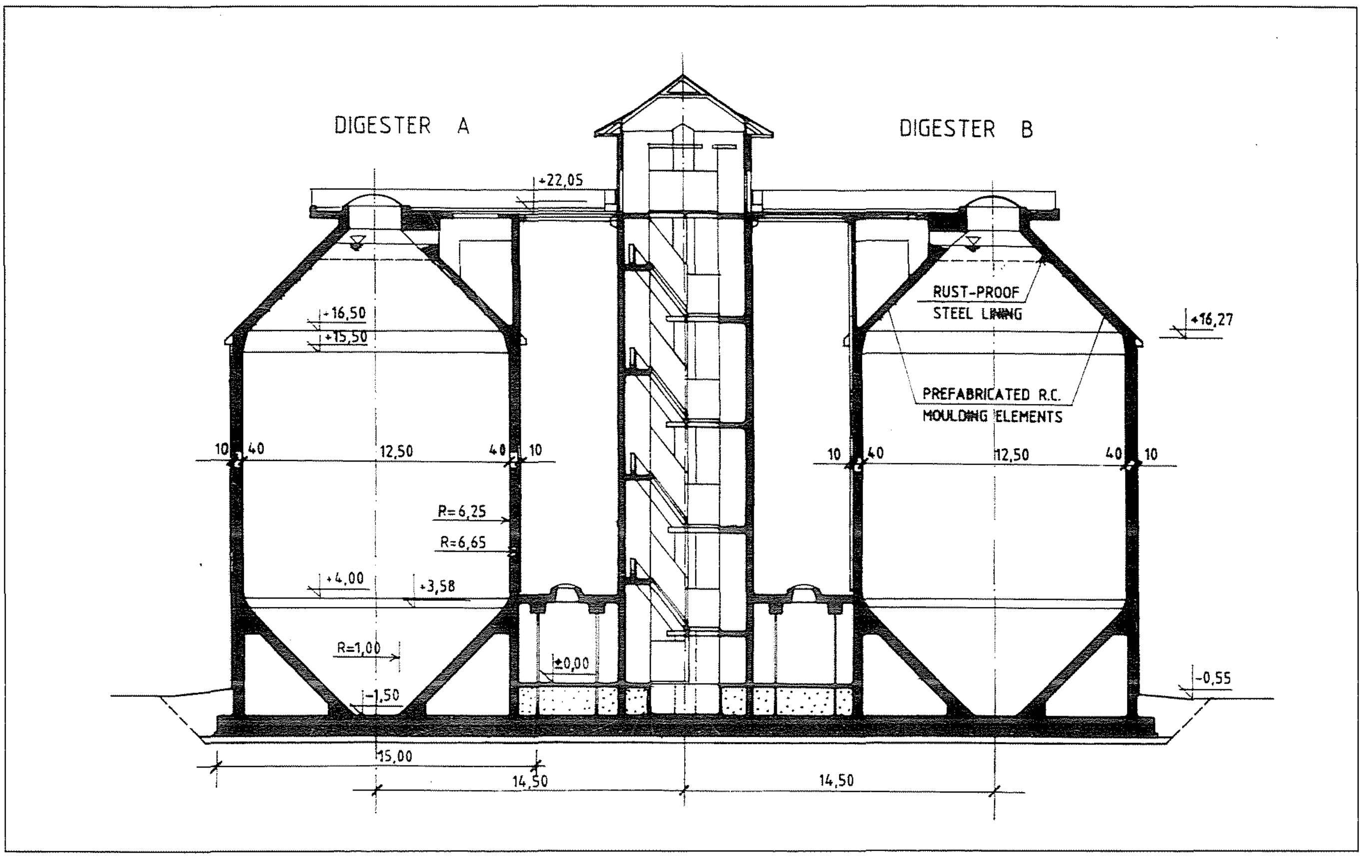


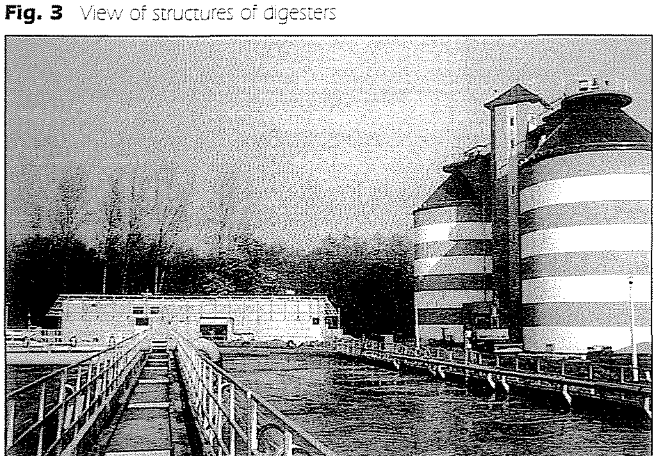
Fig. 2 Vertical section of digesters

change can be recently observed in the direction of monolithic construction as well. Regarding civil engineering structures the same holds, generally speaking.

Structural designers of hydraulic projects usually have to be aware of geometrical parameters which have to meet the demand of the actual process, the volume and land level factors, namely the "narrow margin" considering structure-forming aspects, especially in case of wastewater treatment projects. Frequently, the optimisation of a structure is not achievable from the point of view of load cycles coming in conflict with important key functional purposes. Defined geometrical data depend on the volume of the structure and in some cases formwork construction has to be paid special attention to. On the other hand a greater volume or the geometrical parameters require serious consideration in relation to the force system.

In spite of the foregoing, it is evident that monolithic construction is significantly dominant within the hydraulic engineering project field. Due to up-to-date formwork systems, concrete surface finish of a well-designed and executed monolithic building usually meets the water technology requirements

Fig. 3 View of structures of digesters



to a great extent. In spite of aggressive chemicals, well-constructed projects meet durability requirements adequately due to concrete quality properly chosen or perhaps cover layer application. However this is an important demand considering every communal establishment.

Up-to-date shuttering and concreting technology, and, last but not least, efficient movement joints can cover the complex requirements of projects as defined by the process engineer and specialist designer. It is unrealisable to execute a structure of watertight material without formwork properly designed and without good pouring and compacting of fresh concrete from adequate composition.

3. WASTEWATER TREATMENT STRUCTURES

After overall assessment of developing trends and the proof of the above statements, let us mention some important projects realised in recent years.

3.1 Sludge digesters in the town Nyíregyháza

For treatment and disposal of sludge originating from wastewater treatment processes, a great number of solutions are known by wastewater treatment process engineers. In recent years in Hungary reinforced concrete sludge digesters have come into the limelight. This is because biogas originating from the process can be used as a secondary energy source and, more importantly, sludge quantity can be decreased by half due to digesting. There was a technical article published by G. Péter and L. Tóth in the journal "Concrete Structures"

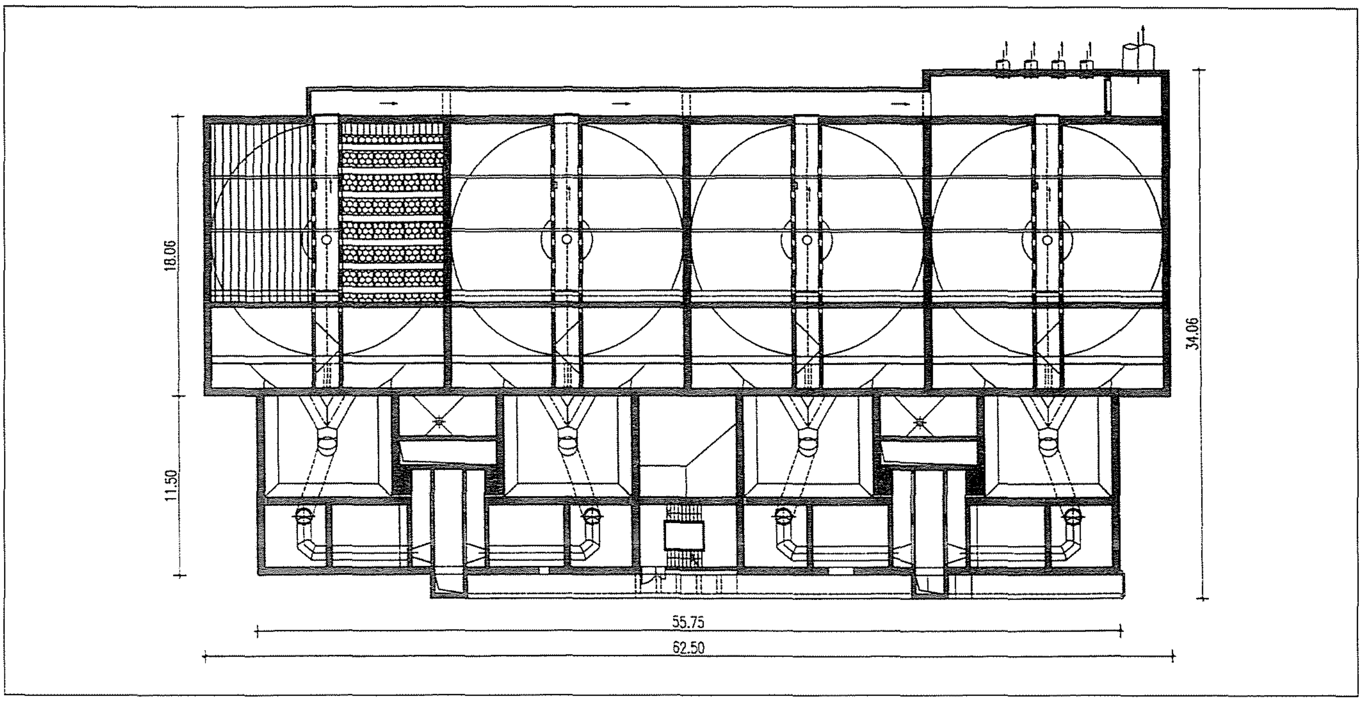


Fig. 4 Layout of pre-settling structure complex

in 1999 on the topic of two post-tensioned concrete sludge digesters each of 4500 m³ useful volume capacity. These digesters remain the structures of the greatest volume capacity in Hungary.

As a result of the development of wastewater treatment in the town Nyíregyháza, two sludge digester structures have been built, each of 2000 m³ useful volume capacity (see *Figs 1 to 3*). There is a closed staircase between the two structures for technical reasons, the substructure of which is functionally and structurally connected to the two structures located 29m apart on the interaxis distance (see *Fig. 1*). The structural solution and topologic installation can be seen from the common cross section of the structures (see *Fig. 2*.). It can also be observed that the two structures of complex shell construction carry very heavy load due to the 20 m high water pressure, which were balanced after settlement of several centimetres, according to the flat foundation. The intermediate staircase and its sub-structure, being almost unloaded, give perceivably less load to the subsoil. That is to say the designer had to take into consideration vertical movement as well. The possibility for these movements is guaranteed by 2-2 collinear dilatation gaps. To take into account the smallest conceivable movements, the units of the substructure provided with a dilatation gap were concreted after the digesters were filled up with water for the first time.

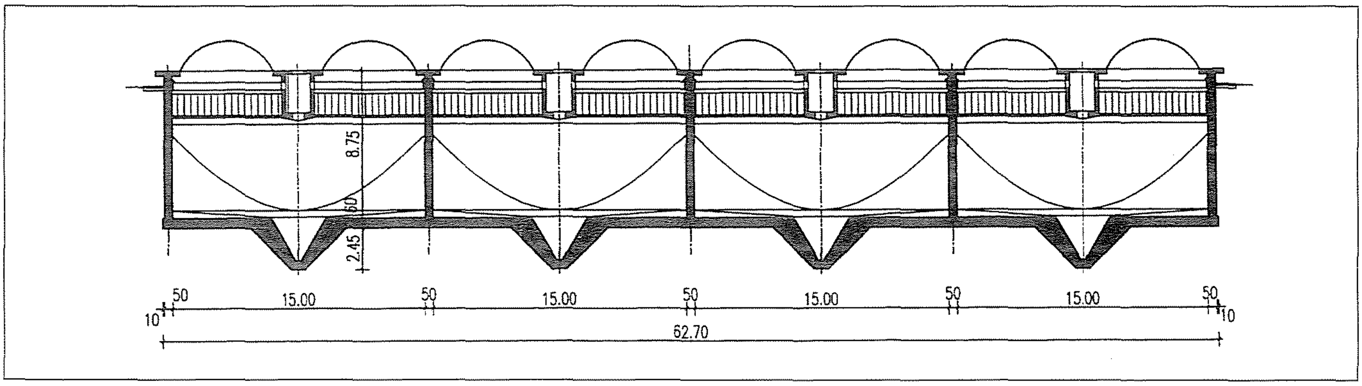
Each of the digesters are of 2000 m³ useful volume, made of cylindrical walls and cone-shaped at the upper and lower parts in order to collect and eliminate digested sludge, and also to derive the excellent biogas. The most important geometrical parameters can be seen in the figures referred.

The digester, as a whole, is a compound shell structure made of normal reinforced concrete. The structure meets the material water tightness requirements, in such a way as not to exceed the limit value of 0.1 mm when considering the cracks derived from hoop stress; namely the requirement of limited crack size provable for the hoop stresses of a 40 cm thick cylindrical shell.

The winning contractor elected to apply slipforming technology. From this fact it emerged that the bottom conical shell should be built afterwards. Annular stresses emerging in the cylindrical shell can be considerably diminished by the application of a stiff structural joint of the conical shells. to the cylindrical ones. Consequently it has to be guaranteed that the fittings are concreted into the actual section of the cylindrical shell built at the first stage of slipforming, and to which the reinforcement assuring the structural joint can be connected.

The internally-threaded fittings between the vertical reinforcing bars in three different levels with horizontal axis, and that of 45° inclination can also be seen. After releasing the fittings capped before, the extension reinforcing steel, which constitutes the reinforcement of the upper ring of the conical shell to be constructed, can be screwed in and stressed into the inner thread fittings by adjustable nut wrench. Added to the aspects of slipforming technology, the cannelure also func-

Fig. 5 Common longitudinal section of the four pre-settling tanks



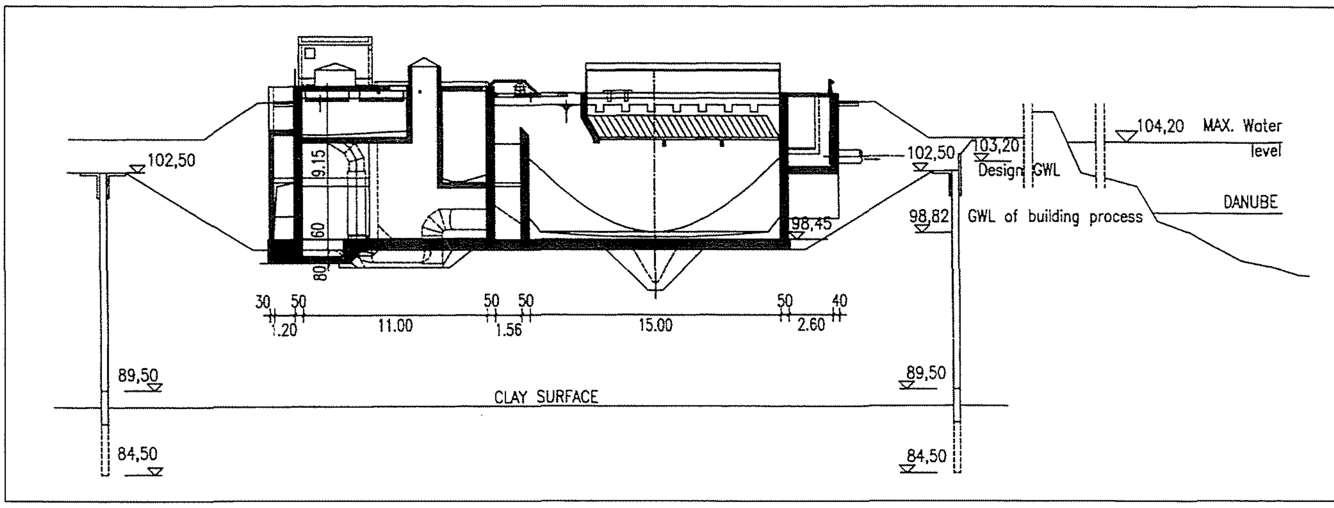


Fig. 6 Cross section of structure complex for wastewater treatment and the Danube's profile

tioned to transfer the vertical forces. In addition to this, a borehole seal or elastic seal was placed into the hole in order to prevent accidental leakage due to the rotation expected along the upper edge. An important demand was to guarantee the location of the inner thread fittings by strict dimensional accuracy. Through experience a successful solution was found by the building engineer. Such a joint is not often applied but guarantees the important requirements with regard to the force system beside the application of the very effective slipforming technology.

The solution of the upper conical structure of the digester was similar to the digesters of Debrecen referred to. Reproduction of the earlier approved methods proved the correctness of the earlier conclusions. The same can be declared with respect to the water tightness of the cylindrical walls. Cycle concreting and adequate management have been successfully executed in order to avoid accidental stop-end joints and the structure has been conclusively qualified as watertight considering the material.

3.2 Pre-settling structure at the North-Budapest Wastewater Treatment Plant

Significant progress has been accomplished in recent months in Budapest in order to enlarge the capacity and upgrade the process system of the North-Budapest Wastewater Treatment Plant built in the 1970s. Within the scope this a new pre-treatment structure complex of considerable size had to be built to effect the so called "mechanical" treatment of the wastewater consisting of 200.000 m^3/day capacity. The structure created was according to the conception of the French Consulting Co. OTV (designer: MÉLYÉPTERV KOMPLEX Co.). It consists of four heavy loaded pre-settling basins and water distributing, delivering and other technological structural elements (cells, shafts and channels). The vertical location of the structure complex of 62.50 m x 31.06 m overall dimensions had to be solved while taking into consideration that the whole structure had to be placed for hydraulic reasons under the surface level of the compacted land. The characteristic depth of the structure is 9 m. In addition to functionality, the structural system is also demonstrated on the ground plan (vertical and horizontal sections) in *Figs. 4 to 6.*

This considerable structure has been built next to the Danube. The vertical location of the structure shown in *Fig. 6* accords to the critical water levels of the Danube. In explaining this structure there are three professional problems which are detailed in the following.

3.2.1 Construction prospects of the structure

This kind of structure, due to its massive size and depth is built over many months and requires the most cautious building technology decisions. As the bottom level of the trench needed for construction of the structure is located 4.0–4.5 m below the average water level of the Danube a drainage system becomes necessary during construction. The aim of the engineering decision was to find the most economic solution, so the whole trench has been rounded by a watertight clay cut-off wall, which needed to be clamped to the required extent into the water tight, deep-seated basic solid bed several meters thick. Naturally, in anticipation of seepage, pumps were applied to lift water from the collecting wells. The more advanced construction gained improving conditions with regard to keeping the trench dry. By a lucky chance the Danube water level was not so high even in the most critical months to put at risk the method of building.

3.2.2 Stability of the structure

According to the relevant standards the designer has to guarantee the structural integrity against uplift in the case of an empty condition. Considering general civil engineering practice – balancing the structure by material in-built – the actual structural solution reflected a very low economic effectiveness, because of the built in excess of reinforced concrete and the additional drainage demand. The following compromise decision emerged. The clay cut-off wall mentioned above which had originally been built for temporary purposes, was held permanently for monitoring and dewatering wells. In case of each incidental requirement of the discharge of the structure, the measure of the discharge has to be considered on the basis of the actual groundwater level. The designer therefore has to determine the critical water levels in the cases where the cells 1 or 2 or 3 can be emptied. As far as the mechanical engineering aspect is considered, the possibility remains of periodical pumping of water above the critical level inside the cut-off wall.

The compromise decision detailed above made the one-shot investment costs more advantageous and did not cause significant addition expenses for the operator. There is no doubt it is necessary to take into consideration safe stability in non-working situations according to the conditions declared by designer.

3.2.3 Problems of load cycles

Structural designing requirements touched on in paragraph 2 are reflected in the geometrical system of the structure. The demand of symmetric water distribution, the fact that large diameter pipes and fittings together with assembly or repair of same and other technologic aspects results in the single-axis symmetric interior walls of the block-like structure. However for the location of those walls considerable bending load can occur as well as partial loading.

In respect of the structural aspect a dilatation across the structure should be reasonable to design but the suitable location of that could not be found when considering either technologic or economic concepts. By the way each dilatation can be a source of error especially in the case of such a deep structure, which in turn could cause other risks of incidental failure.

The absence of dilatation naturally resulted in a more careful analysis by structural designer of the side effect of additional loadings. This problem emerged in the most critical way at the ending floor of the structure complex without superstructure, which is located above ground level. Decisions have been made for the possible methods of thermo and water insulation at the ending floor, but both of them have been abandoned for contractor reasons. However, this imposed significant additional loads from the difference of the temperature Δt which can be calculated from the differences of summer temperature and winter cooling in the ending floor and from the differences of wastewater and air temperature. In addition to that, a great number of big holes needed to be created through the floor for technologic reasons, so tensile strain derived from heat expansion was adversely concentrated in some cases. However watertight and duration demands required the limitation of the cracks on the roof, so the acceptable compromise solution could be accomplished only by using a significant quantity of additional reinforcement.

3.3 Biological treatment structure in the town Székesfehérvár

According to the upgrade and enlargement of the Municipality's Wastewater Treatment Plant a structure block of 59.0x55.5 m^2 overall area has been built with a pre-settling tank, two aeration tanks, a denitrification tank and a secondary settling tank in it (see in *Figs. 7 to 8*). The useful water volume capacity of the whole structure complex is about 13000 m^3.

Fig. 7 Construction process of the block biological treatment structure before finishing

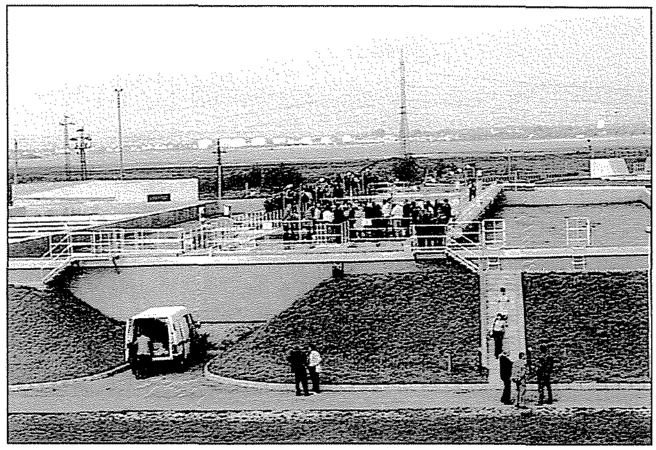


Fig. 8 Celebration of the taking over procedure of the finished biological treatment structures

The structures of considerable area have been built in a deep-seated land which had been used for wastewater sludge disposal before. Naturally, after sludge removal, which had been required also for environmental reasons, the soil change needed by the foundation of the structures was accomplished. The structures were lifted compared to the compacted terrain because of the hydraulic system of the Wastewater Treatment Plant as a whole and on the other hand the uplift problems of the structures did not need further consideration in detail. The functionally co-ordinated structures are constructed without any longitudinal dilatations but in cross direction the settling basin cell is structurally independent due to different water levels. The double longitudinal wall can be seen properly in Figure 9. The load systems of the structures are impacted advantageously by just about total earth backing as a consequence of thermal protection.

4. UPGRADED SWIMMING POOLS

As task emphasised in recent years has been the upgrade of numerous spas and swimming pools of great tourist significance. According to the relevant health requirements the water quality demands must be guaranteed, which can usually be achieved by built in water recycling equipment and supplied water cleaning. As distinguished from the former practice the basins are converted to "smooth" water-plane form, namely the overflow water is collected via collecting channels then cleaned and recirculated. This technologic demand requires structural reconstruction as well. According to experience it is also reasonable because of the old age and deterioration, especially in case of thermal and medicinal baths. Naturally, in addition to structural renovation, new coverage is also applied for hygienic reasons.

This kind of monolithic structure is not to be qualified as a special task, neither from designing, nor building aspects. However, heat expansion movements have to be taken into consideration, especially in case of thermal baths. If movement joints are absent, the adequate receiving of stresses has to be ensured.

For emphasis an example can be mentioned. This is the technical solution to the indoor basins of the most famous spa in Budapest (Széchenyi spa). The cross section of the monolithic reinforced concrete duct for services, located between the two adjacent basins built during the reconstruction, can be seen in *Fig. 9*. In this passable duct are pipes and the mechanical equipment of the Jacuzzi basin. (The technological and mechanical equipment of water treatment are set up in the sub-

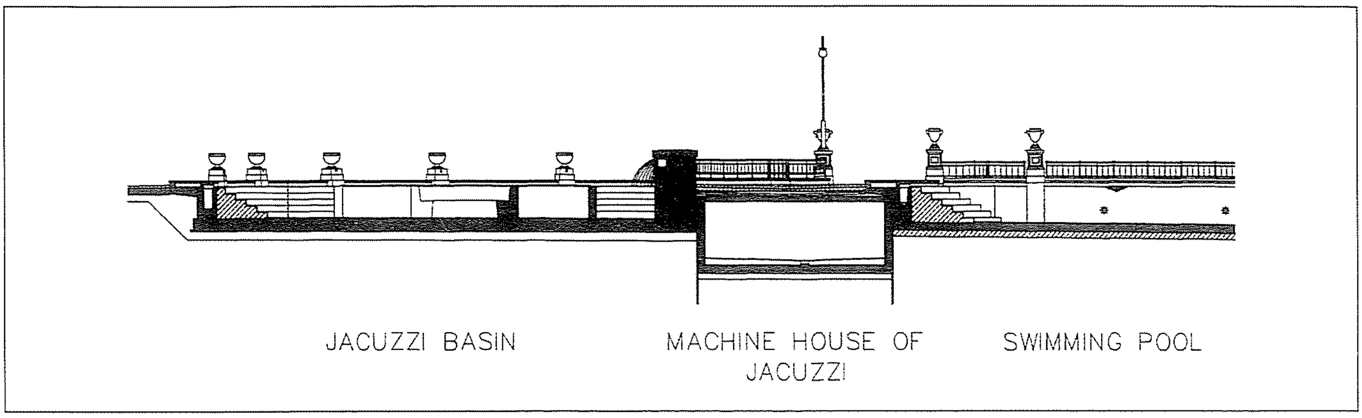


Fig. 9 Cross section of basins and duct for services (Széchenyi spa)

ground floor of the swimming pool building.) The dependency of the structures with different depths has been guaranteed by the expert designers, so no additional stresses derive from joint constraint factors. *Fig. 10* shows the renovated basins. The Széchenyi spa is justly regarded as a monument, its architectural details preserves its historic image and feeling.

5. CONCLUSIONS

The examples above explain the fact that almost all of the structures of technologic purposes are built using monolithic methods in Hungarian engineering practice. The structural designer has to very carefully consider the characters of functionality, because the economy of a complex project can be guaranteed only by optimal co-operative solutions between the different professional fields. There is no possibility to design a structure with an ideal force system in all of cases because the high level realisation of technologic and functional aims can actually be more important. Structures with "prescribed geometry" should also be carefully formed from a structural point of view and in consideration of expedient execution possibilities. The latter aspect is considered in case of execution under adverse subsoil conditions in areas of high groundwater level, which need special drainage solutions.

Watertight material conditions have to be guaranteed for all water-bearing structures, and this applies to both the design as well as the building technology aspects. According to the developments of recent decade, Hungarian structural engineers work with up-to-date formwork and concreting technologies. Structures realised on that basis are generally of excellent quality. Geometric dimensional tolerance is an important requirement in many cases, especially because of the

Fig. 10 View of Jacuzzi and swimming basins

mechanical appliances connected. Last but not least the proper surface finishing is also important from aesthetic reasons.

6. REFERENCE

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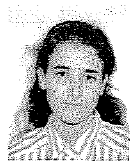
Dr. László Tóth holds an M.Sc. and Dr. Univ. degrees in civil engineering. He took his thesis in 1967 and began to work for MÉLYÉPTERV Enterprise as a structural designer. Designed a great number of potable water basins, clarification basins, reinforced concrete water towers, wastewater treatment structures, digesters, special structures. Developer of the prefabricated pasted reinforced concrete panel system used in the scope of civil engineering. Multi-inventor, publishing regularly, a member of the direction board of the Construction Execution Department of ÉTE and the Hydraulic Engineering Department of the Hungarian Hydrologic Society. Member of the Hungarian Group of *fib*, general director of MÉLYÉPTERV KOMPLEX Co.

RECYCLED GLASS AGGREGATE FOR LIGHTWEIGHT CONCRETE

Assoc. Prof. Zsuzsanna Józsa - Rita Nemes

Manufacturing of a series of cellular pellet products made of waste glass has been started recently in Hungary. This new product was tested for suitability as lightweight aggregate for lightweight structural concrete or for further possible applications. This article summarises the research results until now and deals with future possibilities.

Keywords: lightweight aggregate, lightweight concrete, recycled glass, glass aggregate, glass pellet, environment

1. INTRODUCTION

The self-weight of concrete structures (bridges, tall buildings etc.) may be considerably high compared to the other loads (live load, meteorological load etc.). Density of normal weight concrete (NWC) is between 2200 and 2600 kg/m^3. Density of the aggregate influences mainly the density of concrete, since the volume of the aggregate is significant in the mixture. Using concrete with lower density and the same load bearing capacity requires a smaller cross-section then in case of NWC. Concrete with lower density has also better thermal insulating properties. The density of concrete is lower if there are air voids in the mixture. There are three possibilities to create artificial air voids in the concrete:

1. air in the porous aggregate: lightweight concrete
2. air in the cement paste: cellular concrete
3. air between the aggregate particles: no-fines concrete (Neville, 1995)

For load bearing elements and structures lightweight aggregate concrete (LWAC) may also be used. Structural lightweight aggregate concrete has a density between 1300 and 2000 kg/m^3 and should reach at least 20 N/mm^2 compressive strength.

Application of lightweight aggregate concrete is nowadays not very common in Hungary (Józsa 2000). On the other hand the usage of lightweight aggregates produced out of waste materials has an enormous environmental advantage. Geofil Ltd. produces the so called pellets "*Geofil bubbles*" out of industrial waste material of very high glass content using a recycling technology.

During the last year an experimental study was carried out at the Department of Construction Materials and Engineering Geology at the Budapest University of Technology and Economics to test the suitability of such recycled glass aggregates as lightweight aggregate for lightweight concrete.

2. LIGHTWEIGHT AGGREGATES

2.1 Types

Lightweight aggregates (LWAs) may be made of natural materials (for example: tuff, lava, pumice), may be manufactured out of natural materials (for example: expanded clay, expanded perlite, vermikulit), or can be industrial by-products and wastes (Rudnai 1966).

General requirements of LWAs are:

- low bulk density (max. 1200 kg/m^3) and low particle density (max. 2000 kg/m^3)
- pressure resistance
- thermal insulation capability
- mechanical and chemical resistance
- fire-resistance
- frost-resistance
- shape keeping.

Several LWA products are available at the international market made of expanded clay, shale and glass products. (*Table 1.*) (Faust, 2000; *fib*, 2000) Several famous bridges, buildings, platforms, airport terminals, power stations, slabs were constructed using LWAC like the following examples (*fib*, 2000):

towers:
Marina City Towers, Chicago – Materialite (1962)
Picasso Tower, Madrid – Arlia (1988)
Nationalsbank Corporate Center, Charlotte – Solite (1992)

bridges:
Koningspleijbridge, Arnhem – Lytag (1986)
Støvset bridge, Norway – Liapor (1994)
Stolma bridge, Norway – Leca (1998)

platforms:
Heidrun tension leg platform, North See – Liapor (1995)
Troll West floating platform, North See – Leca (1995)

2.2 Geofil

Under the name "*Geofil bubbles*" a pellet product from industrial waste material of high glass content is manufactured

Table 1 LWAs at he international market

Material	Product name	Nationality
expanded clay	Liapor	German
expanded clay	Leca	Denmark
expanded shale	Materialite	USA
expanded glass	Liaver	German
pulverized fuel ash	Lytag	UK
expanded clay	Arlita	Spain
expanded clay, shale. slate	Norlite	USA
expanded shale	Baypor	USA
expanded shale, slate	Solite	USA

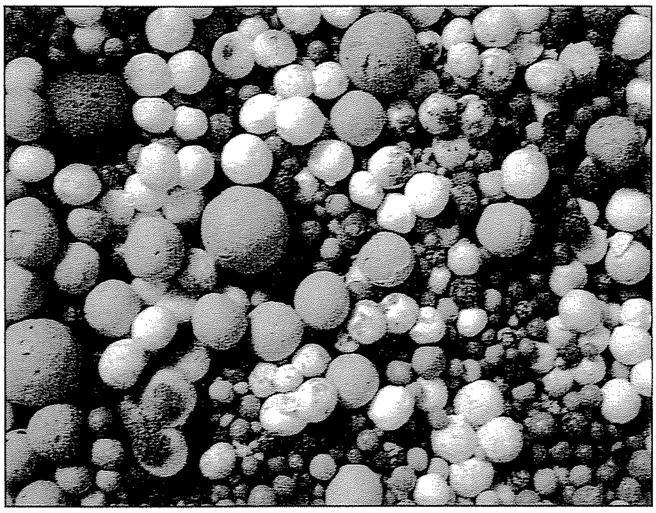


Fig. 1 Geofil LWAs

by recycling technology. The waste glass materials may have organic and inorganic impurities. Waste materials of high glass content are ground to an appropriate particle diameter. Homogenisation is carried out with a blowing agent dosed according to the amount of impurities in the raw materials. Granulation process is carried out by adding melting point reducer and viscosity modifying agents. The granulate is heat cured and coated to decrease water absorbing capability. Firing is carried out in a rotary furnace.

The product is a lightweight artificial gravel with a diameter of 1 to 25 mm having primarily heat and sound insulating properties *(Fig. 1)*. This product has good bonding capability if embedded in gypsum, cement or resin matrix. This product has three main types called *Geofil A*, *B* and *C*. Type *A* is used for thermal insulation, type *B* is used for structural LWAC with thermal insulating property and type *C* is used for structural LWAC.

3. LABORATORY TESTS

3.1 Test programme

The first phase of the laboratory tests was the measurement of the main properties of *Geofil* (three kinds of type *Geofil A*, seven kinds of *Geofil B*, six kinds of type *Geofil C* with several grain sizes, a total of 25 samples), on the other hand *Liapor* (*Liapor 3 4/8*, *Liapor 4 4/8* and *Liapor 6.5 4/8*), and *Liaver* (*Liaver-B 2/4*). The aim of tests was to check the applicability of *Geofil* aggregates and comparing the main properties of *Liapor*, *Liaver* and *Geofil* aggregates (*Table 2*).

Parallel to measuring material properties, LWAs were tested in cement mortar too. Tests were carried out on prisms of 70×70×250 mm sizes when the maximum size of the aggregate pellets was bigger than 12 mm, and 40×40×160 mm sizes for smaller aggregates. Flexure tensile strength was measured

Aggregate type	Density* of concrete kg/m^3	Compressive strength* N/mm^2	Flexural-tensile strength* N/mm^2
Geofil A1 12/16	1436	15,0	3,0
Geofil A1 4/8	1404	21,4	3,2
Geofil A2 8/16	1407	18,6	2,8
Geofil C1 2/12	1674	35,1	4,9
Geofil B1 4/16	1737	41,9	8,3
Geofil B2 4/16	1787	44,3	8,3
Geofil B3 4/16	1692	35,9	7,4
Geofil B4 4/16	1727	40,1	7,6
Geofil B6 8/16	1791	36,7	6,9
Geofil C4 1/16	1959	56,5	9,1
Geofil B6 8/16	1791	34,8	6,9
Geofil B6 8/16 + 2% silica	1789	38,0	7,8
Geofil B6 8/16 + 10% silica	1788	39,9	8,9
normal	2298	49,1	9,1
Geofil C2 2/12	1858	45,8	7,4
Geofil C2 2/12 +10% silica	1953	57,9	8,8
Liapor 6.5 4/8 +10% silica	1784	48,0	7,0

Table 3 Results of preliminary experiments
(* average of three measurements)

at the age of 7 and 28 days and the compressive strength on the remaining half specimens. The composition of cement mortar was the same in all cases:

cement:	385 kg/m^3	CEM I 42,5
natural sand:	465 kg/m^3	0/4 mm
water:	160 kg/m^3	
LWA:	90 % of vol. of the bulk, 12 types of *Geofil* and *Liapor 6.5 4/8*	
super-plasticiser:	SIKA Viscocreate-5 Neu 1,0 % of the cement weight.	

Aim of these preliminary experiments were:
- to compare different lightweight aggregate concretes using different types of *Geofil* LWAs within the possible workability range
- to find the highest available compressive strength using *Geofil* LWAs having high strength and high particle density
- to test the effect of silica fume (by different dosages)
- to compare the compressive strength of concretes of same composition prepared by using normal gravel aggregate (ρ_b=1540 kg/m^3) and lightweight aggregates (*Liapor* ρ_b=685 kg/m^{3}and *Geofil* ρ_b=805 kg/m^3) (*Table 3*).

Based on the preliminary experiments five *Geofil* aggregates

Table 2 Summary of the measured properties of the tested LWAs

Aggregate type	Bulk density kg/m^3	Particle density kg/m^3	Solid density kg/m^3	Gap ratio %	Particle porosity %	Water absorption m%	Water absorption V%	Crushing resistance N/mm^2	Modulus of fineness
Geofil A	260-500	480-900	2,30-2,45	40-50	63-79	18-40	9-22	0,3-3,0	5,8-8,0
Geofil B	260-600	460-1100	2,05-2,30	41-59	50-79	0,4-6,3	0,4-2,7	0,4-2,7	6,0-8,5
Geofil C	600-1100	1000-1850	2,10-2,35	35-43	14-52	0,1-1,1	0,3-1,3	4,8-15,1	6,7-7,1
Liapor	340-700	650-1300	2,50-2,55	46-52	50-74	20-45	25-30	1,5-10,5	7,0-7,7
Liaver	180-190	320-340	2,35	43-44	85-86	50-60	18-20	1,3-1,4	6,0

Compressive strength in N/mm^2				
Aggregate type	Quantity of aggregate bulk			
	90%	60%	50%	40%
Geofil A1 12/16		16,9		
Geofil A3 8/16		17,9		
Geofil B1 4/16		25,8		
Geofil C1 2/12		48,9		54,1
Geofil C5 4/8	20,5	42,9	41,3	43,7

Table 4 Compressive strength of concrete (average of three measurements)

were selected for the standard tests in the second phase of the laboratory tests. The composition of cement mortar was changed to:

cement: 400 kg/m^3 CEM I 42,5
natural sand: 500 kg/m^3 0/4 mm
water: 140 kg/m^3
LWA: 40, 50, 60 or 90 % of vol. of the bulk, 5 types of *Geofil* aggregates
super-plasticiser: SIKA Viscocreate-5 Neu 1,2 % of the cement weight

Aim of these experiments was to determine the compressive and tensile strengths, the Young's modulus and the water tightness of LWAC. Tests were also carried out on specimens made of pure mortar, without aggregates. So a comparison could be made between the results of the further tests. In case of lightweight concrete the quantity of aggregate was calculated in the percentage of the volume. Different volume ratios were used. 100% means the maximum applicable volume of aggregate. Different aggregate ratios were tested only in the cases of two aggregate types. 60 % of the volume of the bulk was used for all selected aggregates. This is the volume where the density of concrete is 2000 kg/m^3, here the aggregate's bulk density was the highest. Even in the case of using the lightest aggregate a compressive strength over 12 N/mm^2 could be achieved. (*Table 4.*) The possible maximum amount of aggregate was 90 % because of the workability requirements.

The density of concrete may be calculated with the formula

$$\rho_{LC} = \frac{V_a \cdot \rho_a + V_m \cdot \rho_m}{V_a + V_m}$$

where V is volume, ρ is density, index a is aggregate and index m is cement mortar. (*Table 5.*) Compaction was made on a vibrating table. Specimens were stored under water till testing.

3.2 Material properties

LWAs were tested according to the standard prEN 13055-1 *Lightweight aggregates - Part 1: Lightweight aggregates for concrete and mortar* and to standards referred in the following list (*Table 2):*

loose bulk density according to prEN 1097-3
particle density according to prEN 1097-6)
density according to EN 196-6
aggregate size, particle distribution according to EN 933-1
water absorption
crushing resistance

All values in *Table 2.* are the averages of minimum three measurements.

Table 5 Densities of concrete (average of measured values of all specimens)

Density of concrete in kg/m^3				
Aggregate type	Quantity of aggregate bulk			
	90%	60%	50%	40%
Geofil A1 12/16		1702		
Geofil A3 8/16		1609		
Geofil B1 4/16		1793		
Geofil C1 2/12		1931		1979
Geofil C5 4/8	1517	1751	1829	1907

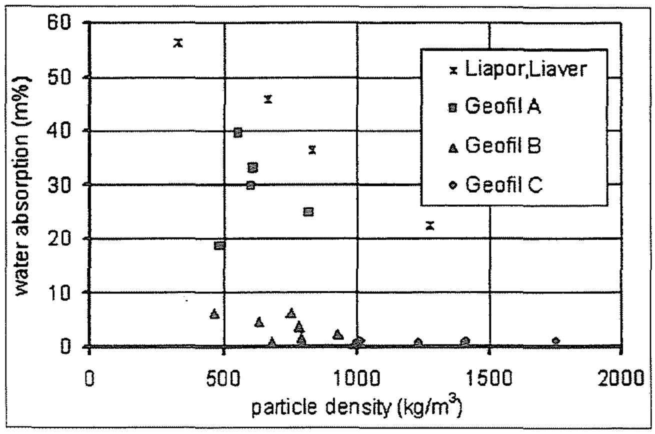


Fig. 2 Relationship between the particle density and the water absorption

Water absorption of LWAs *Geofil type B* and *Geofil type C* is low. The relationship between water absorption and particle density is well determined. *Geofil type A* has higher water absorption than the previous ones but it is in all cases lower than in case of *Liapor* or *Liaver*. (*Fig. 2.*)

Crushing resistance was determined using Hummel cylinder. Force and compressive deformation were measured and recorded in every five seconds. Values in *Table 2.* are the crushing resistances *(C)* at the compressive deformation of 20 mm, calculated by the following equation:

$$C = \frac{L+F}{A} \quad \text{where}$$

L is the force exerted by the piston (self-weight of the piston) (N)

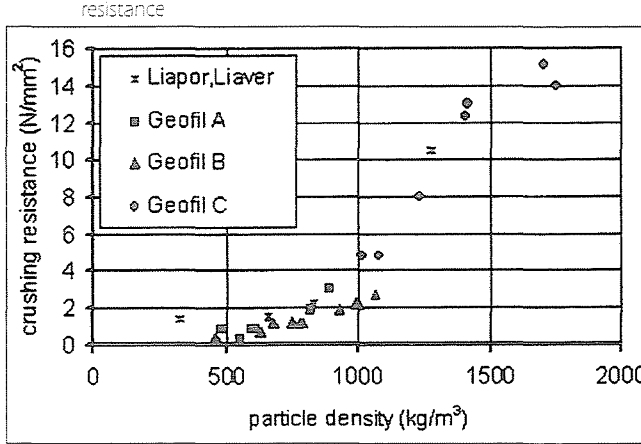
F: is the compression force (N)

A is the area of the piston (mm^2).

In this respect all the tested LWAs behave similarly (*Fig. 3*). In case of *Geofil C* the crushing resistance is outstanding.

Alkali resistance was tested with the method worked out by the German Community for Reinforced Concrete (DAfStb

Fig. 3 Relationship between the particle density and the crushing resistance



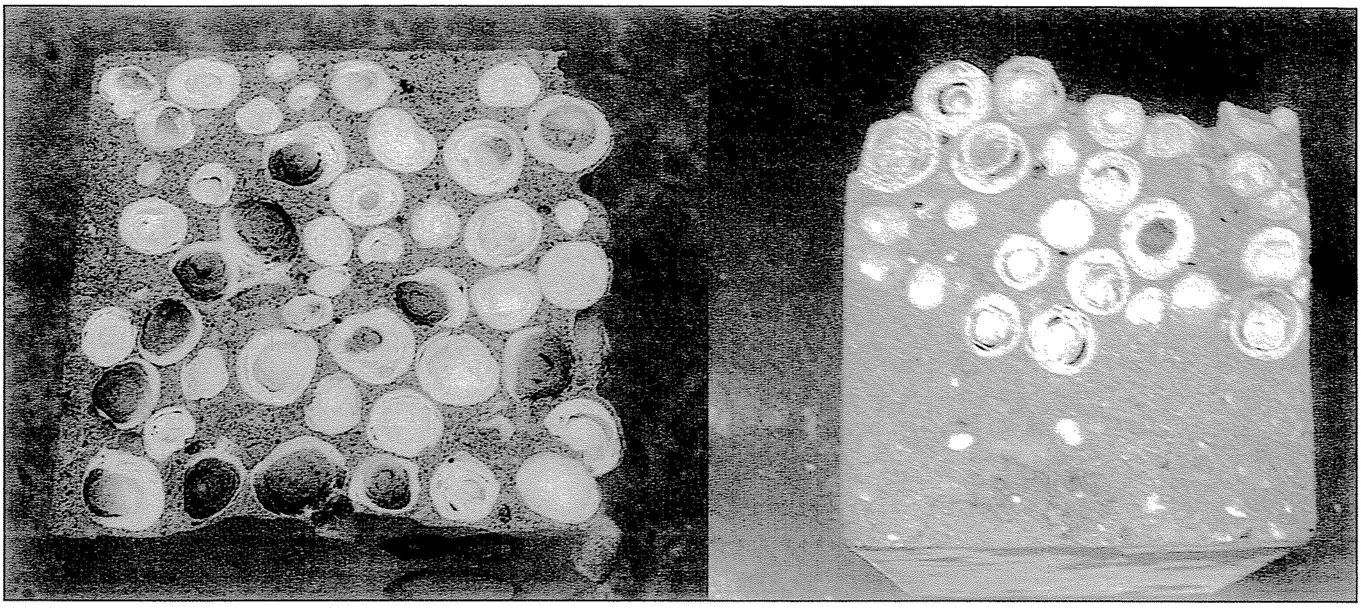


Fig. 4 Typical surface of failure. Left: well compacted Right: upfloating of the pellets

1986). All of the tested products proved to be alkali resistant according to this test.

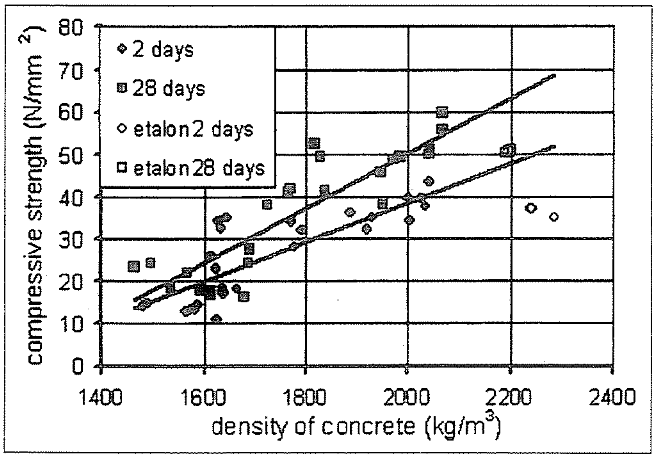
3.3 Observations of experiments with LWAC

Generally, the strength of aggregate grains (crushing resistance) influences the compressive strength of concrete but has no considerable effect on the tensile strength. In case of same strength classes the density of Geofil LWAC is 15 to 20 % lower than NWC (*Table 3*). A minor increase in the compressive strength occurred by the addition of silica fume. The usage of silica fume, however, made it necessary to apply more plasticiser. The mode of failure depended on the strength of aggregate and that of the mortar:

- If the aggregate pellets were crushed then the strength of the cement mortar was higher than that of the pellets.
- If the aggregate pellets lost their bond to the cement mortar matrix then the strength of the cement mortar was lower than that of the pellets.
- If only a part of aggregate pellets were crushed and others lost their bond, then the strength of the mortar was close to that of the pellets.

Quality of *fresh concrete* is very important. After mixing, the consistence for workability has to be checked with slump or flow tests. Very light pellets may float up. An example in *Fig. 4* can be seen for the *Geofil type A*. The pellets floated at plastic consistence. An earth moist consistence should be maintained. Using higher density aggregate, plastic consistence can be applied too, but the cement mortar may flow down from the surface of the aggregate. If the water absorption of the LWA is higher than a few percent, aggregates have to be pre-saturated with water or it has to be regulated by other means.

Fig. 5 Relationship between the density of concrete and the compressive strength



The *compressive strength* was tested at the age of 2 and 28 days on concrete cubes of 150×150×150 mm sizes 3 of each. If the 2- and 28-day strengths were compared, the early compressive strength proved to be high. The 2-day strength was roughly the 75 % of that of the 28-days. This value is higher than that of NWC. It depends on the applied sort of cement too. The values are represented in *Fig. 5*, where the relationship between the density and the compressive strength in case of some *Geofil* concretes and reference cement mortars are given. In case of lower concrete densities was the standard deviation of compressive strength higher. The reason is partly due to the unequal bigger aggregate grains. Lightweight *Geofil* concrete with the same properties may be prepared by using different aggregate types (for example: *Geofil C5 4/8* 60% and *Geofil B1 4/16* 50% ρ_{LC}=1660 to 1680 kg/m^3). However, the lighter the aggregate is, the worse is the workability. It is responsible partially for the different compressive strength too.

The *tensile strength* was determined by two methods: the flexural-tensile tests were made in all cases and splitting tensile tests in the cases of the three considerably different aggregates (*Geofil A1 12/16*, *Geofil C1 2/12*, *Geofil C5 4/8*).

The splitting-tensile tests of the three specimens were performed on a standard cylinder (diameter: 150 mm, height: 300 mm) at the ages of 2 and 28 days. These measured values were considered as tensile strengths. The flexural-strength tests were carried out on three prism of 70×70×250 mm, span: 210 mm, by one concentric load at the midspan. Based on our measurements the flexural-tensile strength is half of the splitting-tensile strength.

The ratio between the splitting tensile and the compressive strengths was 5 to15 %, which is similar to that of the normal concrete. The lowest measured value was in the case of *Geofil* C5 4/8 90%. Here the least amount of cement mortar and the maximum possible amount of aggregate was applied. The ratio of the tensile and the compressive strengths were the high-

Young-modulus N/mm^2	Geofil A3	Geofil C5
measured	10731	20273
calculated to (1)	12273	19515
calculated to (2)	10943	18441

Table 6 Measured and calculated values of Young's modulies

est when relatively much cement mortar was applied and when there was a considerable difference in strength between the cement mortar and the aggregate grains (for example: *Geofil A2* 50%).

The *Young's modulus* was measured at the age of 2 and 28 days, too. The specimens were 120×120×360 mm prisms, 3 of each. Only two series were tested, one with higher strength and one with lower strength using the same aggregate volume percent (*Geofil A3* 60% and *Geofil C5* 60%). The force as well as the axial and transverse deformations were measured and recorded continuously. The force was increased to one third of the expected failure force and unloaded three times, and then it was loaded until failure. The Young's modulus could be calculated from the force vs. deformation diagram.

As the density of concrete decreases, the Young's modulus decreases, too. The Young's modulus of *Geofil* concrete is only 70% of that of the normal concrete of the same compressive strength class. In case of normal concrete, if the aggregate volume increases, the Young's modulus increases too, but in case of lightweight aggregate concrete, the more aggregate is applied, the Young's modulus will decrease. According to Reinhardt (1995) in case of normal concrete, the changing of the aggregate volume results in a change of E by 25%, whereas with lightweight aggregate concrete this difference may reach 50%. *Geofil* concrete acts the same way.

According to the CEB-FIP Model Code 90 the Young's modulus can be calculated with the following formula:

$$E_c = 0{,}85 \cdot E_{ci},$$

where

$$E_{ci} = E_{co} \cdot \sqrt[3]{\frac{f_{ck} + \Delta f}{f_{cmo}}} \quad \text{or} \qquad (1)$$

$$E_{ci} = E_{co} \cdot \sqrt[3]{\frac{f_{cm}}{f_{cmo}}}, \qquad (2)$$

where

E_{ci} Young's modulus at the age of 28 days (MPa)
$_{ck}$ characteristic compressive strength (MPa)
Δf = 8 MPa
f_{cmo} = 10 MPa
E_{co} = 2,15 x 10^4 MPa

Table 7 Relationship between the Young's modulus and the concrete density

Aggregate type	E N/mm^2	ρ kg/m^3	E / ρ
Geofil A3 8/16 60%	10731	1394	7,7
Geofil C5 4/8 60%	20273	1664	12,2
reference mortar	28300	2225	12,7

Relatively few experiments have been performed till now, but the measured and calculated values have been compared. Nevertheless, it can be stated that both calculated values follow the measured ones well. Consequently, these formulas are both useful. (*Table 6.)* Defining a more precise value needs more examination. In respect of the Young's modulus – concrete density ratio, it can be seen that with the heavier aggregate the ratio is almost equal to that of the NWC, but with very light aggregate it is much lower (*Table 7).*

Water tightness has been tested on standard 200×200×120 mm specimens of each series. The water pressure was step by step increased to 6 bars, and maintained for 24 hours. The penetration of water into the specimens was measured. In all cases they were very low, between 5 and 10 mm. Consequently, the water tightness may suit the higher requirements.

4. FUTURE WORK

For the different areas of utilisation, further tests must be carried out according to the area of usage. The main ones should however be mentioned here: frost resistance, shrinkage, possibility of steam curing, possibility of preparing pumpable concrete, bounding to steel (anchorage), heat and sound insulating capability, fire resistance, possibilities of self-compacting concrete, possibilities of fibre-reinforced lightweight aggregate concrete, optimal mixing order, etc.

5. CONCLUSIONS

Different types of "*Geofil* Bubbles" manufactured of waste glass using recycling technology were tested at the Department of Construction Materials and Engineering Geology, Budapest University of Technology and Economics as lightweight aggregate for concrete. The following conclusions can be drawn:

- Material properties of these new lightweight aggregates show the suitability for lightweight concrete:
  - type Geofil A is good for thermal insulation
  - type Geofil B for structural LWAC with thermal insulation properties
  - type Geofil C is the best for structural LWAC
- Water absorption of type Geofil B and Geofil C is low, it should be important for pumpable LWAC.
- This new material is alkali resistant (according to DAfStb 1986).
- Similar compressive strengths can be reached by LWAC as in case of NWC *(Fig. 5).* Even higher strength can bereached by additional silica fume or cement or by lower water-cement ratio.
- Admixtures are required to have appropriate workability.
- 2 days compressive strengths of LWAC was 75% of that of 28 days.
- Water tightness of Geofil concrete can be as good as NWC.
- Dead load of the structure can be considerably decreased by using this new aggregate

6. ACKNOWLEDGEMENT

Authors thank for the fruitful collaboration to László Hoffmann inventor of "*Geofil bubbles*". (Patent Reg. Nr.: PCT (HU 99) 00017, 12 Nov 1998 - Geofil Ltd. Tatabánya)

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CHLORIDE BINDING IN CONCRETE


Prof. György Balázs – Katalin Kopecskó

Chloride ions may be present in concrete from its constituents, from deiceing salts, from seawater or from PVC due to fire. In the last decades application of deiceing salts induced considerable corrosion of steel reinforcement. Research was directed to understand the mechanism of corrosion in order to be able to avoid corrosion and repair of corroded members. One of the major issues is to find the critical chloride content that can be still bound by various cements and the way of chloride binding. On the other hand, to define the circumstances of initiation of corrosion of steel reinforcement.

Present paper summarizes the test results on chloride ion binding capacity in concrete carried out at the Department of Construction Materials and Engineering Geology, Budapest University of Technology and Economics in the last couple of years.

Keywords: corrosion, steel reinforcement, deicing salts, chloride ion binding, Kuzel's salt, Friedel's salt

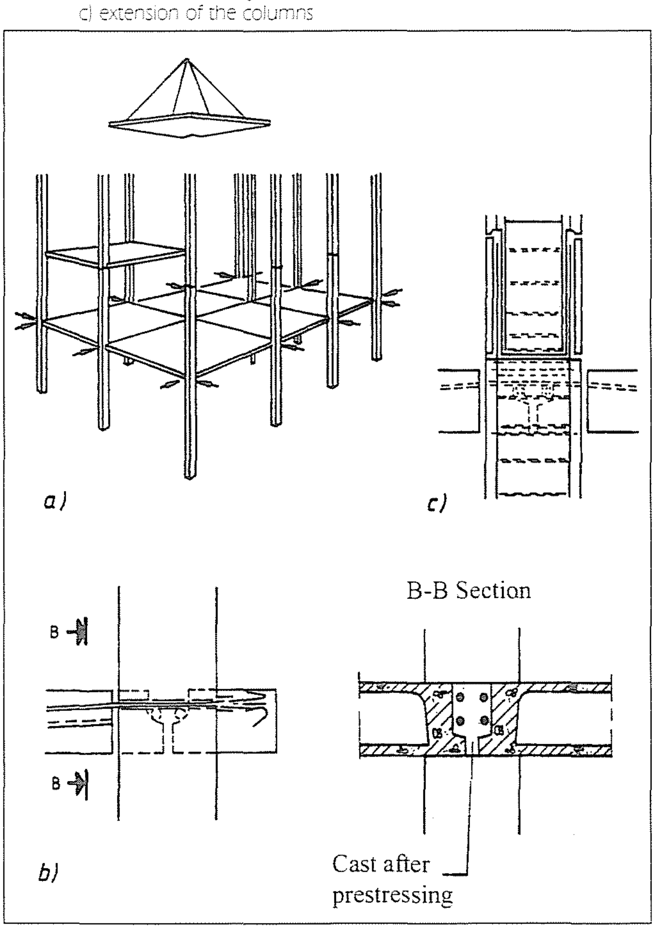
1. INTRODUCTION

Chlorides may be present in concrete due to the following reasons:

- during preparation mixed together with the constituents (cement, aggregate, water, admixture)
- during its lifetime:
  - by using deicing salts
  - originating from seawater
  - produced during fire (e.g. decomposition of PVC).

Herewith we deal only with the first two cases.

Fig. 1 IMS system (Balázs, 1996b)
a) principles of the system b) slab to column connection
c) extension of the columns



1.1 Chlorides originating from the constituents

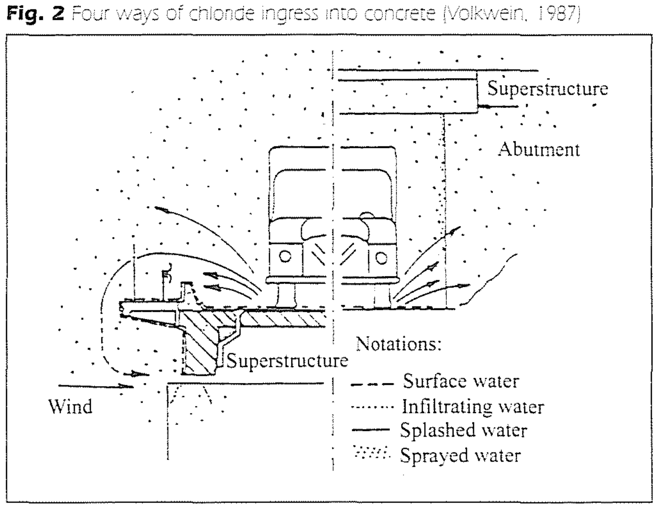
Admixtures, which include $CaCl_2$, were used to increase the hydration rate of cements. Its typical example was in Hungary the Tricosal SIII admixture before the War (Balázs, 1996a). In order to avoid $CaCl_2$ induced corrosion, Kalcidur NV admixture came into use after the War that included $CaCl_2$ as well as $NaNO_2$ as inhibitor in the same amount.

In case of a special construction system (called IMS), prestressing tendons deteriorated due to the chloride content of the filling mortar. Principle idea of the IMS system is that concrete slabs are prestressed against concrete columns by post tensioned tendons in two directions (*Figs. 1.a and 1.b*). After prestressing the channels of tendons are filled with a mortar (*Fig. 1.c*), which included calcium chloride. This $CaCl_2$ induced the corrosion of the prestressing tendons (Balázs, 1996b).

1.2 Chlorides originating from deicing salts

Despite of the strict regulations of environmental protection, NaCl as deicing salt is often used in several countries during winter. Observed chloride corrosion induced research to sub-

Fig. 2 Four ways of chloride ingress into concrete (Volkwein, 1987)



stitute NaCl with other deicing agents; however, efficient and economic solution is still not available.

Chlorides from deicing salts reach the concrete members dissolved in molten ice or snow but also with splashed or sprayed water (*Fig. 2*).

Chloride transport in hardened concrete occurs through the capillaries by the pore water, therefore, the capillary system of concrete has a major importance.

2. IMPORTANCE OF CHLORIDES IN THE PROCESS OF CORROSION OF REINFORCEMENT

One of the reasons of widespread use of reinforced concrete is the passive layer on the surface of reinforcement produced during the hydration of cement. This layer protects the reinforcement from corrosion (Balázs – Tóth, 1997).

Corrosion can start even without chlorides whenever the following three conditions are simultaneously fulfilled:

- alkalinity of concrete is lost due to carbonation
- water is available in the capillary pores
- oxygen is available at the level of reinforcement penetrating through the concrete cover.

Presence of chloride ions change the type of corrosion from surface corrosion to pitting corrosion.

Chloride ions take water, which increases the electric conductivity of concrete around the reinforcement. (electrochemical corrosion).

Chlorides will not be bound in carbonated concrete.

3. CHLORIDE BINDING IN GENERAL

Researchers agree in that *only those chloride ions induce corrosion, which are not bound chemically and are in the pore water as a solution.* However, researchers disagree in how, and how much chlorides can be bound.

It is known since approximately 100 years that chloride ions from $CaCl_2$, which are present by mixing of concrete will be bound in form of Friedel's salt by C_3A:

$$\begin{array}{ccccc} C_3A & \bullet & CaCl_2 & \bullet & 10\,H \\ \text{tricalcium-} & & \text{calcium-} & & \text{water} \\ \text{aluminate} & & \text{chloride} & & \end{array}$$

According to the above formula $CaCl_2$ react with the C_3A clinker of cement. This type of chloride binding mechanism was supported by our experiments to increase the natural hardening (Balázs, 2001). Our experiments also indicated that the amount of bound chloride ions might be less in steam-cured concrete.

The form of Friedel's salt did not give an answer why sulphate resistant cement (CEM 32.5 S) binds chloride ions, without C_3A clinkers.

Application of deicing salts differ from the above situation in two ways:

- NaCl is used instead of $CaCl_2$
- the age of concrete is at least one month by the opening of a bridge. By this time hydrates are formed from the cement constituents. In order to be able to bind the chlorides of deicing salts, any of the hydrates are needed to be dissolved.

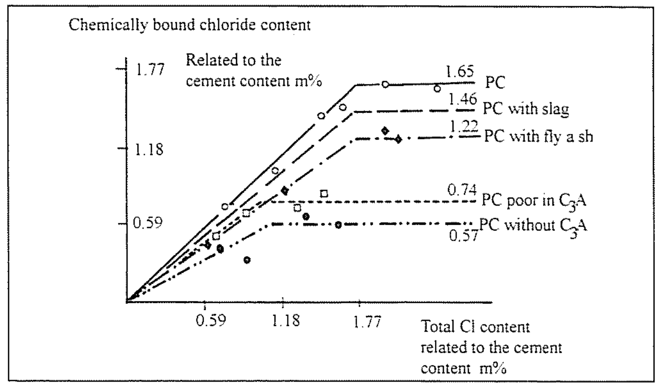


Fig. 3 Chemically bound chloride content as a function of the type of cement (Lukas, 1983)

Volkwein (1987) assumed the following mechanism, based on the investigation of 22 years old bridges:

$$\begin{array}{ccccccc} 3\,[C_3(A,F) \bullet Cs \bullet H_{12}] & + & 2\,CH & + & 4\,NaCl & + & 16\,H \rightarrow \\ \text{(monosulphate)} & & \text{(portlandite)} & & \text{(salt)} & & \text{(water)} \end{array}$$

$$\begin{array}{ccccc} 2\,[C_3(A,F) \bullet CaCl_2 \bullet H_{10}] & + & [C_3(A,F) \bullet 3Cs \bullet H_{32}] & + & 4\,NaOH \\ \text{(Friedel's salt)} & & \text{(ettringite)} & & \text{(sodium-hydroxide)} \end{array}$$

According to this equation:

- monosulphate dissolves
- Friedel's salt is produced both from C_3A and C_4AF
- ettringite is re-established from monosulphate
- sodium-hydroxide (NaOH) is released, which increases the sensitivity to alkali-aggregate reaction.

Chloride ion content of concrete can be determined by the classic Mohr method and the chloride content (bound and free) should be related to the cement content, since only the cement clinkers can bind chloride ions.

The considerable amount of chloride induced corrosion led to research on the circumstances of chloride corrosion. On the other hand research started in two directions to avoid corrosion of reinforcement:

- to determine the maximum amount of chlorides to be bound and the circumstances of binding of the chlorides
- to determine the amount of chlorides that start to induce corrosion. This amount is considered to be the allowable chloride content (Breit, 2001).

Comparison of test results is often difficult because the type of cements and measuring methods differ. A series of measurements are shown in *Fig. 3* by Lukas (1983) indicating considerable differences in chemically bound chloride contents in the function of the cement type. Lukas considered the chloride content to be bound, which is soluble in alcohol.

4. RESEARCH AT THE DEPARTMENT OF CONSTRUCTION MATERIALS

4.1 Chloride binding capacity of C_3A, C_4AF and C_3S clinkers

Reason of research supported by the Hungarian Research Found (OTKA 3000) was to study how clinkers bind NaCl. In

Name and formula of hydrate	Temperature range of dissotiation, °C	H		Composition, m%			
		mol	m%	C	A	SO_3^{2-}	Cl⁻
ettringite $C_3A \cdot 3Cs \cdot H_{32}$	130-150	10	26.54	49.58	15.02	35.40	-
monosulphate $C_3A \cdot Cs \cdot H_{12}$	150-250	10	44.33	55.20	25.09	19.71	-
tricalcium-aluminate-hexahydrate C_3AH_6	280-430	5	33.33	62.27	37.73	-	-
trichloro-aluminate-hydrate $C_3A \cdot 3CaCl_2 \cdot H_{30}$	240-290	10	29.87	27.89	16.91	-	55.20
Friedel's salt $C_3A \cdot CaCl_2 \cdot H_{10}$	280-340	5	23.63	44.13	26.75	-	29.12

Table 1 Formulas and data of hydrates originated from C_3A

one part of the study NaCl was added to the samples during mixing, while in the other part of the study the hardened samples were immersed into chloride solution in various ways or the samples were sprayed, respectively.

Samples of 10×10×50 mm were prepared out of C_3A, C_4AF and C_3S clinkers. The samples made us possible to have a strength analysis record for every chemical investigation by cutting 10 mm slices. The investigations took place at ages of 1, 28, 56, 90 and 180 days. Investigation methods were thermal (TG, DTG, DTA) and X-ray diffraction analyses as well as splitting strength analysis. Derivatograms are evaluated by using the values *Table 1.*

Summary of the test results (Balázs-Csizmadia-Kovács, 1997):

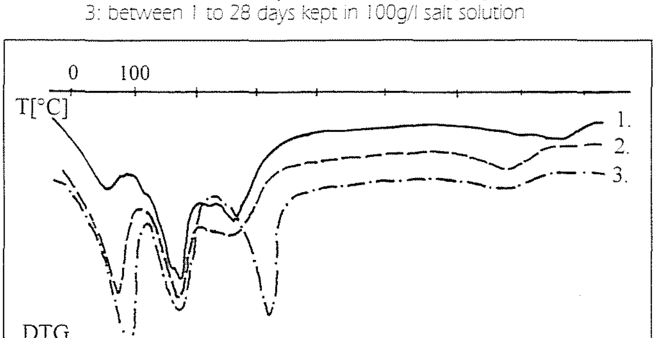
C_4AF binds chlorides in the form of monochloro-aluminate-hydrate (Friedel's salt) or monochloro-ferrite-hydrate. Analogous aluminate or ferrite hydrates form solid solution with each other. *Thermal and X-ray analysis showed that the chloride ion either from NaCl added to the mixing water or from outer source into the already settled cement may be bound in the presence of either C_3A or C_4AF to form $C_3A \cdot CaCl_2 \cdot 10H_2O$ or $C_3F \cdot CaCl_2 \cdot 10H_2O$ (Fig. 4).*

Therefore, NaCl has to be transformed first to $CaCl_2$ to be able to react with C_3A and C_4AF. This is the reason why the addition of Ca^{2+} ion to the system [with addition of $Ca(OH)_2$ or $CaSO_4$] increases the production rate of $C_3A \cdot CaCl_2 \cdot 10H_2O$.

Our investigations also indicated that binding of NaCl was possible even in hardened specimens. Only the chloride ions of NaCl will be bound in the hydrates while the Na^+ ions increase the alkali content of the pore water. Water content of samples treated anyhow with NaCl (added to the mixing water or after hardening) was 5 to 6 % higher than that of non-treated samples due to the strong polarisation of NaCl.

We investigated the chloride ion binding capacity of C_3S.

Fig. 4 Comparison of DTG diagrams at 28 days (Balázs-Csizmadia-Kovács, 1997)
Composition: 20g C_3A, 6g gypsum and 20.8g water
Curing: 1:without salt treatment
2: between 1 to 28 days 1 minute/day spraying of NaCl solution
3: between 1 to 28 days kept in 100g/l salt solution



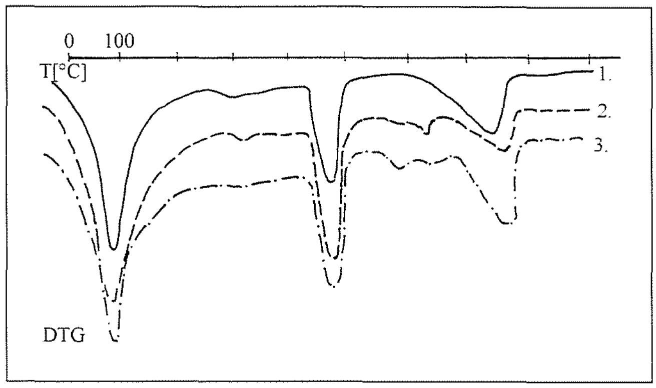


Fig. 5 DTG diagrams at 180 days (Balázs-Csizmadia-Kovács, 1997)
Composition: 10g C_3S and 3g water
Curing: 1:without salt treatment
2: mixing water included 0.2g NaCl
3: between 28 to 56 days kept in 10 m% salt solution

The preparation and the salt treatment were the same as in case of C_3A. NaCl added to the C_3S increases the hydration rate. Even X-ray diffraction analysis did not detect any sign of chloride building into the crystal structure of calcium-silicate-hydrate. We could only identify original C_3S, portlandite and calcite. *However, all samples containig or treated with NaCl showed an increase in their water content the hydration in the beginning phase of the reaction was reasonably fast in C_3S samples containing NaCl.* Even if the literature reports only about chloride ions that are bound in C-S-H with secondary forces it is possible that NaCl not only increase the rate of reaction but perhaps it influences the structure of C-S-H. Because of the temperature range of 550 to 670 °C (which is indicated in the literature as dehydration of C-S-H) we observed DTG peaks at 580 and 630 °C both for samples including NaCl from the mixing water or treated with NaCl after hardening (*Fig. 5*).

4.2 Chloride binding capacity of cements

Next question within the above research (Grant OTKA 3000) was if binding of chloride ions were possible also in cements, which are more complex than clinkers contributing to the durability of concrete. Heterogeneity of cements and variation in the quality of the clinkers make to find the answer rather difficult. Even in this case a general picture of chloride binding capacity of cements is necesary. Our specimens were subjected to salt treatment from 1 to 28 or 28 to 56 days and studied with X-ray diffractometry and derivatography.

Summary of our observations is the following (Balázs, 2001):

Both X-ray spectra and derivatograms indicated that the chloride ions of NaCl will be bound by C_3A and C_4AF in form of $C_3A \cdot CaCl_2 \cdot 10H_2O$ or $C_3F \cdot CaCl_2 \cdot 10H_2O$.

Production of $C_3A \cdot CaCl_2 \cdot 10H_2O$ was observed by all of the four investigated cements, which were treated by NaCl during hydration.

$C_3A \cdot CaCl_2 \cdot 10H_2O$ was produced either from hexagonal calcium-aluminate-hydrate [$C_3A \cdot Ca(OH)_2 \cdot 12H_2O$, $C_3A \cdot CaSO_4 \cdot 12H_2O$] by the substitution of anions or was directly produced during hydration reacting with C_3A, but it is hard to be predicted from the reaction $2NaCl + Ca(OH)_2 \rightarrow CaCl_2 + 2NaOH$. It was also observed that $C_3A \cdot CaCl_2 \cdot 10H_2O$ can be produced after 28 days (i.e. after hardening) by NaCl treatment.

Amount of $C_3A \bullet CaCl_2 \bullet 10H_2O$ is mainly determined by the content of C_3A and C_4AF of the cement considering constant concentration of NaCl solution.

We were not able to find calcium-silicate-hydrates, which include chlorides by X-ray. According to the literature, it was not yet possible to show if the chloride ions are chemically bound by primary forces. Chloride ions are possibly bound by chemisorption. Type of binding was not detectable in our tests. *Slag and fly ash did not have a considerable influence. Binding of chlorides was observed only with C_3A and C_4AF clinkers.*

Our tests clearly indicated that NaCl reacts with C_3A and C_4AF of cement increasing the durability of concrete.

Chloride binding capacity of cements with decreasing rate (highest capacity first and lowest capacity last):

CEM I 42,5 R
CEM II A-V 32,5 (20 m% fly ash content)
CEM III A (60 m% slag content)
CEM I 32,5 S (sulphate resistant PC).

4.3 Chloride ion binding capacity of calcium-aluminate-ferrites

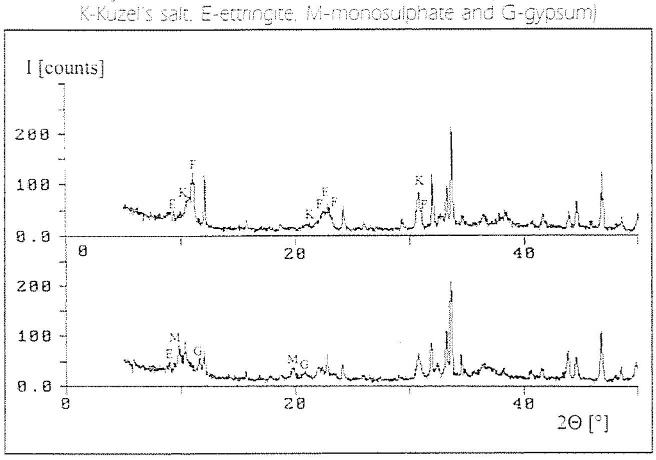
Our tests in the previous Chapter indicated that the C_3A of cements is able to bind even the chlorides of deicing salts used in winter. Sulphate resistant cement however does not contain C_3A. According to our tests with concrete sulphate resistant cement is also able to bind chlorides, which is possible only by the calcium-aluminate-ferrite clinkers (Csizmadia-Balázs-Tamás, 2000; Balázs, 2001; Csizmadia-Balázs-Tamás, 2001).

An extensive review of the literature indicated that chloride binding of calcium-aluminate-ferrite is almost a new research field. Therefore, we selected further two clinker compound: C_6A_2F and C_6AF_2 in addition to C_4AF (research Grant OTKA 019414).

Purpose of research project was then the chloride binding capacity of C_6A_2F, C_4AF and C_6AF_2 by using NaCl (as deicing salt). Specimens had 10×10×50 mm sizes. Mass ratios of gypsum to clinkers were 1/10, 2/10, 3/10, 4/10 and 5/10. Half of the specimens were kept in 100% relative humidity while the other half were subjected to salt treatment of 10% NaCl solution in the period 28 to 56 days of age.

In addition, specimens were prepared with mass ratio of gypsum to clinkers 3/10 and steam cured at 60 to 70 °C also.

Fig. 6 X-ray diffractograms of specimens including gypsum and C_6AF_2 in 1/10 mass ratio Curing: 1:without salt treatment, hydrated for 28 days
2: hydrated and salt treated between 28 to 56 days (F-Friedel's salt, K-Kuzel's salt, E-ettringite, M-monosulphate and G-gypsum)



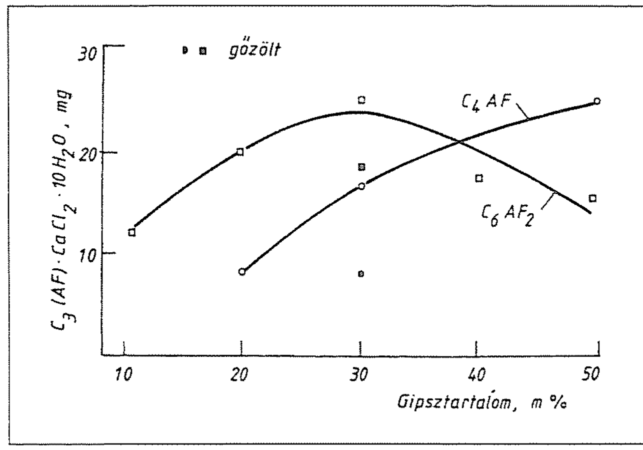


Fig. 7 Chemically bound chloride content as a function of the content of gypsum for aluminate ferrite clinkers (naturally cured and steam cured samples)

After steam curing, the specimens were treated as the non-steam cured specimens.

Splitting tensile strength was measured first at 24 hours, 28, 56, 90 and 180 days. Then the remaining pieces were pulvarised and subjected to derivatography and X-ray diffractometry.

Following results were observed:

1. During hydration of aluminoferrites (C_6A_2F, C_4AF and C_6AF_2) the same three types of hydrates [$C_3(AF)H_6$, $C_4(AF)H_{13}$, FH_3 or AH_3] were formed without gypsum. Reaction rate decreases in the following sequence: $C_6A_2F > C_4AF > C_6AF_2$.
2. *The most important results from the point of view of durability that all of these three clinkers are able to bind chlorides penetrating into the hardened concrete in form of Friedel's salt ($C_3A \bullet CaCl_2 \bullet H_{10}$) or in form of its analogue ($C_3F \bullet CaCl_2 \bullet H_{10}$). Our results obtained with X-ray diffraction also indicated that Kuzel's salt*
$C_3(AF) \bullet ½CaSO_4 \bullet ½CaCl_2 \bullet H_{11}$
is formed from Friedel'salt and monosulphate (Fig. 6). Similar hydrate was recorded by Kuzel in 1966.
3. Influence of gypsum in chloride binding is complex:
   - gypsum helps on one hand in chloride binding, preventing the reaction $C_4(AF)H_{13} \rightarrow C_3(AF)H_6$ (through blocking the formation)
   - on the other hand gypsum prevent forming of monosulphate from ettringite which prevents forming of

   $$C_3(AF) \bullet ½CaCl_2 \bullet ½CaSO_4 \bullet H_{11}.$$

   The most amount of $C_3(AF) \bullet CaCl_2 \bullet H_{10}$ hydrates were observed on the derivatograms by C_6AF_2 és C_4AF in case of 5/10 mass ratio of gypsum to clinkers and by C_6AF_2 with 3/10 mass ratio of gypsum to clinkers at 180 days (*Fig. 7*).
4. The same figure indicates that *steam cured samples which included 3/10 mass ratio of gypsum to clinkers were able to bind less chlorides than non-steam cured samples.* This was the basis for the experiments in the next Chapter.

4.4. Chloride ion binding capacity of steam cured concretes

A large portion of concrete structures in Hungary is made out of steam cured precast concrete. Purpose of our new experimental programme (OTKA T034467) is to study the chloride binding capacity of steam cured cement stones (Csizmadia-

Balázs-Tamás, 2000; Balázs, 2001;). Studies are related to the following fields:

- to test the chloride binding capacity of C_3A and C_4AF clinkers up to 180 days.
- Mass ratios of gypsum to clinkers were 1/10, 2/10, 3/10, 4/10 and 5/10.
- Ways of hardening: (a) natural hardening at 20°C and (b) steam curing at 60°C or 90°C.
- Salt treatment starts at 28 days of age. Specimens are cyclically kept in 10% NaCl solution and in 100% relativ humidity
- to study the cements mentioned in Chapter 4.2 subjected to curing and salt treatment according to the previous group of specimens.

The investigations include the determination of

- types of hydrates by X-ray diffractometry and derivatography (TG, DTG, DTA)
- the chloride content that is bound in form of Friedel's salt
- the splitting tensile strength.

5. CONCLUSIONS

Corrosion of steel reinforcement starts whenever its passivating layer dissolves. Chloride ions induce electrochemical corrosion in form of pittig corrosion.

Chlorides may be present in concrete from its constituents (like admixtures), from de-icing salts, from seawater or from PVC during fire.

Chemically bound chlorides do not induce corrosion. It is also known since hundred years that C_3A clinkers bind $CaCl_2$ in form of Friedel's salt. However, NaCl of deicing salts can be only bound by the cement stones if any hydrates first dissolves.

Our tests indicated that $CaCl_2$ and NaCl will be bound by C_3A and C_4AF clinkers in form of Friedel's salt ($C_3A \bullet CaCl_2 \bullet 10H_2O$, or. $C_3F \bullet CaCl_2 \bullet 10H_2O$) independently if $CaCl_2$ and NaCl get into the concrete during mixing or after hardening. We also observed that chloride ions are not bound chemically with C_3S clinkers.

Chloride binding is produced in cements only with C_3A and C_4AF clinkers. Addition of fly ash or slag to cements do not influence chloride binding capacity except if they reduce the pH of cement stone.

Our test results indicated that C_6A_2F, C_4AF and C_6AF_2 clinkers bind NaCl even if they penetrate after hardening, however, the type of chloride binding depends on the amount of gypsum. Steam curing seemed to reduce chloride binding.

Purpose of our present tests with C_3A and C_4AF clinkers and four various cements is to study the chloride binding mechanism of steam cured concretes.

6. LIST OF NOTATIONS

A	=	Al_2O_3
C	=	CaO
C_3A	=	$3CaO \bullet Al_2O_3$
C_3S	=	$3CaO \bullet SiO_2$
C_4AF	=	$4CaO \bullet Al_2O_3 \bullet Fe_2O_3$
C_6A_2F	=	$6CaO \bullet 2Al_2O_3 \bullet Fe_2O_3$
C_6AF_2	=	$6CaO \bullet Al_2O_3 \bullet 2Fe_2O_3$
CH	=	$Ca(OH)_2$
Cs	=	$CaSO_4$
H	=	H_2O

7. ACKNOWLEDGEMENTS

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SELF COMPACTING CONCRETE – SOME HUNGARIAN EXPERIENCES


Dr. István Zsigovics – Prof. György L. Balázs

The new, high efficiency admixtures open new prospects in the concrete technology. The first applications of self compacting concrete (SCC) in Hungary were created by difficult circumstances in concreting. In these cases SCC was the optimal solution and could be eliminated the mistrust and the disadvantage of high concrete price.

We took on this challenge and established large scale application of SCC, and economically adaptable SCC development. In this article we intended to report these results.

Keywords: Self Compacting Concrete (SCC), concrete technology, limestone filler, consistency, compressive strength

1. INTRODUCTION

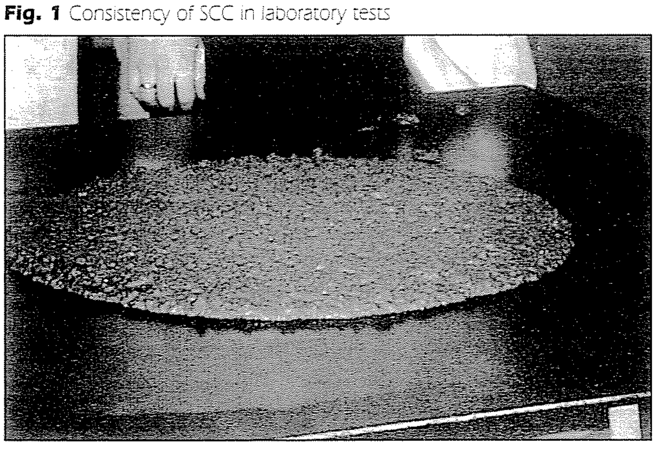
Self compacting concrete (SCC) is a new challenge and real possibility for the improvement of concrete technology, however, it needs further research. Introduction of newest superplasticizers on policarboxylatether bases (e. g. Sika Viscocrete, Mapei Dynamon SR3 or Stabiment FM 38/40/210/352) allowed the first Hungarian applications.

Tests were started at the Department of Construction Materials and Engineering Geology Budapest University of Technology and Economics in 1999 with the purpose of industrial applications of SCC *(Fig. 1)*. Considering the Hungarian possibilities, the fine filler content was solved by limestone.

An overview on the State-of-Art can be obtained from References Okamura and Uomoto (1998) as well as from König, Holschermaher, Dehn (2001). In the world, various types of self compacting concretes are produced: normal strength concrete, high strength concrete, fibre reinforced concrete, sandrich concrete, exposed concrete, ect. (König, Holschermaher, Dehn 2001). Up to now the bases of the design of the SCC has been developed (Kolczyk, Rings 2001). Every mix design emphasises the adjustment of the concrete compositions regarding to the local availabilities. Therefore, every mix has to be checked by tests. The need for trial mixes is even more pronounced now than it was earlier.

In Chapter 2 some practical applications are detailed from the last couple of years in Hungary and in Chapters 3 experimental results are shown for the influence of limestone filler on the consistency and compressive strength.

Fig. 1 Consistency of SCC in laboratory tests



2. INDUSTRIAL APPLICATIONS

2.1 Construction of MOM PARK

Casting of concrete was allowed only by night without using vibrators. It automatically defined the reasonable application of SCC.

We developed the following concrete mix with trial tests in the laboratory.

Applied concrete mix:

Cement	350 kg/m^3
CEM II/A-S 32.5 R	
Water	165 kg/m^3
Limestone filler	160 kg/m^3
0/4	750 kg/m^3
4/8	300 kg/m^3
8/16	660 kg/m^3
Sika Viscocrete	7 kg/m^3

Slump-flow: 770/780/760/720 mm (at 5/30/60/90 minutes)

The concrete mix that was found to be appropriate in laboratory circumstances has shown some bleed and segregation because of 10 to 15 l/m^3 overdosed water by the site application.

Compressive strength:

in 7 days	52 N/mm^2
in 14 days	57 N/mm^2
in 28 days	61 N/mm^2.

2.2 Parking House in KÉSZ MESTER Shopping Center

Columns of 300×400 mm cross-section and 2.5 or 4 m highes contained extremely congested reinforcement. Casting by normal concrete was so unsatisfactory that the first two columns had to to be demolished because appropriate repair was not possible. The solution was the application of SCC.

Applied concrete mix:

Cement	350 kg/m^3
CEM II/A-S 32.5 R	
Water	180 kg/m^3

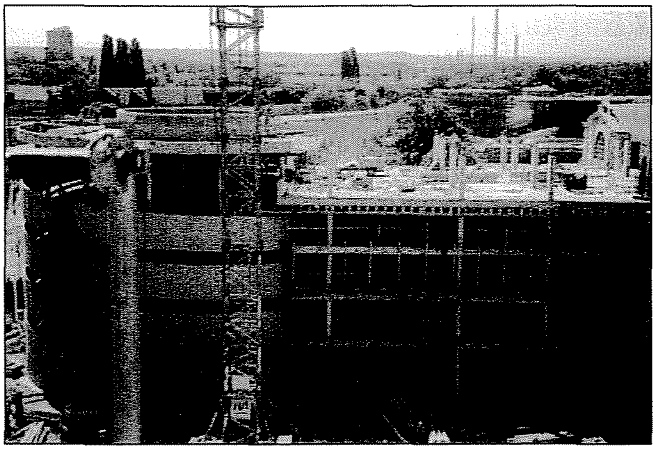


Fig. 2 Columns of Parking House in KÉSZ MESTER Shopping Center made of SCC

Limestone filler	100 kg/m³
0/4	905 kg/m³
4/8	225 kg/m³
8/16	610 kg/m³
Sika Viscocrete	5,6 kg/m³

Slump-flow: 700 to 750 mm was tested in site.

The surface of the columns made of SCC were homogeneous, there was no segregation. The building contractor was satisfied with the results *(Fig. 2)* (Berecz, Székács 2000).

2.3 Filmhouse Budapest

Casting of a large auditorium stair slab was stopped unforeseen. The new and old concrete layers had to work together appropriately, without an adhesive layer. Therefore, the new concrete layer was constructed of SCC *(Fig. 3)*.

Fig. 3 Concreting of the slab in hot summer (Filmhouse Budapest)

Applied concrete mix:

Cement	360 kg/m³
CEM II/B-S 32.5 N	
Water	140 kg/m³
Limestone filler	190 kg/m³
0/4	790 kg/m³
4/8	260 kg/m³
8/16	700 kg/m³
Sika Viscocrete	7,2 kg/m³

Slump-flow: 720 to 760 mm was tested in site.

The air temperature was 30 to 36 °C but the concrete was workable even after 2.5 hours not only after 2 hours. In spite of the high air temperature retarding admixture was not used.

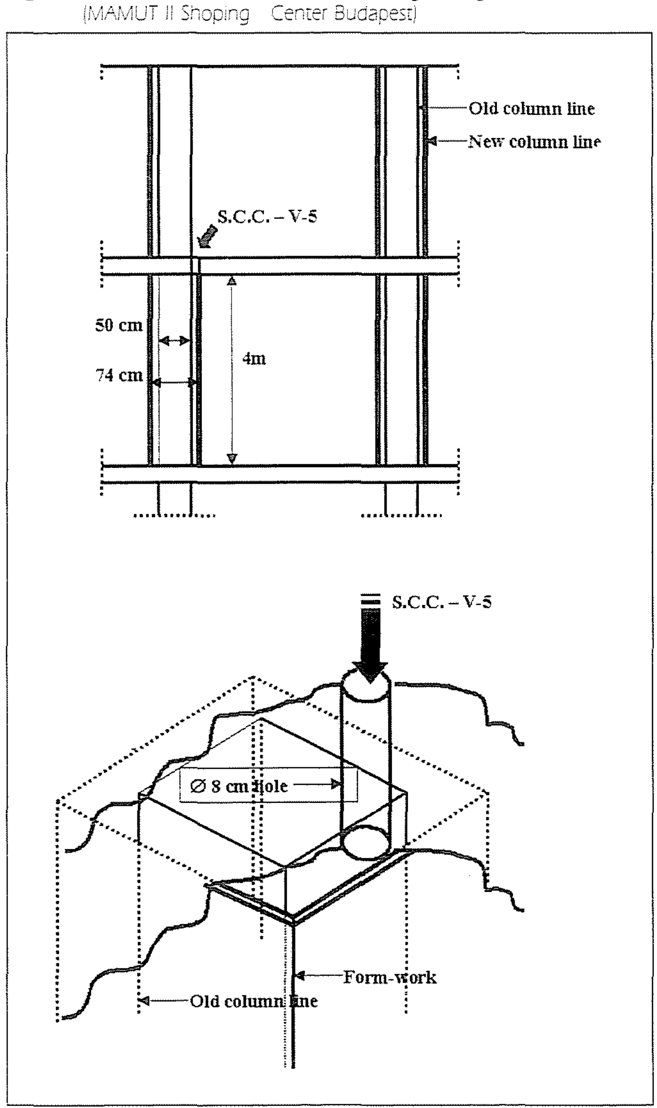
2.4 MAMUT II Shopping Center Budapest

4 to 5.6 m high columns of a shopping center had to be strengthened by an additional 100 mm concrete cover. Owing to the thin concrete layer application of SCC seemed to be reasonable. Casting of SCC concrete was carried out through Ø100 mm holes in the top slab *(Fig. 4)*.

Applied concrete mix:

Cement	350 kg/m³
CEM I 42.5 N	
Water	155 kg/m³
Limestone filler	150 kg/m³

Fig. 4 Sketch of the cross section of the strengthening of the column (MAMUT II Shoping Center Budapest)



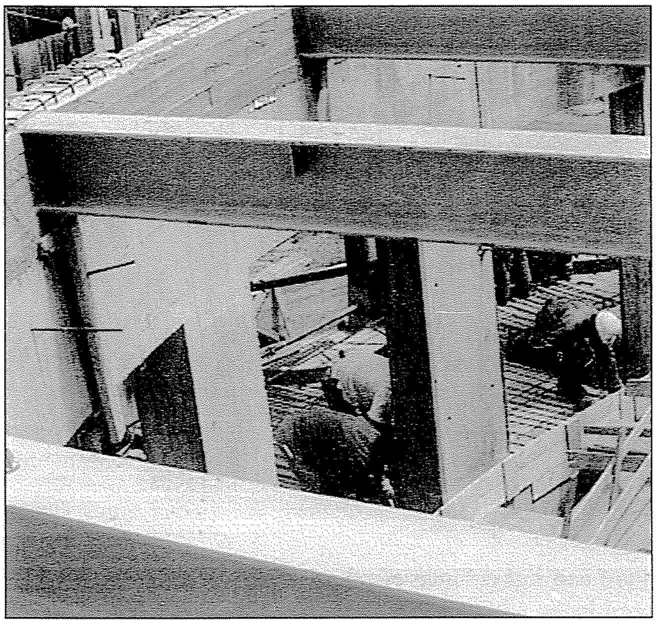


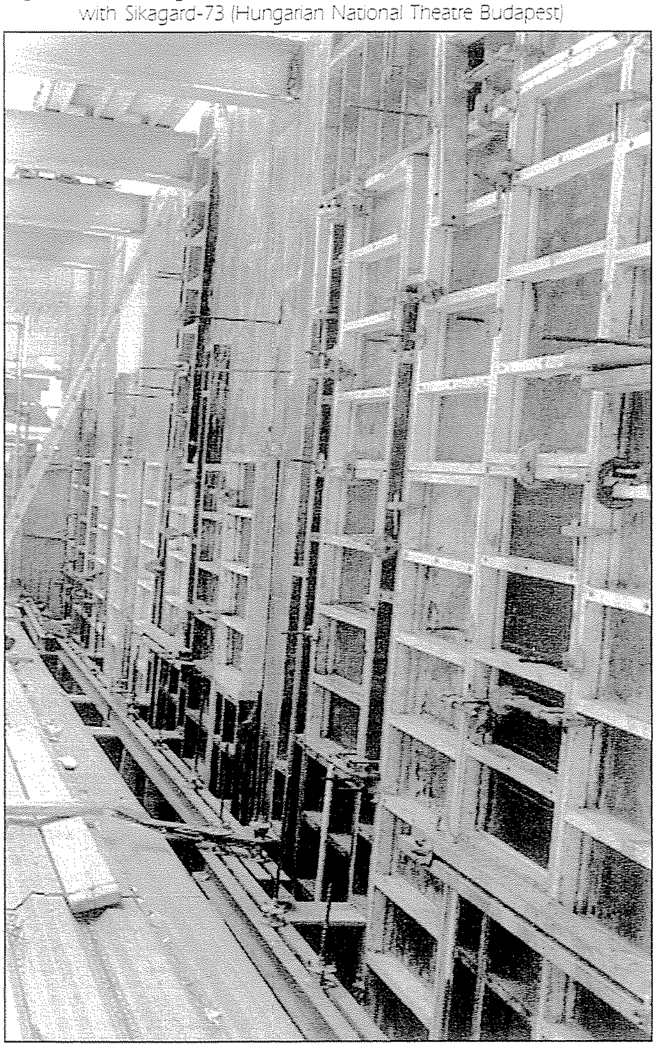
Fig. 5 Auditorium wall made of SCC (Hungarian National Theatre Budapest)

0/4	615 kg/m³
4/8	1140 kg/m³
Sika Viscocrete	6 kg/m³

Slump-flow: 720 to 750 mm was tested in the mixing plant.

The strengthening was successful. SCC properly filled out even the top of the column well. There was no remarkable postcompaction of SCC.

Fig. 6 Demoulding of wallbeam and the surface of the concrete cured with Sikagard-73 (Hungarian National Theatre Budapest)



2.5 Hungarian National Theatre Budapest

In May 2001 SCC was used for the semicircular walls of 400 mm thickness and 8 to 10 m height on the upper floor during the construction of the new Hungarian National Theatre Hungary. Then two rigid framed reinforced concrete walls of 24 m span and 5 m height were constructed above the auditorium.

Applied concrete mix:

Cement	350 kg/m³
CEM II/A-S 42.5 N	
Water	150 kg/m³
Limestone filler	150 kg/m³
0/4	919 kg/m³
4/8	230 kg/m³
8/16	618 kg/m³
Sika Viscocrete	6 kg/m³

Slump-flow: 680 to 750 mm was tested in site.

Walls appeared to be in good quality in spite of the big dropping height *(Fig. 5)*. Wallbeams appeared to be in good quality as well. According to the structural engineer cracks should have turned up, but they did not appear. Wallbeams were demoulded at the age of 2 days. Curing was done with Sikagard-73 successfully *(Fig. 6)*.

2.6 PAKS MVDS Concreting of the Charge Face Structure footwalk

MVDS = Modular Vault Dry Storage

In August 2002 SCC was used for the casting of the heavily reinforced footwalk charge face structure of the nuclear power station in Paks, Hungary. Concrete had to be poured under the steel sheet. The level of the concrete was checked in drilled holes and found appropriate.

Applied concrete mix:

Cement	350 kg/m³
CEM I 32.5 S	
Water	150 kg/m³
Limestone filler	190 kg/m³
0/4	917 kg/m³
4/8	208 kg/m³
8/16	605 kg/m³
Sika Viscocrete	6 kg/m³

Slump-flow: 650 to 700 mm was tested in site.

Because of the short transporting time SCC was mixed for 2 minutes in the mixing machine after adding the superplasticizer. The concrete mix is sensitive to the preciseness of the waterdosage. According to our experience an additional 20 to 30 kg/m³ sandcontent requires approximately 10 l/m³ water additionally. Therefore, the SCC technology needs more control than the technology of normal concrete.

2.7 Pipe Works in Csepel

In Pipe Works of Csepel, Hungary, columns had to be strengthened which have been already modified by additional corbels for cranes. Therefore, the present strengthening layer varied between 60 to 200 mm. SCC was applied instead of shotcrete.

Applied concrete mix:

Cement	350 kg/m³
CEM II/B S 32.5 R	
Water	205 kg/m³

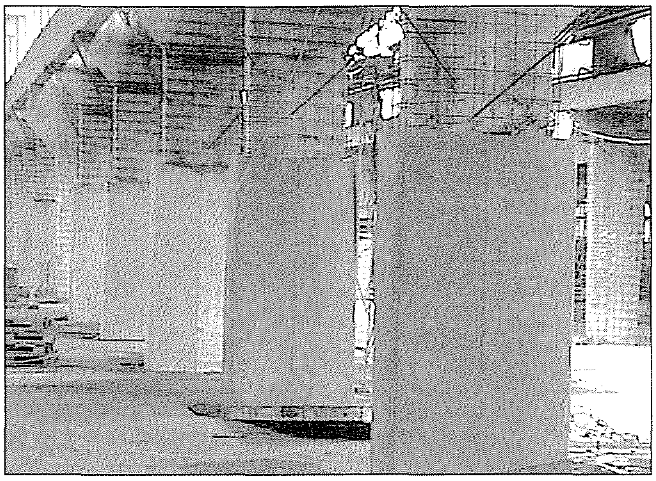


Fig. 7 Strengthening of the columns (Pipe Works Budapest – Csepel)

Limestone filler	185 kg/m^3
0/4	760 kg/m^3
4/8	390 kg/m^3
8/16	390 kg/m^3
Mapefluid X 524 SCC	5.3 kg/m^3
Antigelo S	3.5 kg/m^3

Initial bleeding and segregation of SCC by the site application was solved by additional 50 kg/m^3 limestone filler and 0.5 kg/m^3 Viscofluid SCC stabilizing admixture. Final surface appeared to be in good quality *(Fig. 7)*.

2.8 Strengthening of a reinforced concrete girder highway bridge

During strengthening SCC was applied instead of shotcrete owing to the dense reinforcement in a girder bridge Galgamácsa, Hungary.

Applied concrete mix:

Cement CEM I 42.5 N	340 kg/m^3
Water	150 kg/m^3
Limestone filler	195 kg/m^3
0/4	864 kg/m^3
4/8	830 kg/m^3
Mapefluid X 404	5.5 kg/m^3
Viscofluid SCC	0.5 kg/m^3

Slump-flow: 750 mm was tested in mixing plant,
700 mm was tested in site.

Casting of SCC was carried out through the deck slab. Concrete arrived even to the furthermost points of the formwork. Surface seemed like of precast concrete *(Fig. 8)*.

Fig. 8 Surface of the bridge girder after casting through the deck (Galgamácsa)

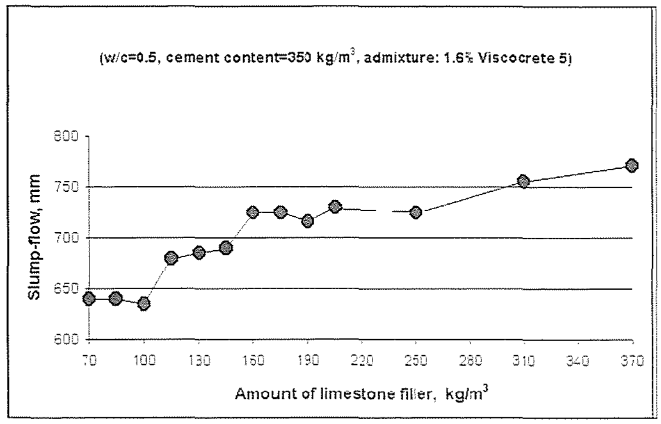


Fig. 9 Influence of the amount of limestone filer on the slump-flow of SCC

2.9 NN House in Dózsa György street

Stair walls were made of 200 to 400 mm thick exposed SCC concrete.

Applied concrete mix:

Cement CEM II/B S 32.5 R	390 kg/m^3
Water	190 kg/m^3
Limestone filler	170 kg/m^3
0/4	790 kg/m^3
4/8	395 kg/m^3
8/16	395 kg/m^3
Dynamon SR3	4.8 kg/m^3

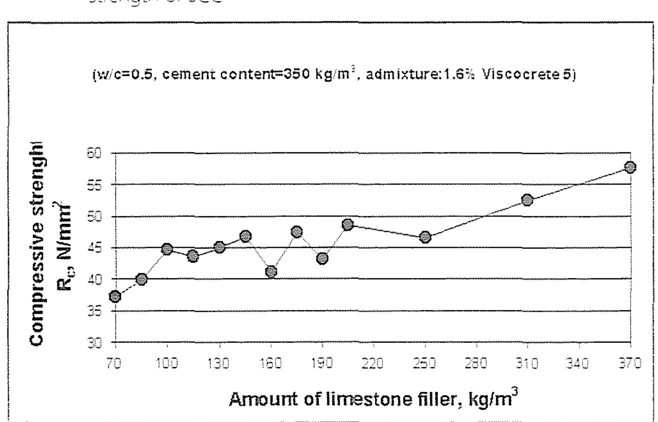
Slump-flow: 700 to 750 mm was tested in site.

During casting the tube of the concrete container was pushed into the previously cast concrete layer. In this way there was no visible sign between the casting layers. The surface prepared without air bubbles.

3. LABORATORY TESTS

Content of fine filler in SCC is a major issue. Our experiments with Sika Viscocrete indicated that an increase in the amount of limestone filler increased the slump-flow of the fresh concrete (Zsigovics, Berecz 1999). Changing of the concrete mix influences the behaviour of the fresh concrete (Rings, Kolczyk, Löschnig 2001). According to the previous experiences for normal concrete an increase in filler content requires additional water to achieve the same consistency and the air content increases as well. Therefore, we intended to study the influence of limestone filler in SCC (Zsigovics, 2002).

Fig. 10 Influence of the amount of limestone filler on the compressive strength of SCC



Amount of limestone filler was varied between 70 kg/m^3 and 370 kg/m^3 while cement content was 350 kg/m^3, w/c=0,5 and superplasticizer Sika Viscocrete 1,6% related to the amount of cement. Measured consistency and compressive strength are presented in *Figs. 9* and *10* in the function of the amount of limestone filler.

Results indicated an increase both in the slump-flow value as well as in the compressive strength by increasing the amount of limestone filler.

The optimal content of limestone filler in our tests was 200 to 220 kg/m^3. By higher amounts of limestone filler the relative increase in consistency and compressive strength was lower.

The consistency increased by 14% in case of 70 to 200 kg/m^3 limestone filler, while the consistency increased only by 5,4% in case of 200 to 370 kg/m^3 amount of limestone filler.

Compressive strength increased by 30% in case of 70 to 200 kg/m^3 limestone filler, while it increased by 19% in case of 200 to 370 kg/m^3.

Practical application of these test results leads to a technologically less sensitive material which is more stable and flows like honey.

4. CONCLUSIONS

Self compacting concrete (SCC) is nowadays the highest challenge in concrete technology. Most important issues are its optimal grading and water content.

Technological investigations on self compacting concrete enable us to understand its behaviour and applicability.

Present paper summarizes our test results on the influence of various amounts of limestone filler to the consistency and to the compressive strength.

According to our test results the optimal amount of limestone filler is between 160 to 220 kg/m^3 for the consistency as well as for the compressive strength.

Some examples of successful application of self compacting concrete in Hungary is presented from the last couple of years.

5. ACKNOWLEDGEMENT

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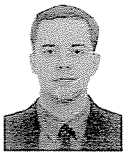
STRUCTURAL BEHAVIOUR OF STEEL FIBRE REINFORCED CONCRETE




Imre Kovács – Prof. György L. Balázs

The application of steel and polymeric fibres is continuously increasing in Hungary. Results of an experimental study on 18 reinforced concrete (RC) beams indicate that steel fibres do not only increase shear capacity, but also provide substantial post-peak resistance and ductility in conventionally reinforced beams as well as in prestressed pretensioned concrete beams. Focusing on the modelling of fibre reinforced concrete, a simple rheological model is presented here which takes into consideration a plastic matrix to fibre coupling.

Keywords: fibre, reinforced concrete, FRE, fibre content, failure mode, cracking, modelling

1. INTRODUCTION

Steel fibre reinforced concrete (SFRC) improves properties such as toughness (Naaman 1992; Naaman-Reinhardt, 1995), ductility, fatigue and impact resistance (Kormeling-Reinhardt-Shah, 1978). SFRC is particularly used in road construction, industrial floor slabs, airfield runways, pavements and refractory materials amongst other applications (Romualdi-Mandel, 1964; Dombi, 1993). Several publications cover basic research on SFRC as a special concrete with characteristics different from those of conventional concrete (Romualdi-Mandel, 1964; Reinhardt-Naaman, 1992; Naaman-Reinhardt, 1995).

However, few results are available concerning the behaviour of SFRC in structural reinforced concrete members (Kormeling-Reinhardt-Shah, 1978) and partially and axially prestressed members (Wafa-Hasnat-Tarabolsi, 1992). Tests have though been carried out to investigate the punching shear resistance of prestressed SFRC flat slabs (Falkner-Kubat-Droese, 1994). As previous tests indicated, steel fibre reinforcement is not effective in improving the moment capacity of reinforced concrete members (Sanat-Niyogi-Dwarakanathan, 1995). However, fibres may have a significant effect on the shear resistance of reinforced concrete beams (El-Niema, 1991; Narayanan-Darwish, 1978; Tan-Murugappan-Paramasivam, 1993), and slabs (Falkner-Kubat-Droese, 1994) (punching shear). Also, the application of fibres may reduce the amount of stirrups and the congestion of reinforcement in high shear regions. Fibres do not only increase shear capacity but also provide substantial post-peak resistance and ductility in shear.

The application of fibre reinforced composite materials is growing rapidly worldwide due to their high performances. Nowadays, fibre reinforced composites are widely used from civil engineering (Fibre Reinforced Concrete = FRC, Fibre Reinforced Plastics = FRP) to the space industry (FRP). The improvement therefore of the properties of materials applying fibre reinforcement technology is not a new idea. Historically, short fibres have been used to reinforce brittle matrices since ancient times. Fibres are effective in improving tensile strength (FRP), toughness, ductility and impact resistance of the matrix (FRC). But, despite the importance of composite materials in the modern technologies, only a few engineering models have been developed. So far, only a limited quantity of results are available concerning the modelling of steel fibre reinforced concrete as a structural material (RILEM, 1978; ACI; 1978; Dulácska, 1994; Dulácska, 1996). Generally empirical models have been used based on experimental investigations on beams in three or four point loading (RILEM, 1978), (ACI, 1978). Whilst, leading fibre producers such as Bekaert have developed industrial recommendations for Dramix type hooked-end steel fibres (Bekaert, 1994), there is still due to the lack of proper design methods, a general model consideration which is required independently of the fibre type used. In the light of this, a simple rheological device has been developed for the modelling of fibre reinforced concrete.

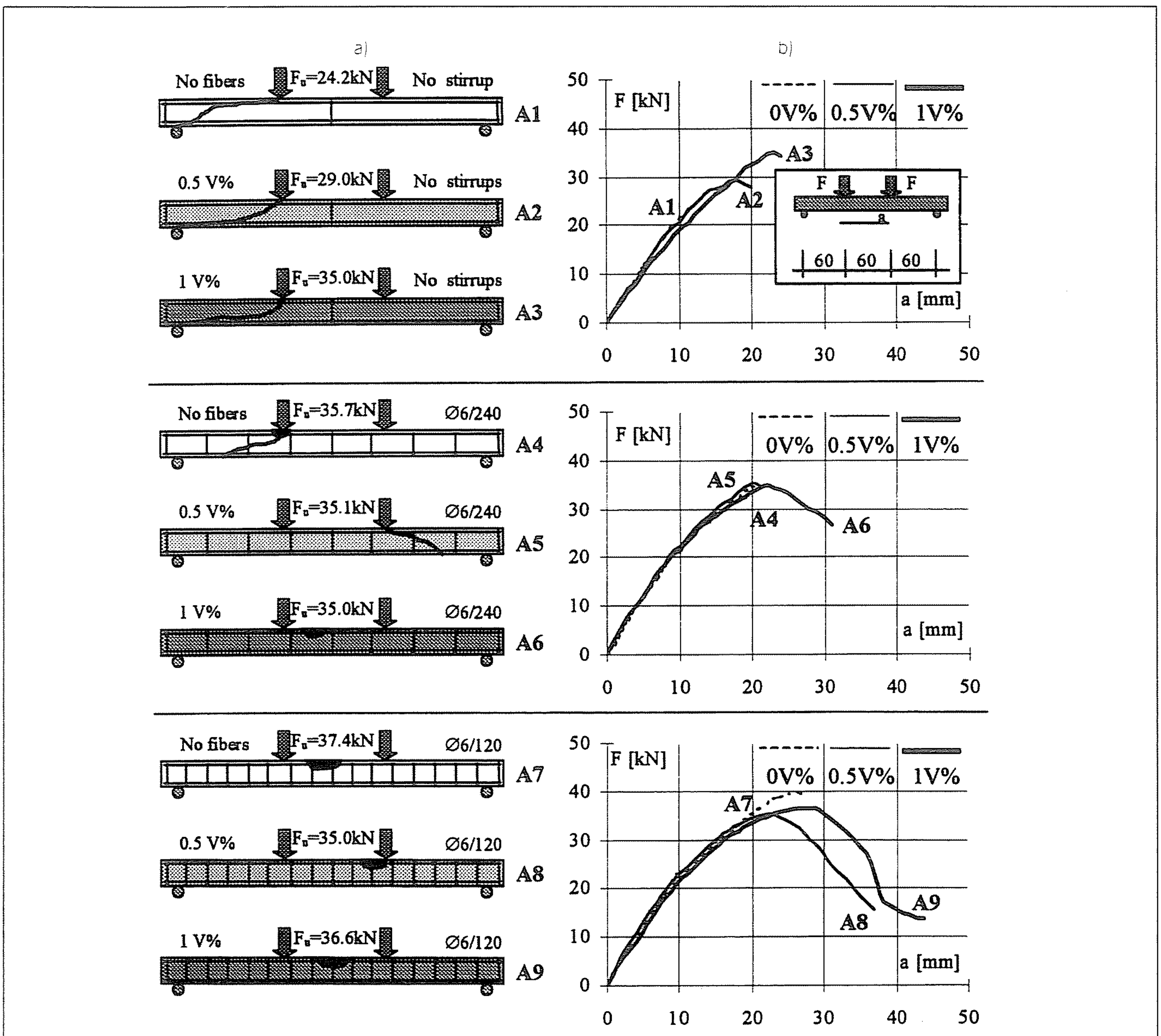
Table 1 Experimental variables for beam tests.

	Prestressed concrete beams	Reinforced concrete beams	
Fibre content	Series P hooked-end fibres	Series A hooked-end fibres	Series B crimped fibres
		Stirrup reinforcement	
0 V%	0	0	0
0.5 V%	0	0	0
1.0 V%	0	0	0
0 V%	Dramix ZC30/.5	∅6/240	∅4/240
0.5 V%	hooked-end fibres	∅6/240	∅4/240
1.0 V%		∅6/240	∅4/240
0 V%	D&D~30/.5	∅6/120	∅4/120
0.5 V%	crimped fibres	∅6/120	∅4/120
1.0 V%		∅6/120	∅4/120

Type reinforcement	Sign of specimen	A_s	A_s'	Stirrups content V%	Fibre type	Fibre load kN	Failure Mode S: shear B: bending	Failure strength N/mm²	Cube strength N/mm²
	P1				0		8.20	S	37.58
	P2				0.5		8.85	S+B	39.58
Prestressed	P3	Ø12.5	–	–	1.0	Dramix ZC 30/.5	9.45	B	38.55
	P4				0			S	42.70
	P5				0.5			S	43.60
	P6				1.0			S	41.60
	A1			–	0		24.2	S	37.58
	A2				0.5		29.0	S	39.85
	A3				1.0		35.0	S	38.55
	A4			Ø6/240	0		35.7	S	37.58
Non-prestressed	A5	2Ø16	2Ø6		0.5	Dramix ZC 30/.5	35.1	B	39.85
	A6				1.0		35.0	B	38.55
	A7			Ø6/120	0		37.4	B	37.58
	A8				0.5		35.0	B	39.85
	A9				1.0		36.6	B	38.55
	B1			–	0		21.6	S	42.70
	B2				0.5		33.6	S	48.80
	B3				1.0		44.7	S+B	47.16
	B4			Ø4/240	0		27.5	S	42.70
Non-prestressed	B5	2Ø16	2Ø6		0.5	D&D 30/.5	44.3	S	48.80
	B6				1.0		45.7	B	47.16
	B7			Ø4/120	0		35.2	S	42.70
	B8				0.5		46.6	S	48.80
	B9				1.0		45.0	B	47.16

Table 2 Failure loads and failure modes of RC and PC beams

Fig 1 Four point bending tests on SFRC beams (single test results). Fibres: DRAMIX ZC30/.5 hooked-end fibres. (Series A) Failure loads and failure modes b) Load vs. mid-span deflections



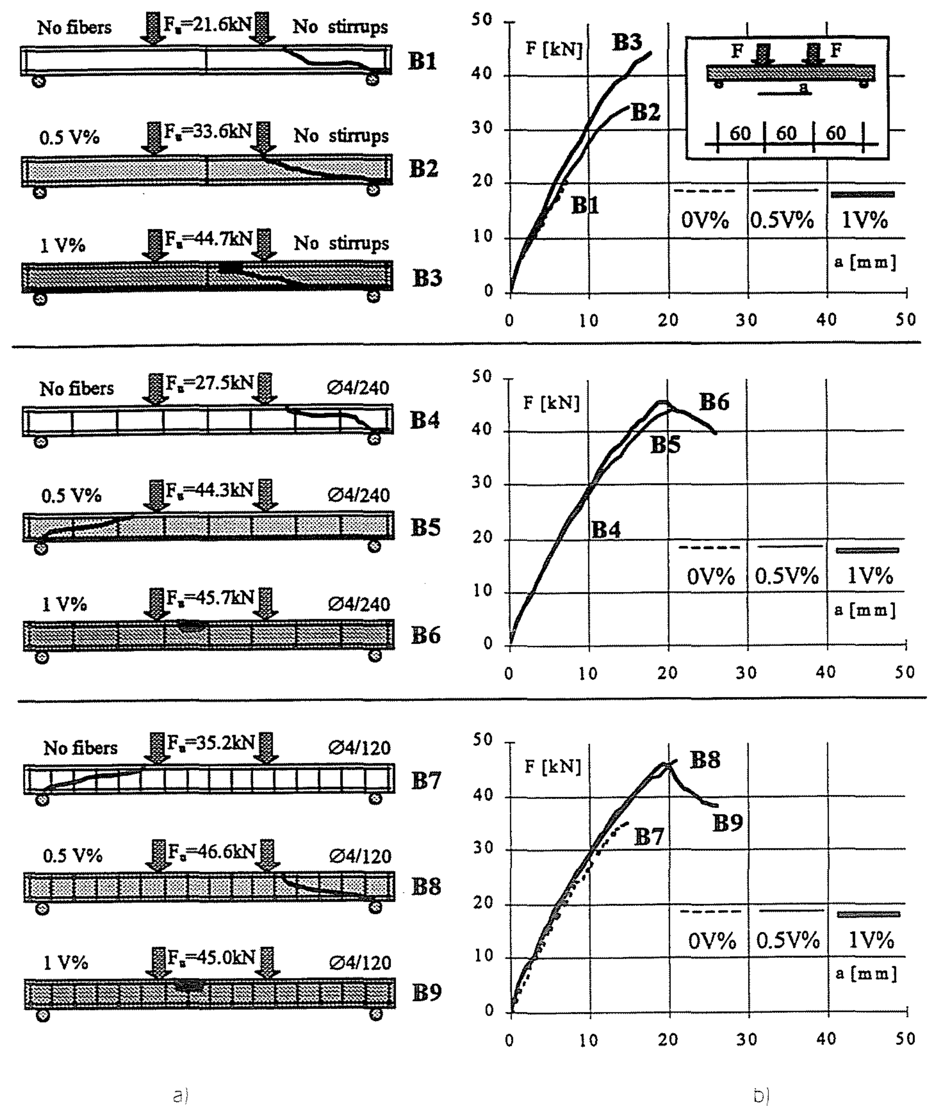


Fig 2 Four point bending tests on SFRC beams (single test results). Fibres: D&D~30/.5 crimped fibres. (Series B)
a) Failure loads and failure modes b) Load vs. mid-span deflections

2. BEHAVIOUR OF RC BEAMS IN SHEAR AND BENDING

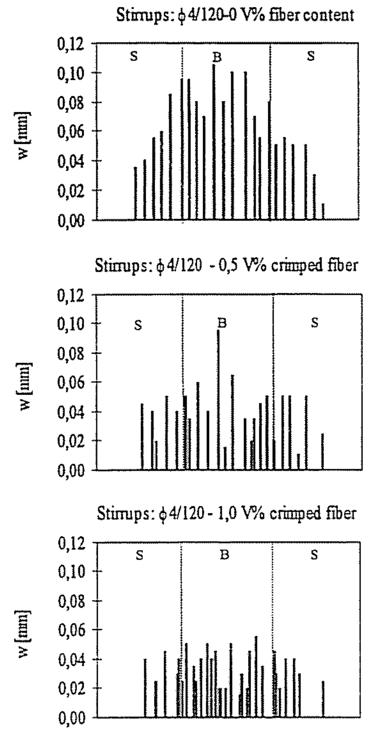
2.1 Experimental variables

A total number of 18 fibre reinforced concrete beams of 2 m length (span 1.8 m) with a cross section of 100×150 mm were tested in four-point bending. Longitudinal reinforcement was the same for each beam, 2Ø16 bars in the tension zone and 2Ø6 bars in the compression zone (*Fig. 1*). The load was applied with two LUCAS hydraulic jacks with 100 kN capacity, both at the third points of the span. In each load step the crack pattern, the crack propagation and the crack widths were detected at the level of the main longitudinal reinforcement. Load versus mid-span deflection relationships were also recorded. The experimental parameters of the beam tests are summarised in *Table 1*.

2.2 Failure loads and failure modes

The failure loads and modes as well as cube strength of RC beams are summarised in *Table 2*. As *Fig. 1* illustrates, a significant effect of the fibre reinforcement could be observed on the shear resistance of fibre reinforced concrete beams. Due to increasing fibre content, the increase in failure load was found independently of the fibre type used (A1...A3, B1...B3), particularly in the case where no stirrup reinforcement was placed in the beams. On the other hand, the failure mode changed from shear (B1 and B2) to simultaneous shear + bending failure for beams containing 1.0 V% crimped fibres and no stirrup (B3 *Fig. 2*). Considering stirrup reinforcement of Ø4/240 mm, the shear capacity increased when applying crimped fibre (B4...B6). In this case, higher fibre content led to the change in failure mode, from shear failure to bending failure.

Due to the higher shear reinforcement ratio of Ø6/120 mm, and applying hooked-end type fibres, bending failure occurred independently of the applied fibre volume (A7...A8). However, moment capacity was not influenced by the fibre content. All beams in this group failed at about the same load level. In order to have clear fibre effect on shear resistance, Ø4 mm stirrup diameter was used for beams with crimped fibres (B series). In beams having Ø4/120 mm shear reinforcement and crimped fibres, an increase in shear capacity was found with a change in failure mode from shear failure to bending failure. 0.5 V% crimped fibres showed higher shear capacity (B8: 46.6 kN) than



	Cracks S+B	Cracks B	Cracks S
No.	22	9	13
Σa [mm]	1.45	0.74	0.71
a_m [mm]	0.066	0.082	0.055
s_a [mm]	82	67	92

	Cracks S+B	Cracks B	Cracks S
No.	23	9	14
Σa [mm]	0.945	0.445	0.500
a_m [mm]	0.041	0.049	0.036
s_a [mm]	78	67	86

	Cracks S+B	Cracks B	Cracks S
No.	30	17	13
Σa [mm]	1.030	0.585	0.445
a_m [mm]	0.034	0.034	0.034
s_a [mm]	60	35	92

Fig 3 Crack distribution in beams having crimped fibres and Ø4/120 shear reinforcement measured at F = 20 kN jacking force (S+B: shear and bending region = whole beam, B: bending (middle) portion, S: shear (outer) portion). The horizontal axis of the diagrams give the position of the cracks

beams prepared with only stirrup reinforcement (B7: 35.2 kN). Moreover, 1.0 V% crimped fibres yields bending failure since the increasing fibre content was able to carry the increasing shear forces in the member (B9).

2.3 Load versus deflection relationships

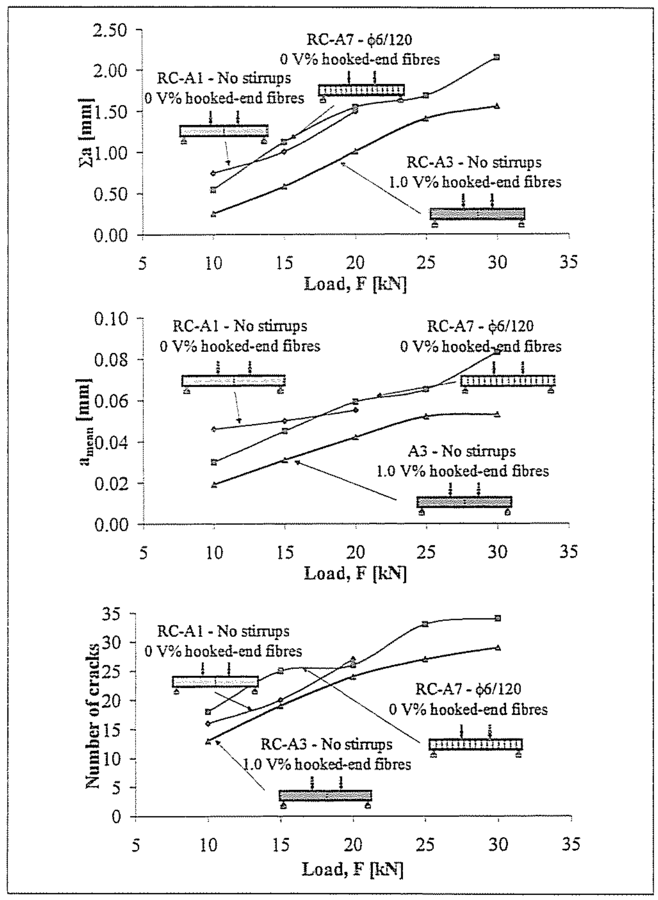
Load-deflection relationships are also indicated in *Fig. 1* and *2* for every specimen. The addition of fibres considerably increased the mid-span deflection at failure. The curves indicate higher elastic stiffness for beams applying crimped fibres, which may be attributed to the higher concrete strength. However, ultimate mid-span deflections were found to be higher for beams made with hooked-end steel fibres than that of beams prepared with crimped fibres.

2.4 Crack propagation

Steel fibre reinforcement is commonly used for increasing toughness and energy absorption in concrete as well as to distribute crack widths. In our tests full crack mapping was done during loading in each load step. *Fig. 3* shows the crack pattern of the beam cast with hooked-end fibres and Ø6/120 mm stirrup reinforcement (A7…A9) under service load of 20 kN jacking force. On the right hand side of *Fig. 3*, tables summarise the number of cracks, sum and average crack widths, and mean crack spacing. The results clearly indicate that fibres decrease the crack width. Fibres were more effective in the middle portion of the beam where (B), the bending moment is constant, than in the shear span (S).

Load vs. sum of crack width, load vs. average crack width and load vs. number of crack relationships were also developed for all beams. *Fig. 4* and *5* respectively summarise these relationships for beams A1, A3 and A7 and for beams B1, B3 and B7. As the curves indicate, 1.0 V% fibre content resulted in a better cracking behaviour

Fig 4 Load vs. sum of crack width, load vs. mean crack width and load vs. number of crack relationships for beams A1, A3 and A7



independently of the fibre type and stirrup reinforcement used. Both the sum of crack widths and the average crack width were found to be lower in the case of fibre reinforcement than that of in case of no fibre and no stirrup at all (A1 and B1). Similarly to A1 and B1, when Ø4/120 or Ø6/120 stirrup reinforcement were applied (A7 and B) sum of crack widths and average crack width also decreased.

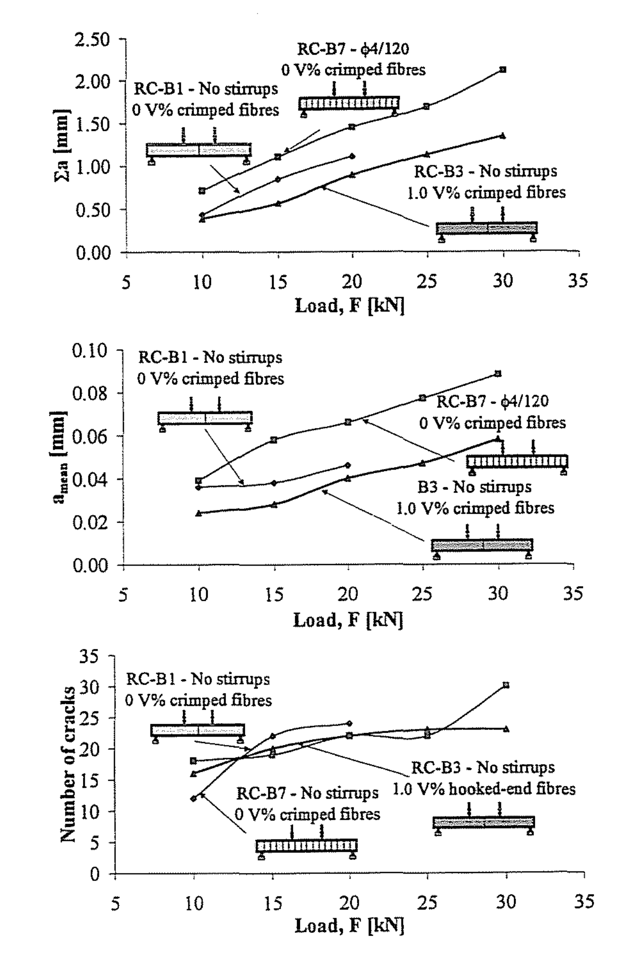


Fig 5 Load vs. sum of crack width, load vs. mean crack width and load vs. number of crack relationships for beams B1, B3 and B7

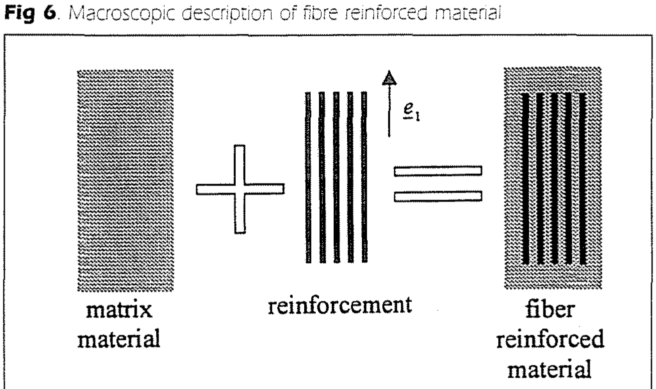
3. MODELLING OF STEEL FIBRE REINFORCED CONCRETE

In the light of the motivation to develop a macroscopic material model for steel fibre reinforced concrete, let us consider a continuous medium reinforced by a system of uniformly distributed reinforcement parallel to a given direction characterised by a unit vector $\underline{e}_1$ as shown in *Fig. 6* (Kovács, 1998; Kovács, 1998). This composite material is understood at the macroscopic scale as the superposition of the two macroscopic composite constituents, namely the matrix material and the reinforcement (unidirectional fibre system).

Govern the composite matrix behaviour an elastic-brittle material law represented by a spring of rigidity C_m, as the elastic modulus of the composite matrix, with a fragile crack device, strength f_t, in the uniaxial stress-state shown in *Fig. 7*. Meanwhile, the behaviour of the unidirectional fibre system, represented by an elastic stiffness C_f, as the elastic modulus of the reinforcement, in series with a frictional element, strength f_y, is governed by an elastic-perfectly plastic material law (*Fig. 7*).

Furthermore, coupling the behaviour of the composite material from the behaviour of the composite constituents is shown in *Fig. 8*, the two parallel sub-devices are coupled by an elas-

Fig 6. Macroscopic description of fibre reinforced material



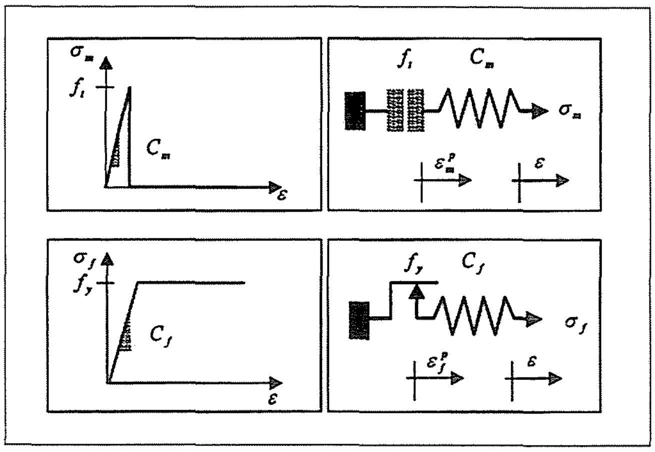
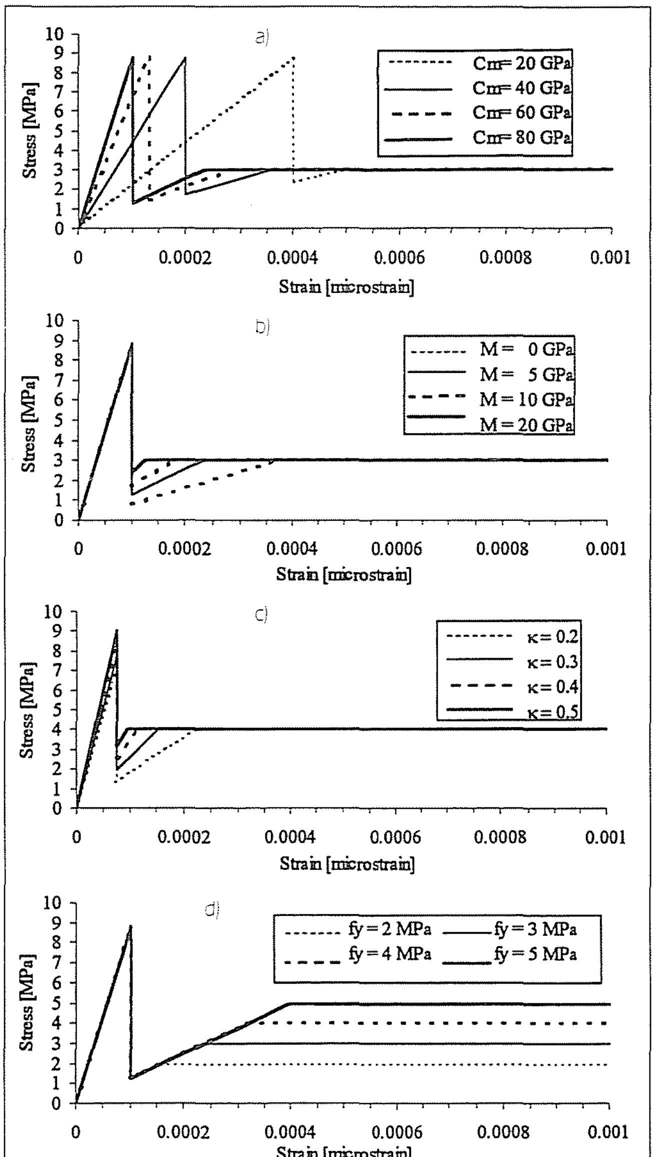


Fig 7 Material models for the composite constituents

Fig 8 Effect of composite constituents and their material parameters (Kovács 1998)
a) effect of C_m b) effect of M c) effect of $k = C_f / C_m$ d) effect of f_y



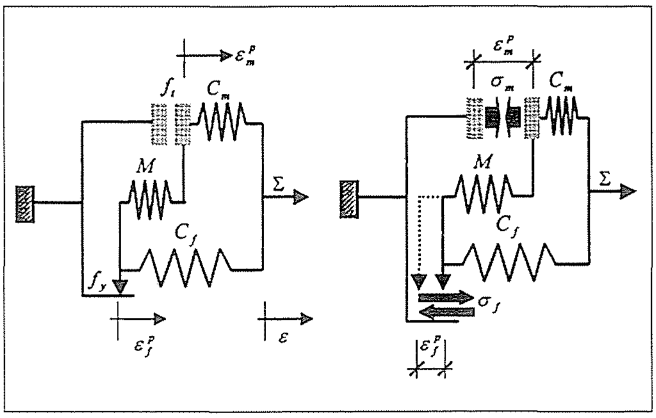


Fig 9: Rheological device for fibre reinforced concrete and its force-flow.

tic spring of rigidity M (see *Fig. 9*), representing the plastic matrix-fibre interaction in the plastic stress state (i.e. after matrix cracking or after fibre yielding). In other words, M, called coupling modulus, links the irreversible matrix deformation (i.e. ε^p_m) with the irreversible deformation of the reinforcement (i.e. ε^p_f). Σ and ε, respectively, represent the macroscopic stress related to equilibrium of the external forces and the total applied strain. Note σ_m and σ_f, respectively, are the resulting stresses on the cracked element and on the frictional device. In this case we can read for the stress-state of the rheological device shown in *Fig. 9*:

$$\Sigma = \sigma_m + \sigma_f = \left(C_m + C_f\right)\varepsilon - C_m\varepsilon^p_m - C_f\varepsilon^p_f$$

$$\sigma_m = C_m\left(\varepsilon - \varepsilon^p_m\right) - M\left(\varepsilon^p_m - \varepsilon^p_f\right)$$

$$\sigma_f = C_f\left(\varepsilon - \varepsilon^p_f\right) + M\left(\varepsilon^p_m - \varepsilon^p_f\right)$$

The composite stresses σ_m and σ_f will be referred to as composite matrix stress and composite fibre stress, respectively. These composite stresses are not a priori related by equilibrium to external forces, they are of pure rheological nature.

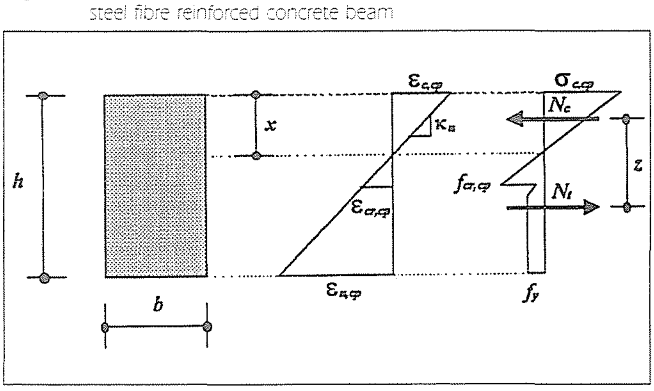
Results of the discussed 1-D rheological model are presented in *Fig. 8*. Characterising model parameters by compressive and direct tensile tests, an engineering model can be developed for modelling the bending behaviour of steel fibre reinforced concrete beams as shown in *Fig. 10*.

4. CONCLUSIONS

Experimental and theoretical studies were carried out at the Budapest University of Technology and Economics to investigate flexural behaviour of 18 fibre reinforced concrete beams. Test variables were:

- type of fibre reinforcement (hooked-end or crimped fibres)

Fig 10 Strain and stress distribution in the cross-section of steel fibre reinforced concrete beam



- fibre volume fraction (0, 0.5 V%, 1 V%)
- stirrup reinforcement.

Other parameters like beam cross-section and span, shear span-to-depth ratio were constant for the prestressed and for the non-prestressed members. Based on the test results the following conclusions can be drawn:

4.1 General comments on experimental study

Steel fibres can be efficiently used as shear reinforcement. Fibres may allow the spacing of stirrups to be increased and the reduction of the amount of shear reinforcement in high shear regions. Steel fibres do not only increase shear capacity but also provide substantial post-peak resistance and ductility in conventional reinforced concrete members and prestressed pretensioned members as well. With a sufficient amount of fibre the failure mode could be changed from shear failure to bending failure.

- If reinforced concrete beams contain the required amount of conventional shear reinforcement, and hence fail in bending, the addition of fibre does not considerably increase the failure load.
- The addition of steel fibres in reinforced concrete beams with no shear reinforcement or with low amount of shear reinforcement may give a considerable increase in the shear strength.
- The midspan deflection up to failure increases by increasing the quantity of fibre.
- The average crack width and sum of crack widths decreased in the proportion of fibre application (almost in all cases).

4.2 Modelling of steel fibre reinforced concrete

The material model for fibre reinforced concrete is developed taking plastic matrix to fibre interaction into consideration.

As the stress–strain relationships indicate, the model captures the essential features of the composite material and its constituents considering uniaxial tension.

The main advantage of this model is its clear physical significance and the low number of material tests characterising the model. In the simplest 1-D case, considering the known composite matrix and composite fibre behaviour, the only undetermined parameter is the coupling modulus M, which can be calibrated from a uniaxial tensile test.

The presented 1-D rheological model can easily be extend to a 3-D case using the energy approach replacing the scalar quantities by their tensorial counterparts. In this case the first term of the state equation reads:

$$\underline{\underline{\Sigma}} = \underline{\underline{\sigma}}_m + \sum_{1}^{f} \sigma_f\, \underline{e}_f \otimes \underline{e}_f$$

Interpretation of the coupling modulus also needs further investigation in the 3-D case. Similarly to the introduced coupling modulus describing the plastic matrix to fibre interaction (interaction of the frictional forces on the fibre surface), we may define a shear and fibre-fibre coupling as well. The 4th order tensorial counterpart of the coupling modulus M maybe written in the following form:

$$M = \sum_{1}^{f} M e_f \otimes e_f \otimes e_f \otimes e_f$$

5. ACKNOWLEDGEMENT

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EFFECTS OF CRACKING ON THE DYNAMIC CHARACTERISTICS OF CONCRETE BEAMS


Tamás Kovács – Prof. György Farkas

The paper deals with the assessment of damage in concrete beams by measurable dynamic characteristics. The aim of the presented laboratory experiment was the investigation of the relationship between the degree of cracking and the first natural frequency under defined deterioration conditions. The accumulated damage was assessed by the "total cross section of cracks" and the strain energy of the internal forces. As a result of the test, a definite relationship between the investigated parameters could be demonstrated. Then the main results and experiences gathered from on-site investigations carried out on existing concrete highway bridges will be summarised.

Keywords: vibration, natural frequency, excitation, amplitude-spectrum, bending stiffness

1. INTRODUCTION

During the design life of the existing concrete (mainly bridge) structures unfavourable serviceability conditions occur due to various corrosive effects and actions coming from extreme traffic loads and imposed deformations. Afterwards these may lead to insufficient load bearing capacity of the structure. Prevention or early detection of these deterioration processes are the most important tasks of maintenance (Farkas, 1999). This can be achieved by applying regular state-control methods in practice that are reliable and which can provide a continuous indication of the current level of accumulated damage in the structure with the lowest possible costs.

One of the possible procedures for monitoring concrete bridges is the regular observation of the change in the dynamic characteristics. This method is based on the assumption that for reinforced concrete structures subjected mostly to bending, the change in the cracking state is accompanied by the change in the bending stiffness (Illéssy, 1991). If the mass, the geometric dimensions and the bearing conditions are constant in the course of time, change in the bending stiffness induces a change in the dynamic characteristics and their relationship can be determined. Discontinuities in the material due to cracks cause change in the degree of the "internal friction", which leads to a change in the damping characteristics. For prestressed concrete beam structures, change in the normal force due e.g. to ruptures of prestressing tendons directly influences the dynamic characteristics.

Most of the existing concrete bridges are subjected to bending or to the combination of compression and bending. The reasons for the degradation in the bending stiffness are the decrease in the cross section of reinforcement due to corrosion, the degradation of the concrete material due e.g. to freezing, the developing cracks and the growing deflections due to transient overloading. In order to analyse these effects, the relationship between the dynamic characteristics and the degree of deterioration of a structure has to be determined numerically.

For this purpose laboratory test series were started at the Budapest University of Technology and Economics, Department of Structural Engineering, in 1999. These consist of laboratory experiments carried out on model beams intended for describing an exact relationship between the main dynamic characteristics and the different states of deterioration under artificially defined conditions and in-situ measurements intended to verify the experimental results on existing structures and for determining the achievable measurement accuracy on site.

For ordinary reinforced and prestressed concrete bridges, the first two or three natural frequencies and the logarithmic decrement of damping can easily be determined in a statistical way with adequate accuracy from the in-situ-registered deflection-time or acceleration-time functions. Additional advantages of a dynamic investigation are the availability of examples compared to the static investigations such as shorter required time and no restriction of traffic (Illéssy, 1980).

2. EXCITATION POSSIBILITIES

The solution to the differential equation of an excited vibrating system consists of two parts as follows, if the effect of damping is neglected (Vértes, 1972).

$$x = \left[C_1 \cdot \sin(\omega_0 \cdot t) + C_2 \cdot \cos(\omega_0 \cdot t)\right] + \frac{\frac{P_0}{c}}{1 - \left(\frac{\omega}{\omega_0}\right)^2} \cdot \sin(\omega \cdot t)$$

where:

- x the displacement of the system in the plane of vibration
- ω the radian frequency of the exciter force
- ω_0 natural radian frequency of the system
- P_0 maximum value of the exciter force
- c spring-constant
- t time
- C_1, C_2 constants

The first part as the result of the homogenous equation corresponds to the deflection-time function of the free-vibrating system and contains one of the natural frequencies as a constant value. This natural frequency belongs to the lowest level of internal energy and so it is mainly the first one. The second part indicates the inhomogeneous solution taking into account the effect of excitation. The deflection-time function of the system's forced vibration as the mix of the above two parts essentially depends on the type of excitation.

For the load bearing structures used in the civil engineering field the linear internal spring-constant can be assumed which results in harmonic free vibration. In addition, for these structures, low intensity of internal damping exists and the differences between the natural frequencies coming from taking and not taking into account the effect of internal damping are low enough to be neglected from the point of view of determining the natural frequencies.

In case of strictly harmonic excitation the natural frequencies quickly diminish from the vibration pattern at the beginning of the vibration due to the generally occurring damping effects. Further on the structure vibrates at the frequency of the excitation. In case of non-harmonic excitation both the natural and the exciter frequencies exist in the vibration pattern. The actual natural frequencies existing in the vibration pattern and in the homogenous part of the solution depend on the frequency range of the initial excitation. This relationship between the actual natural frequencies and the type of the excitation gives the possibility of measuring the natural frequencies of structures on site.

In case of on site measurements, the first few natural frequencies are determined. In laboratory conditions it is possible to apply harmonic excitation. In this case, by continuously changing the excitation frequency resonance effects appear in the vibration pattern (in the recorded deflection-time function). Those indicate the equality of the excitation and one of the natural frequencies. For on-site investigations, the excitation used has to satisfy both of the following conditions. The frequency range in the amplitude - frequency function of the excitation has to be wide enough to contain all the natural frequencies to be determined and the intensity of the amplitudes in the amplitude-frequency function have to be sufficiently high at the frequencies close to the natural frequencies in order to produce a significant excitation at those places. The valuation and the selection of the excitation possibilities in a real situation can be carried out on the basis of the mentioned aspects (Kovács, 1998).

The ideal excitations in this respect are the so-called "white-noise"-type excitations, whose significance is only theoretical. In these cases the amplitude in the amplitude-frequency function of the excitation is constant all over the frequency range. In ideal circumstances, the excitation effect of wind can be taken into account in a similar way where the nearly constant amplitudes exist along only a certain part of the frequency range.

The approximation of the so-called Dirac-δimpulse (*Fig. 1*) and that of the impact effects shown in *Fig. 2* are mostly used as artificially produced excitations carried out on existing bridges. In practical cases the amplitude of the impulse is $A \neq \infty$ and its time is $dt \neq 0$.

Common characters of the above impulse excitations are the significantly higher intensity of excitation (as a result of the $\sin(x)/x$ and the $1/x$ functions) in the lower domains of frequency range, mostly containing the first few natural frequencies compared to that in the higher domains containing the higher natural frequencies. Therefore these can only be applied for determining the first few natural frequencies. The Dirac-δexcitation and the excitations shown in *Fig. 2* can be performed by impact effects and by applying objects falling on or off the examined structure.

For highway bridges the road traffic is frequently used for excitation. The wide amplitude-spectrum of the exciter effect comes from the non-uniform running-properties of different vehicles. Therefore it is essential to examine a sufficiently long time interval to reach an appropriate resolution of the resulting frequency range. Because of the stochastic character of excitation, it is necessary to evaluate the results in statistical way. In this way, excitation frequencies which are not close enough to any of the natural frequencies, will occur in a random manner and will be sifted out from the vibration pattern. Excitation frequencies which are close to one of the natural frequencies will amplify the weight of free vibration in the vibration pattern enabling a determination of the corresponding natural frequencies (Kálló, 1997).

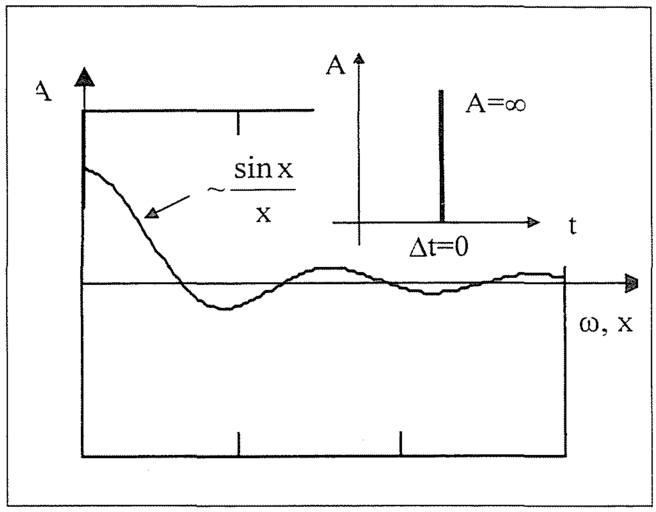


Fig. 1 Excitation function of Dirac

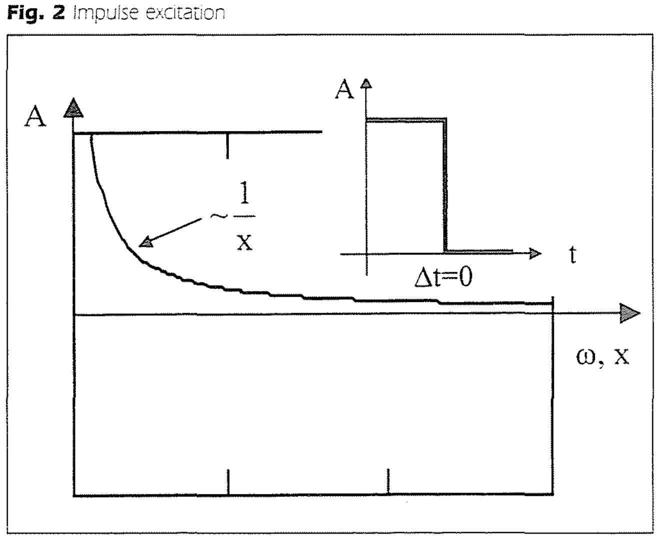
2. LABORATORY TEST

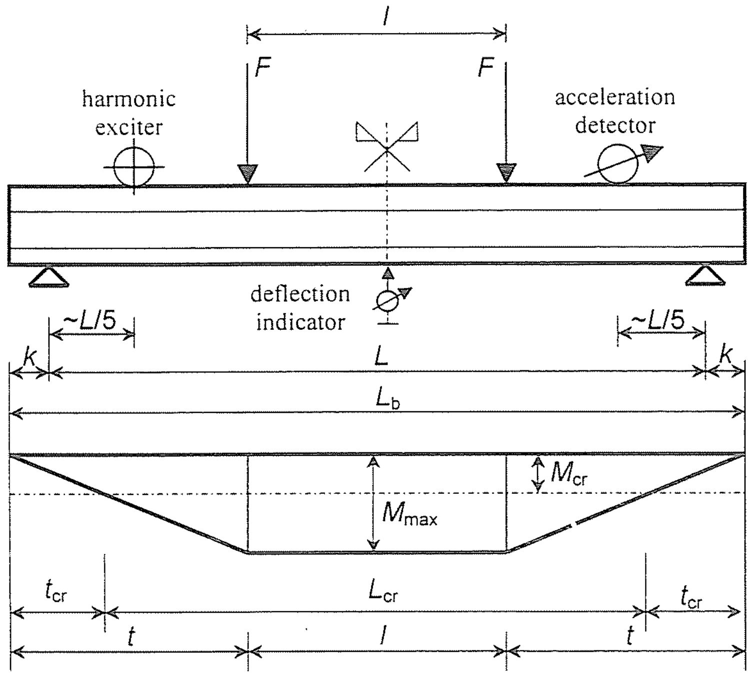
In order to determine a relationship between the first natural frequency of a structure and the current states of deterioration under defined conditions, a simply supported reinforced concrete model beam with relatively low reinforcement ratio has been investigated (Kovács, Farkas, 2000, 2001; Farkas, Kovács, 2001).

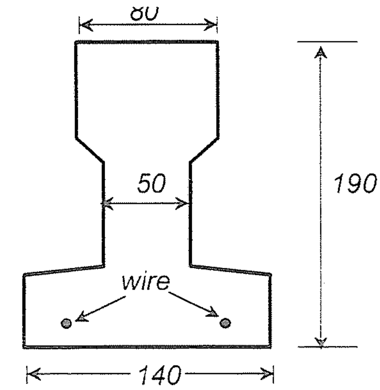
2.1 Geometry and materials

The tested specimen was a so-called E-beam, which was commonly used in the floor structures of houses in Hungary. This was originally a precast, prestressed concrete product with constant concrete section and with 7 prestressing wires in the tension zone and a single wire in the compression zone. However, for this experimental purpose, it had been manufactured

Fig. 2 Impulse excitation







Mark of the deterioration state	Load		Length of the cracked zone
	F [kN]	l [m]	L_{cr} [m]
0	0	-	0
1	5.13	0.5	1.59
2	6.63	1.0	1.94
31	8.13	1.5	2.16
32	11.13	1.5	2.42
4	9.63	1.0	2.42
5	11.13	0.5	2.42

Fig. 3 Arrangement of the four-point bending test

only with 2, unstressed prestressing wires in the tension zone along the full length. The cross section and the side view of the specimen can be seen in *Fig. 3*. The beam was made of concrete grade C35/40 and prestressing steel grade ST 1770-5.34. Based on the given material properties and geometric dimensions, the design and the mean values of the bending capacity (M_{Rd} and M_{Rm}) and the cracking moment (M_{cr}) as well as the reinforcement ratios (μ) based on the total concrete section (A_c) and on the width of the web (b_w), have been calculated according to the usual assumptions of the EC-2. The results can be found in *Table 1*, where d is the effective depth.

2.2 Experimental arrangement and program

For modelling a deterioration process, different cracking states were produced by a four-point bending test in 7 different loading steps whose arrangement is shown in *Fig. 3*. The acting forces (F) were equal to each other and symmetric to the midspan in each load position.

A harmonic exciter fixed directly to the beam at approximately one-fifth of the span from the one support induced a forced vibration whose frequency could be continuously changed during the dynamic measuring phase. Certain parts of the acceleration-time function of vibration were recorded by an acceleration detector placed similarly at approximately one-fifth of the span from the other support. During the dynamic measuring phase, the signs of vibration got through the acceleration detector to an analyser, which recorded the acceleration-time function and immediately made a Fourier-transformation on it directly producing its amplitude spectrum. By changing the revolution of the harmonic exciter, the frequency of the forced vibration could be equalized by the natural frequency of the beam, which resulted in resonance effects. These effects appeared as suddenly-increasing, relatively high peak amplitudes in the acceleration-time function having finite values due to the effect of structural damping. In this way the first natural frequency as the frequency, which belonged to the parts of the acceleration-time function that covered the maximum amplitudes, could be determined. A deflection indicator was placed under the middle section of the specimen to register the static deflections under loading as well as the residual deflections after reloading.

Table 1 Cross sectional properties

	Designed value	Real value
Bending capacity, M_{Rd}, M_{Rm} [kNm]	9.33	14.08
Cracking moment, M_{cr} [kNm]	4.52	4.47
Reinforcement ratio [%]		
$\mu = A_s/A_c$	0.284	
$\mu = A_s/(b_w d)$	0.527	

The different deterioration states of the specimen were modelled by producing different cracking states. Cracking is one of the most important properties of existing reinforced concrete beam-structures subjected to flexure. Under normal circumstances the current level of damage in these beams can be most effectively measured by the current degree of cracking. In addition, before the bending failure, a considerably cracked deterioration state always develops in a lightly or normally reinforced concrete beam due to the fairly large plastic strain in the reinforcement that may significantly influence the dynamic characteristics.

Altogether seven deterioration states were produced by choosing different values and different positions for the concentrated loads according to the table shown in *Fig. 3*. Therefore, as a first step the length of the cracked zone (L_{cr}) was gradually increased considering approximately the same level of bending moment at midspan. In the next step the length of the cracked zone was taken to be constant and the level of bending moment was increased.

The general course of the experiment was the following. First the loading step i was applied to the beam producing the appropriate cracking state. The deflection under the maximum load was registered at the middle section. After the loads were removed from the specimen the residual deflection was also registered at the same place after which the dynamic measuring phase took place. After the measuring phase the next loading step (i+1) followed with a changed position and value of acting forces (F).

Approximately 30 amplitude spectrums have been registered in each deterioration state. Resolution of spectrums (Δf) depended after all on the length of the analysed, individual parts of the time function (t_{max}). In the course of this experiment, t_{max} was set as 16 s so the resolution of spectrums could be obtained as $\Delta f = 1/t_{max} = 0.0625\ s^{-1}$. In order to neglect the effects coming from the uncertainties of synchronizing the excitation frequency to the natural frequency of the beam during the dynamic measuring phases, the natural frequencies have been evaluated in a statistical way.

2.3 Results

To access the accumulated damage in a structure in any deterioration state, it is also necessary to define the concept of damage numerically. In the followings, the degree of deterioration will be represented on the basis of the degree of cracking and the strain energy of internal forces under maximum load.

2.3.1 Examination of cracking

Within the frame of this examination, the crack widths at the midspan and the total cross section of cracks have been calculated and their relation to the first natural frequency has been investigated in each deterioration state.

The average crack width (w_{cr}) at the middle section was calculated by multiplying the average final crack spacing (s_{rm}) and the mean strain in the reinforcement (ε_{sm}) as follows. The effect of tension stiffening was also taken into account and included in ε_{sm}.

$$w_{cr} = s_{rm}\, e_{sm}$$

The relationship between the crack width at the midspan and the first natural frequency can be seen in *Fig. 4a*. As shown, the greatest decrease in the first natural frequency came out between the deterioration states number 0 and number 1, simultaneously with the appearance of the first cracks. Parallel to the increase of the crack width, the first natural frequency generally decreased but in the higher domains of crack width, the intensity of this decrease became smaller.

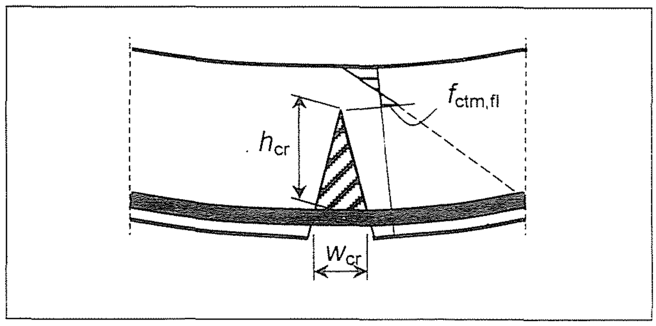


Fig. 5 Cross section of a crack

As the crack width decreased between the deterioration states number 2 and number 31 owing to the smaller bending moment in state number 31, only the crack width at the midspan was unable to characterize the change of the natural frequency during the full deterioration process. For this reason, a new quantity, the total cross section of cracks (A_{cr}) has been defined and determined in each deterioration state according to *Fig. 5* and the following equation:

$$A_{cr} = \sum_{L_{cr}} \frac{s_{cr,i}}{s_{rm}} \frac{h_{cr,i}\, w_{cr,i}}{2}.$$

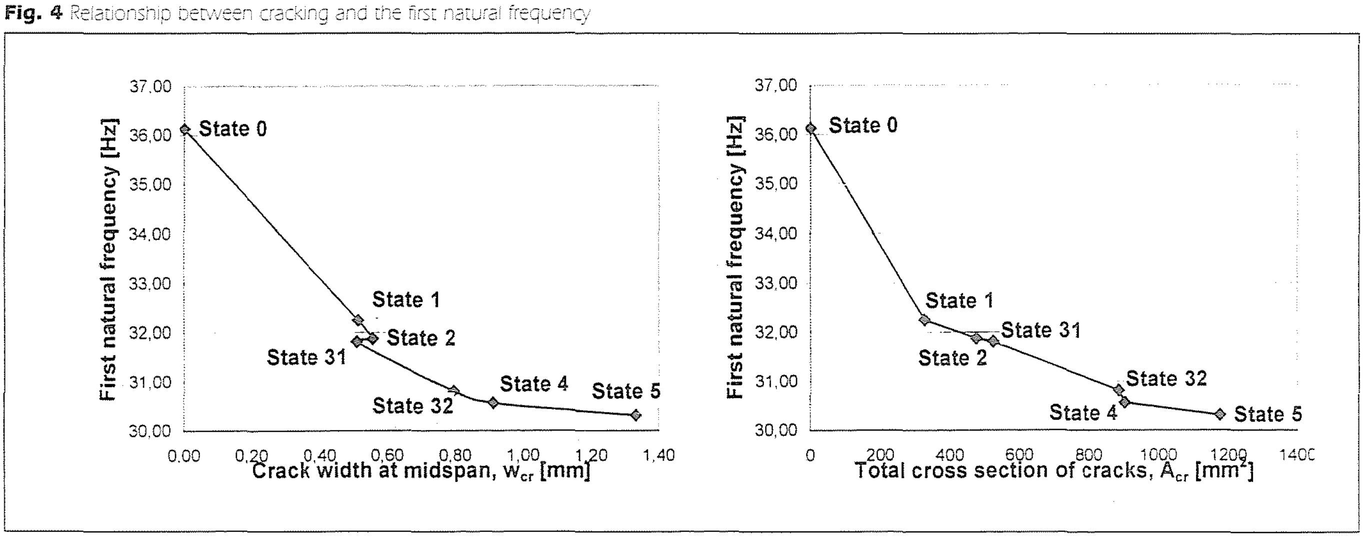
It depends both on the total number of cracks in the whole beam and on their widths. Here $s_{cr,i}$ was the distance along L_{cr} on which the averaging of h_{cr} and w_{cr} took place.

The relationship between A_{cr} and the first natural frequency is shown in *Fig. 4b*. The shape of this curve was similar to the curve shown in *Fig. 4a* but there was a monotonic decreasing function between the total cross section of cracks and the first natural frequency. In addition, A_{cr} was not a quantity linked with a cross section like the crack width but could take into account all the damages in the whole beam. In this way, A_{cr} proved to be able to characterize the degree of damage in a reinforced concrete beam subjected to bending.

2.3.2 Approach based on the strain energy of internal forces

This approach is based on the assumption that the degree of damage in a concrete beam without prestressing and subjected to bending is proportional to the amount of strain energy caused by internal bending moments greater than the cracking mo-

Fig. 4 Relationship between cracking and the first natural frequency



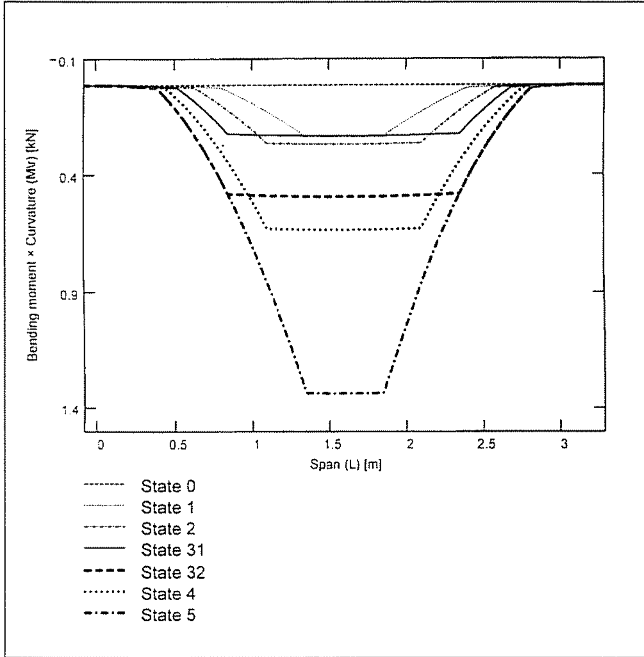


Fig. 6 Strain energy functions in different deterioration states

ment in a non-linear elastic, cracked stage. The behaviour of a cross section subjected to bending moment less than the cracking moment is assumed to be linear elastic before the appearance of the first cracks. That is, no damage and change in the bending stiffness occurs at this stage.

The previously referred strain energy linked with a cross section being in the non-linear elastic, cracked stage was calculated by the following, summing along the distance of L_{cr} in each deterioration state:

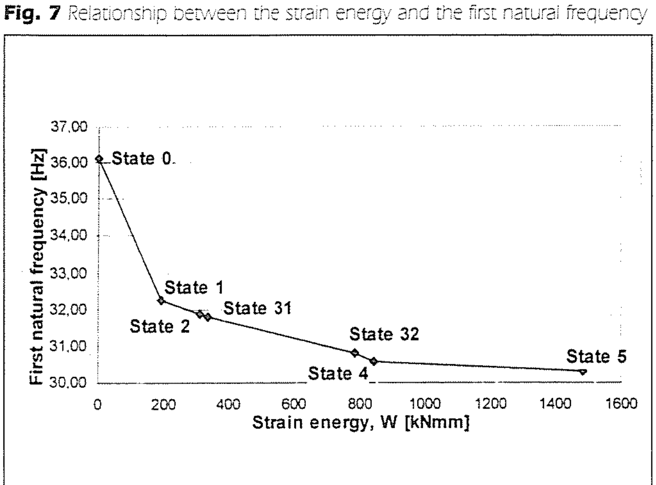
$$W = \int_{L_{cr}} M(x)\rho(x)\mathrm{d}x \,,$$

where $M(x)$ was the bending moment function according to *Fig. 3* and $\rho(x)$ was the curvature function taking into account the effect of tension stiffening.

The $M(x)\rho(x)$ function-product can be seen in *Fig. 6* for the different deterioration states. Areas under the functions shown in *Fig. 6* along the length of L_{cr} represented the strain energy of internal bending moments.

The relationship between the strain energy and the first natural frequency is shown in *Fig. 7*. As can be seen, the shape of the curve shown in *Fig. 7* was similar to the shape of the curve shown in *Fig. 4b*, as it was derived from the same input parameters (bending moments and cross sectional properties).

Fig. 7 Relationship between the strain energy and the first natural frequency



2.3.3 Conclusions based on the experimental results

Parallel dynamic and static investigation of a simply supported, reinforced concrete model beam under artificially created, defined deterioration conditions has been introduced in this paper. The different deterioration states were modelled by different cracking states, which led to gradual decreases of the bending stiffness. Based on the results, the following statements can be made:

- The new quantities defined (total cross section of cracks, strain energy of internal forces) were useable in the assessment of the accumulated damage in the specimen. In calculating these quantities, the quantity and the type of reinforcement were the most important input parameters.
- The change of the first natural frequency was unambiguously demonstrated to be a function of the defined new quantities.
- The first natural frequency simultaneously decreased with the increase of all the quantities characterizing the degree of cracking (e.g. with decrease of bending stiffness).
- For the analysed lightly reinforced concrete beam where the degree of plastic deformation in the reinforcement was not significant during the load history, this decrease in the first natural frequency was approximately 15-20% between the extreme states of deterioration.
- The tendency for decrease in the first natural frequency was monotonic during the full damage accumulating process. The highest intensity of decrease could be observed when the first cracks appeared, while this intensity became smaller in deterioration states close to failure.

3. ON SITE INVESTIGATIONS AND THEIR CONCLUSIONS

Dynamic measurements have been carried out on reinforced concrete highway bridges using the normal road traffic for excitation. The essence of the measuring and evaluating method was the assumption of a stochastic character for the exciting effect. The dynamic characteristics came out from statistical analysis for which a sufficient volume of measuring data was needed. With respect to the information gathered from this field of research the following statements can be made:

- Using the normal road traffic for excitation and based on the on-site-recorded, sufficiently long acceleration-time functions average amplitude spectrums can be produced in statistical way and which have significant peak-ordinates (Kálló, 1997). Based on the resulting spectrums derived from combining these average amplitude spectrums, the first three natural frequencies and the belonging eigenforms can be easily determined (Kovács, Farkas, Kálló 1998).
- Determination of the logarithmic decrement of damping using the normal road traffic for excitation is only possible on the basis of representative sample containing large number of statistical data because of the high standard distribution values. The main reason of the high standard deviation values is the fact that in the presence of normal road traffic during measurement it is difficult to select the periods of the acceleration-time function, which clearly contain the signs of free vibration. After the statistic analysis of the previously referred set of data the logarithmic decrement value can be given in an interval with a relative frequency of ~0.01 instead of a definite numerical value (Kovács, 1999).

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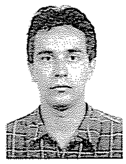
PRESTRESSING IN FLOORS USING UNBONDED TENDONS



Prof. György Farkas - Tamás Kovács

This paper deals with the prestressing of reinforced concrete flat slabs for buildings using unbonded tendons and introduces the main benefits and disadvantages of its application. The calculation method of the equivalent prestress and a practical procedure regarding the determination of the slab thickness will be presented as the main steps of the preliminary design. Then a short outline of the simplest horizontal arrangements of prestressing tendons over a unit slab follows, which will be extended by a comparison analysing the required specific amounts of prestressing and reinforcing steel in case of different design requirements.

Keywords: unbonded prestress, arrangement of tendons, equivalent prestress, flat slab, required amount of steel

1. INTRODUCTION

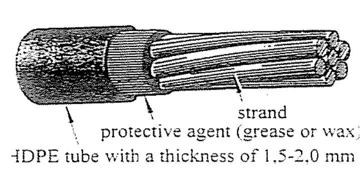
By prestressing the reinforced concrete flat slab floors of buildings (mainly of car parks), a more advantageous horizontal arrangement of columns can be achieved from both the architectural and structural designer's point of view and, owing to this, the structures more efficiently adjust to the function of the buildings compared to those without prestressing.

Sometimes, after the opening of basement car parks built in the last few years in Hungary, massive cracks have appeared on the reinforced concrete flat slabs with a thickness of between 250-300 mm because of static and concrete technological problems. This caused cracks through the top wearing layer that allowed water with a high de-icing salt content to drip from the cars directly to the load bearing structure (Armuth, Deák, 2001). An appropriate solution for this unfavourable durability problem could be the limitation of crack widths or the complete elimination of cracks, which can be satisfactorily achieved using post-tensioning of these floor slabs.

The consequently designed prestressing system favourably influences both the structural behaviour - such as the load bearing capacity and the serviceability - and the durability of reinforced concrete slabs.

Recently, the internal unbonded prestressing strands running in high-density polyethylene (HDPE) tubes filled with high-melting grease and supplied with factory-made corrosion protection (according to *Fig. 1*), have been widely used for these purposes. The arrangement of these strands in a prestressing cable can be nearly circular or placed in one row next to each other. The latter type of cable cross section is useful in order to reach the maximum effective depth of prestressing force at the critical cross sections of the structure. Additionally, a significantly low loss of prestress due to friction and a relatively small radius of curvature can be achieved because of the low value of friction coefficient between the steel wires and the HDPE tube.

Fig. 1: Individual unboned strand



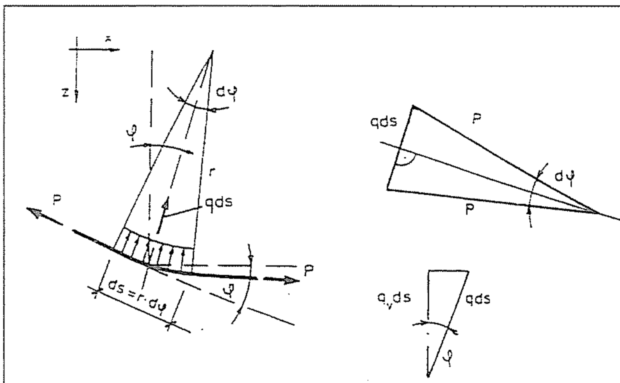
Prestressed floor slabs as newly designed structures are practically used for high intensity of live load and/or for long spans between the supporting columns. In order to reduce the number of anchorages, it should be attempted to use as long tendons as possible and therefore the advantages of prestressing can be efficiently utilised in the first place with large floor-space slabs. According to the experiences obtained from this field, for usual spans the total cost of a prestressed flat slab floor is approximately the same as that of a cast-in-place slab having the same load bearing capacity. In these cases the other advantages and disadvantages should be considered with regard to both types of floor slabs in order to make the correct decision about the construction method.

2. PRELIMINARY DESIGN OF PRESTRESSED SLABS

2.1 Equivalent prestress

During the structural analysis the effects of prestress can be calculated in a very simple way by determining the equivalent prestress and superimposed on the effects of weight (permanent and variable) loads. The tendons usually run along a nearly guadratic curve to their self-weight. The equivalent prestress of a prestressing tendon consists of concentrated forces (P) acting at the anchorages and a knife-edge load system (q).

Fig. 2 Calculation model of the equivalent prestress



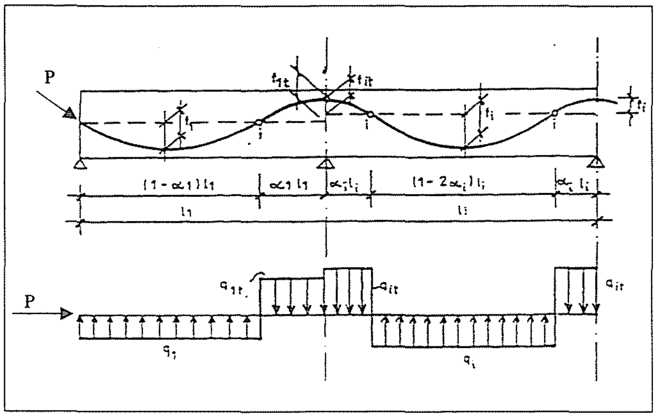


Fig. 3 Equivalent prestress of a prestressing tendon running along two spans

If the φ shown in *Fig. 2* is small enough ($< 12^0$) and the prestressing tendon runs with constant curvature (quadratic curve) along a particular section of the structure, the intensity of q can also be taken constant and perpendicular to the middle surface of the slab along the section in point as shown in *Fig. 3* and its value comes from the following equation (Bölcskei, Tassi, 1982):

$$P\,\mathrm{d}\varphi = q\,\mathrm{d}s = q\,r\,\mathrm{d}\varphi \;\Rightarrow\; q = \frac{P}{r} = P\frac{\mathrm{d}^2 z}{\mathrm{d}x^2} = 8\frac{P\,f}{\ell^2}$$

Here P is the mean prestressing force along the section in point, f is the arch rise of the quadratic curve having a horizontal span of ℓ.

2.2 Optimal slab thickness

The slab thickness plays a significant role in designing prestressed floor slabs while it determines the maximum eccentricity of prestressing tendons as well as the intensity of the self-weight of the slab to be equalized by the equivalent prestress and essentially influences the bending rigidity of the structure. The optimal slab thickness derives from the minimization of the specific cost function of the slab according to the following equation and depends on the applied design requirement.

$$C = C_p V_p + C_c V_c \;\Rightarrow\; \text{min!}$$

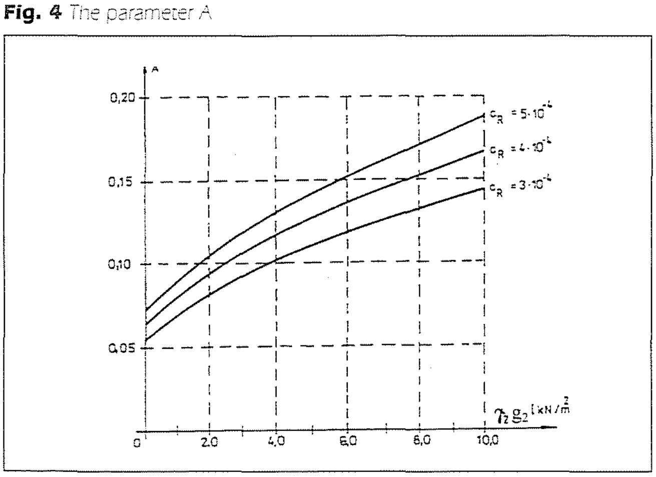
Here C_p is the specific cost of the prestressing system including tendons, anchorages and site assembly, C_c is that of the reinforced concrete slab including concrete, reinforcement, site assembly and casting, V_c and V_p are the specific volume of reinforced concrete slab and prestressing tendons respectively. Designing the structure subjected to the self-weight (g_1) and the superimposed dead load (g_2) without tensile stresses, the optimal (and constant) slab thickness derives as follows in case of a continuous structure with constant ℓ spans,

$$v = \frac{\ell}{8}A + \frac{3}{2}u$$

where u is the concrete cover. The values of A can be taken from the diagram shown in *Fig. 4* where γ_2 is the partial safety factor of g_2 and,

$$C_R = \frac{C_p}{\sigma\, C_c}$$

Fig. 4 The parameter A



3. HORIZONTAL ARRANGEMENTS OF PRESTRESSING TENDONS

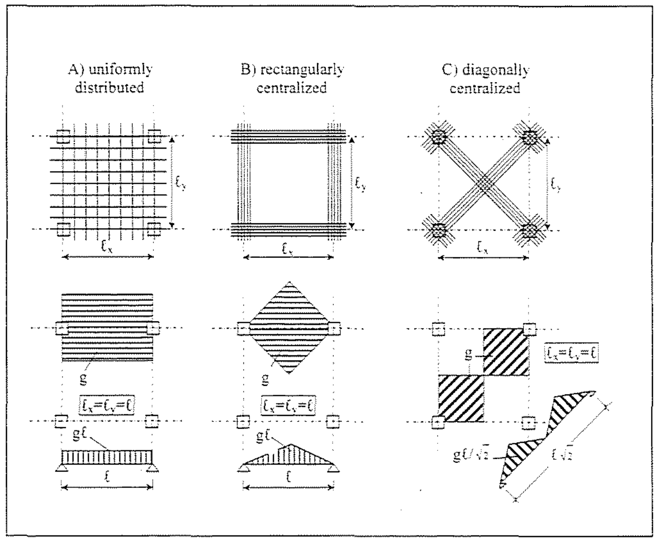
The distribution and the intensity of the resulting internal bending moments coming from the design load including prestress and the deflections over the analysed part of the slab can efficiently be influenced by the horizontal arrangement and the vertical alignment of prestressing tendons.

For rectangular arrangement of supporting columns and for constant distances between them, the simplest examples for horizontal arrangement of prestressing tendons are shown in *Fig. 5* (Farkas, Kovács, 2002).

The counter-distribution of bending moments coming from the *uniformly distributed* arrangement of prestressing tendons most perfectly adjusts to the corresponding linear elastic distribution of bending moments coming from the external unit load ($g = 1{,}0\ \frac{kN}{m^2}$). An additional advantage of this arrangement is the possible application of tendons with small diameter (possibly single strands) so the eccentricity of tendons that is the effective depth at the critical sections (highest, deepest and intersection points of tendons) can be maximized.

However, a significant disadvantage of this arrangement is the fact that the tendons crossing each other along the lines connecting the axes of columns and led with opposite curvature at the same points, give their loads directly to each other

Fig. 5 Examples for the horizontal arrangement of prestressing tendons



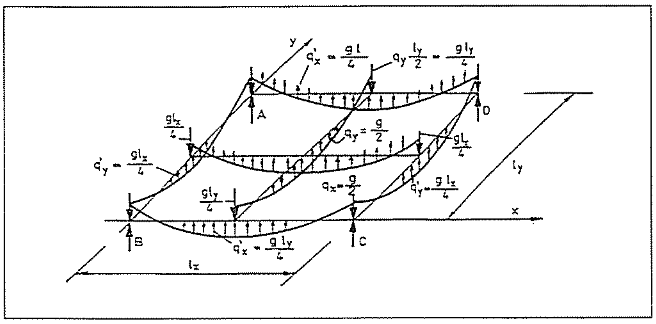


Fig. 6 Load bearing model of a uniformly distributed tendon-system

according to *Fig. 6*. In this way, the same load has to be carried separately by tendons led in both directions that results in an extra amount of prestressing steel. The eccentricity of the tendons crossing each other above the columns are to be smaller than that in the span far from the columns.

The main advantage of the *rectangularly distributed* arrangement of tendons is the fact that the tendons give their loads directly to the columns so they do not load each other. Assuming the same geometric and support conditions and prescribing the same design requirement, in general cases, this arrangement results in less amount of prestressing steel than the uniformly distributed arrangement. An additional benefit of this arrangement is the possibility of designing holes and openings in the floor in much simpler way compared to the uniformly distributed arrangement. A disadvantage of this arrangement is that the equivalent prestress concentrates along the lines connecting the axes of columns so the bending moments coming from the uniformly distributed external loads can not be equalized by the prestress in each point of the slab. For this reason, the equivalent prestress needed, e.g. for equalizing the bending moment at the midspan, can cause significant overloading at other points of the slab (e.g. along the lines connecting the axes of columns). A further disadvantage is the reduced effective depth of the prestressing cables having large diameters above the columns for geometric and constructional reasons.

The *diagonally centralized* arrangement of prestressing tendons has the same benefits as the rectangularly centralized arrangement relating to the direct load transfer from the tendons to the columns and the possibility of designing or subsequently making holes and cuttings off in the slab. However, the deepest points of the tendons are at the midspan for tendons led in both directions so they are more efficient from the point of view of limiting the maximum deflection compared to the rectangularly centralized arrangement. The bending moments coming from the uniformly distributed external loads also can not be equalised by the prestress in each point of the slab. The same geometric and constructional problems arise at the eccentricities of tendons above the columns and additionally, the same arch rise with longer length of prestressing tendons between two columns belongs to the tendons compared to the corresponding dimensions for the rectangularly centralized arrangement. According to the equation of the equivalent prestress written above, higher prestressing force, which means a greater amount of prestressing steel, is needed for producing the same equivalent knife-edge load (q) compared to the rectangularly centralized arrangement. For this arrangement, the tendons are usually not parallel to the edges of the slab which results in more complicated distribution of internal forces at the edges and may cause difficulties during the time of construction.

Comparison of the above three horizontal arrangements of prestressing tendons has been made by approximate and numerical models on the basis of the calculated specific amounts of prestressing steel:

a) required for the most perfect "equalization" of the bending moments coming from the unit weight-load (g) over a unit slab shown in *Fig. 5*;

b) required for eliminating the deflection at the midspan coming from the unit weight-load (g).

3.1 Approximate model

This method is based on the following assumptions:

- The bending moments are calculated on fictitious, simply supported beams running in the centre line of the groups of tendons subjected to loads shown in *Fig. 5*. In this way, the curvature of the slab in the two perpendicular directions is not taken into consideration.
- The bending rigidity of fictitious beams is constant along their full length in contrast with the intensity of loads.
- Concentrated supports are assumed.

Based on the above assumptions, the amounts of prestressing steel required for equalizing the bending moment coming from the unit weight-load g at the midspan of a unit slab area of $\ell \times \ell$ have been calculated for the different horizontal arrangements of prestressing tendons. The results can be found in *Table 1*.

3.2 Numerical model

In order to analyse the required amounts of prestressing steel numerically, a finite element model of a unit slab area of $\ell \times \ell$ (ℓ=9,0 m) as an internal part of a two direction continuous slab has been made using the appropriate symmetry conditions. The prestress was modelled by the equivalent prestress according to the section 2.1 and the applied unit load (g) was uniformly distributed.

The required amounts of prestressing steel satisfying the condition **b)** according to the section 3 are also included in *Table 1*. The values in the last column of *Table 1* give the ratios of the required amounts in percentage where the necessary amount according to the approximate model in case of the uniformly distributed horizontal arrangement of prestressing tendons was taken as 100%.

The required amounts of prestressing and reinforcing steel satisfying the condition **a)** of the section 3 is shown in *Fig. 7*. The quantity plotted on the vertical axis is in almost direct proportion to the required amount of reinforcing steel. The values of the prestressing steel read from the horizontal axis at the minimum point of the functions belonging to the different horizontal arrangements can directly be compared to the values being in the last column of *Table 1*. The $m_{g,i}$ and $m_{p,i}$ refer to the mean values of the bending diagram coming from the g and the prestress respectively over the slab area of dA_i. The σ means the effective stress in the prestressing tendons.

3.3 Results

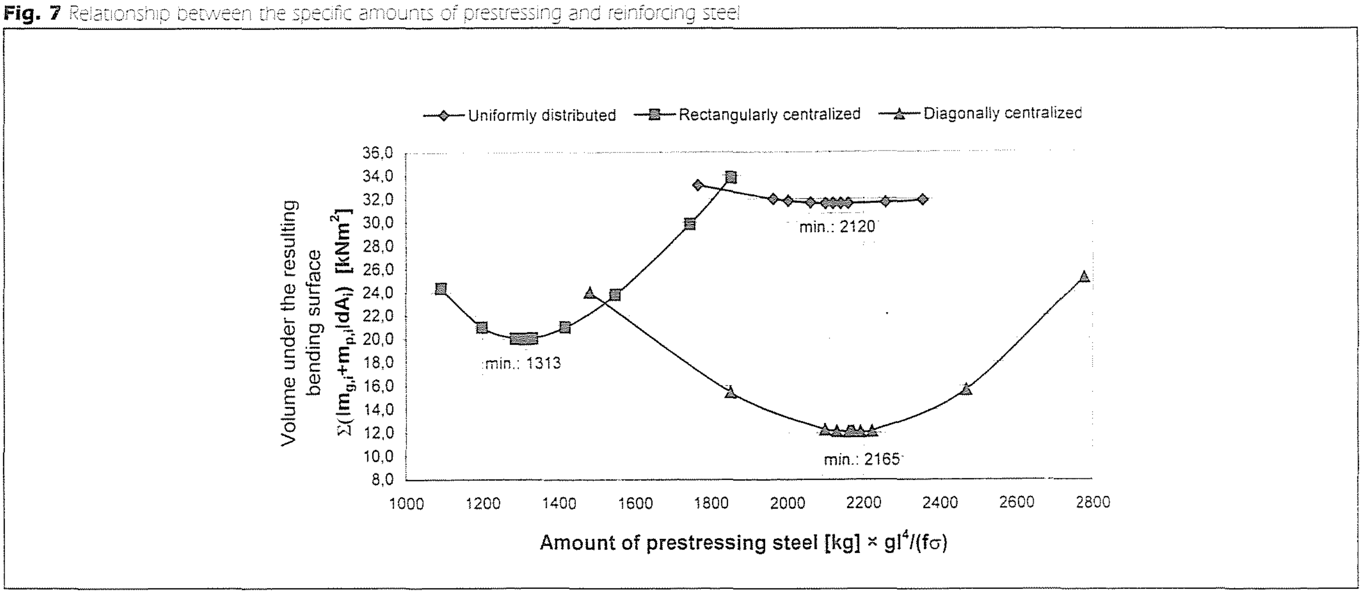
Based on the analysis of a general, internal part of a continuous flat slab with constant distances between the supporting columns in both direction and subjected to a uniformly distributed load g, the following statements can be made in connection with the necessary amounts of steel:

	Condition	Sum of the required prestressing force along the distance ℓ	Length of cables belonging to P over an $\ell \times \ell$ unit slab	Constant $\left[\frac{m^2}{kN}\right]$	Required amount of prestressing steel
		P [kN]	ℓ_p [m]		m [kg]
Uniformly distributed prestress					
Approximate calculation		$1{,}0 \ \frac{g \ell^3}{8f}$	2ℓ	$\frac{1}{\sigma}$	$1963 \ \frac{g \ell^4}{f \sigma}$ (100%)
Numerical calculation	deflection b)	$1{,}157 \ \frac{g \ell^3}{8f}$			$2269 \ \frac{g \ell^4}{f \sigma}$ (116%)
Rectangularly centralized prestress					
Approximate calculation		$0{,}667 \ \frac{g \ell^3}{8f}$	2ℓ	$\frac{1}{\sigma}$	$1311 \ \frac{g \ell^4}{f \sigma}$ (67%)
Numerical calculation	deflection b)	$0{,}888 \ \frac{g \ell^3}{8f}$			$1743 \ \frac{g \ell^4}{f \sigma}$ (89%)
Diagonally centralized prestress					
Approximate calculation		$0{,}708 \ \frac{g \ell^3}{8f}$	$2\ell\sqrt{2}$	$\frac{1}{\sigma}$	$1963 \ \frac{g \ell^4}{f \sigma}$ (100%)
Numerical calculation	deflection b)	$0{,}716 \ \frac{g \ell^3}{8f}$			$1986 \ \frac{g \ell^4}{f \sigma}$ (101%)

Table 1 Required amounts of prestressing steel over a unit slab area of $\ell \times \ell$ for different horizontal arrangements of tendons

- Focusing on the necessary amounts of prestressing steel that require a minimum amount of reinforcing steel: If the required amount belonging to the *rectangularly centralized* arrangement is taken as 100% (1313), the corresponding amounts are 165% (2165) and 161% (2120) for the *diagonally centralized* and the *uniformly distributed* arrangements respectively.
- Focusing on the required minimum amounts of reinforcing steel, these are very different from each other. If the required minimum amount of reinforcing steel belonging to the *rectangularly centralized* arrangement is taken as 100%, the corresponding amounts are 60% and 160% for the *diagonally centralized* and the *uniformly distributed* arrangements respectively.
- To choose the optimal ratio of prestressing and reinforcing steel it is also necessary to take into account the current steel prices.
- Focusing on the limitation of deflections, the amounts of prestressing steel required for equalizing both the bending moment and the deflection at the midspan are almost the same for the *diagonally centralized* (2165/1986) and the *uniformly distributed* (2120/2269) arrangements, while for the *rectangularly centralized* arrangement the amount required for equalizing the deflection at the midspan (1743) is significantly greater and goes together with about a 50% increase of reinforcing steel than that required for equalizing the bending moment at the same place (1313).

Fig. 7 Relationship between the specific amounts of prestressing and reinforcing steel



4. CONCLUSIONS

The paper dealt with the aspects of applying internal unbonded prestressing system in flat slabs of buildings. It introduced the most important steps of the preliminary design of these structures such as the calculation method of the equivalent prestress and the determination of the optimal slab thickness. The possible horizontal arrangements of prestressing tendons was analysed and compared to each other in case of a general, internal part of a two direction continuous flat slab from the point of view of the required amounts of prestressing and reinforcing steel and the limitation of deflections.

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PRESTRESSING PROVIDED BY CFRP TENDONS

Adorján Borosnyói – Prof. György L. Balázs

Corrosion induced deterioration of reinforced concrete members directed interest towards non-metallic (i.e. non-corrosive) reinforcement. Non-metallic reinforcement has some superior properties compared to steel reinforcement in addition to their corrosion resistance. This paper details the main characteristics of non-metallic reinforcement and the results of an experimental project on CFRP prestressed concrete beams.

Keywords: Durability, non-metallic reinforcement, CFRP prestressing, bond, cracking, pivot point

1. INTRODUCTION

Corrosion induced deterioration of reinforced and prestressed concrete structures over the past decades has resulted in lower service life and higher maintenance costs. Due to environmental pollution and use of de-icing salts, the durability of reinforced and prestressed concrete highway bridges are therefore of high interest.

Corrosion can form in reinforced concrete in several ways (pitting corrosion in the presence of chloride ions, stress corrosion in prestressed reinforcement, contact corrosion between highly alloyed prestressing tendons and mild steel non-prestressed reinforcement, etc.) and whenever the initial high alkalinity of concrete drops under pH 9. The conditions needed for corrosion of reinforcement in concrete to occur are:

- The presence of a material to corrode (steel),
- The presence of water,
- The presence of oxygen,
- Concrete with a pH < 9.

To overcome corrosion induced deterioration in reinforced concrete members several proposals exist:

- Construction with concrete of low w/c ratio (low permeability),
- The addition of corrosion inhibitor chemicals in the concrete,
- The application of special coatings on the concrete surface,
- Cathodic protection of embedded reinforcement,
- The use of epoxy coated reinforcement,
- The use of stainless steel reinforcement,
- *The use of non-metallic reinforcement.*

Every proposal (except the last one) tries to reduce corrosion in the presence of steel, the material which is subject to corrosion. On the other hand, the application of non-metallic and therefore entirely corrosion resistant reinforcing materials provides a promising alternative by changing the endangered material to another material (in this case steel is eliminated from the system).

Non-metallic reinforcement is made of Fibre Reinforced Polymers (FRP). High strength fibres of FRPs can be made of glass, aramid or carbon with a volumetric fibre ratio of 60 to 65%. The matrix resin is usually epoxy. Carbon Fibre Reinforced Polymers (CFRP) show excellent fatigue strength, low relaxation and creep behaviour besides high tensile strength and corrosion resistance. CFRPs seem to be most suitable to use in prestressed bridge girders (Arockiasamy et al., 1995; Uomoto et al., 1995; 1999; Rostásy, 1996; Sen et al., 1998; Saadatmanesh and Tannous, 1999).

The objectives of present paper are, (i) to indicate possible applications of CFRP prestressed members, (ii) to compare flexural behaviour of steel and CFRP prestressed beams and, (iii) to consider further research needs.

2. CHARACTERISTICS OF NON-METALLIC (FRP) REINFORCEMENTS

FRPs have tensile strengths of between 700 to 3500 N/mm², modulus of elasticity from 38000 to 300000 N/mm² and a failure strain range of 0.8 to 4.0 %. FRPs show no yielding and behave linearly elastic up to failure with brittle rupture. Stress-strain diagrams of commercial FRP bars are indicated in *Fig. 1*. Mechanical properties in the transverse direction are minor compared to those of the longitudinal direction due to the influence of resin matrix.

The short-term characteristics of FRPs can be more or less easily determined, however, the long-term properties may require specific considerations. The long-term characteristics of FRP reinforcement can be different from those of ordinary steel reinforcement due to not only the different materials but also the composite behaviour: time dependent phenomena can take place within the material phases (fibre, matrix) and on

Fig. 1 Qualitative stress-strain diagrams of FRPs (Balázs – Borosnyói, 2000)

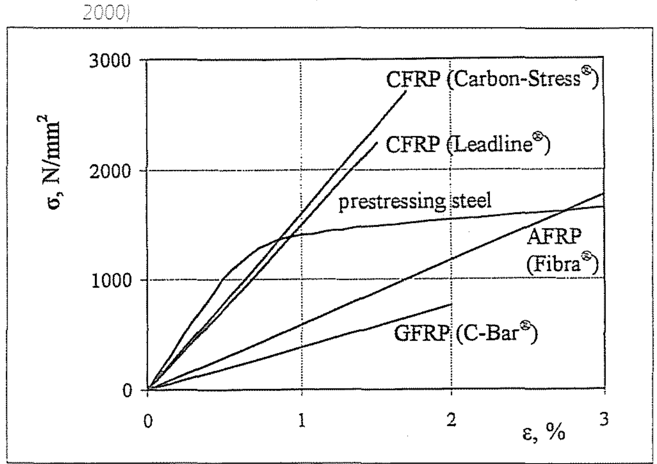


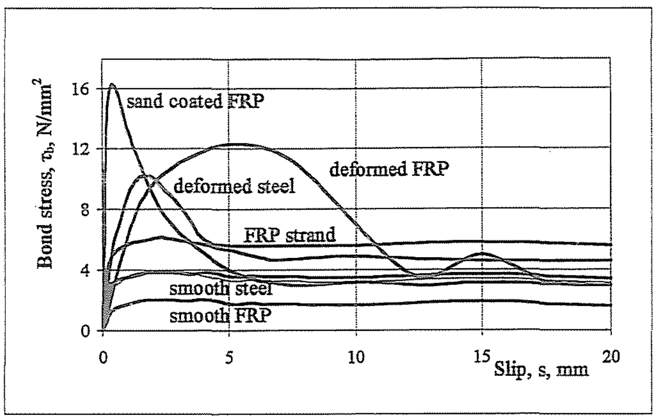
Table 1 Long term properties of FRPs (Balázs – Borosnyói, 2002)

	GFRP	AFRP	CFRP	prestressing steel
Creep $\Delta\varepsilon$ [‰] ($\sigma = 0.8f_{fu}$; $t = 1000$ h)	3 – 10	1.5 – 10	< 0.1	2.5 – 6.0
Relaxation ρ_{1000} [%] ($\sigma = 0.8f_{fu}$)	1.8 – 2.0	5.0 – 10.0	0.5 – 1.0	2.0 – 12.0
Long term tensile strength (estimated for 100 years)	$0.4f_{fu} - 0.7f_{fu}$	$0.5f_{fu} - 0.7f_{fu}$	$> 0.9f_{fu}$	–

the interface (bond, delamination). Generally, FRPs show better creep, relaxation and fatigue behaviour than steel prestressing tendons (Machida, 1993; 1997; Rostásy, 1996; Saadatmanesh-Tannous, 1999; Uomoto et al., 1995). However, the influence of environmental effects on the long-term properties needs further experimental investigations. The available data on long-term properties of FRPs are given in *Table 1*.

FRP reinforcement is usually produced using the pultrusion process. Rods with an almost smooth surfaces are not suitable for concrete structures due to the lack of an adequate bond. Therefore, surface treatments (such as spiral fibre winding, indentations, periodic ribs, stranded or braided shapes or sanded surfaces) are needed to improve bond characteristics. These treatments can increase bond strength of FRP reinforcement even more than that of steel tendons. Qualitative bond-slip diagrams are indicated in *Fig. 2*. During pull-out bond failure the outer layers of FRP reinforcement can be damaged which never occurs in the case of steel tendons or reinforcing bars. This difference may influence structural behaviour.

Fig. 2 Qualitative bond-slip diagrams of FRPs (Balázs – Borosnyói, 2002)



Environmental effects can considerably influence the long-term characteristics of FRPs. Liquids (water, alkali and salt solutions) can diffuse into resins of FRPs that can cause a decrease in mechanical characteristics.

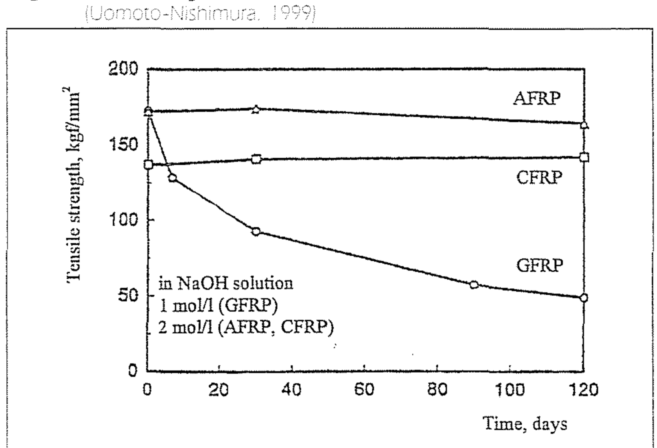
Concrete is highly alkaline due to the high calcium hydroxide content of hardened cement stone (pH 12.5 to 14), and needs special attention with regard to the durability of FRP reinforcement. Carbon fibres cannot absorb liquids and are resistant to acid, alkali and organic solvents and therefore, they do not show considerable deterioration in any kind of harsh environment (Machida, 1993; 1997; Tokyo Rope, 1993). The deterioration of glass fibres in an alkaline environment is well known (Wolff-Miesseler, 1993) and consequently the function of resins is of great importance in protecting glass fibres. Experimental studies of GFRP reinforcement embedded in concrete or under accelerated ageing tests in strong alkaline solutions have demonstrated that glass fibres show significant degradation due to alkalis, almost independently of the resin (Uomoto et al., 1995; 1999). The decrease in tensile capacity can be in the range of 30 to 100 percent according to saturation and acting time. The best protection can be ensured with vinyl ester resin. Results of accelerated tests usually show more deterioration than tests with embedded reinforcement. The rate of deterioration of glass fibres in an alkaline environment is highly dependent on the type of fibres. Aramid fibres may also suffer deterioration in alkaline an environment, generally to a less degree than glass fibres but this may depend on the actual fibre product (Uomoto et al., 1995; 1999). The decrease in tensile capacity can be 25 to 50 percent (Rostásy 1997) and an alkaline environment can deteriorate links between molecules of resins as well.

Similarly to the resistance against water absorption the alkaline resistance of vinyl ester resins is the best while epoxy and polyester resins can give sufficient and poor resistance, respectively (Machida, 1993; 1997). Effect of highly alkaline solutions on the tensile strength of FRPs can be seen in *Fig. 3* (Uomoto et al., 1995; 1999). Superior properties of CFRP can be clearly observed.

The contact between water and FRP reinforcement is evident in fresh concrete and material changes associated with water are usually produced in the resins. Water can absorb into polymer chains and can create weak chemical reactions causing considerable changes in characteristics (e.g. strength, Young's modulus, bond). These effects are mostly reversible, however swelling of resin can cause micro-cracks in the matrix that can initiate fibre debonding and higher permeability. Vinyl ester resins show the best resistance to water absorption, epoxy resins can provide sufficient resistance, while polyester resins usually have poor performance (Machida, 1993; 1997). Concerning fibres, carbon and glass fibres cannot absorb water, contrary to aramid fibres (Uomoto et al., 1995; 1999). Water absorption of aramid fibres causes a reversible decrease in tensile strength, Young's modulus or increase in relaxation and irreversible decrease in fatigue strength. The decrease in characteristics of AFRP due to water absorption may reach 15 to 25 percent (Gerritse, 1993). With regard to swelling of AFRP reinforcement, bond cracking can be induced by wet/dry cycles (e.g. in splash zones of marine structures) that causes deterioration. The swelling of CFRP reinforcement is negligible and attributed only to the swelling of the resin.

In many civil engineering applications reinforced concrete members are subjected to a high number of freeze-thaw cycles (mostly combined with water and chloride ion penetration into the concrete). However, experimental data on the influence of

Fig. 3 Tensile strength of FRPs in alkaline solution (Uomoto-Nishimura, 1999)



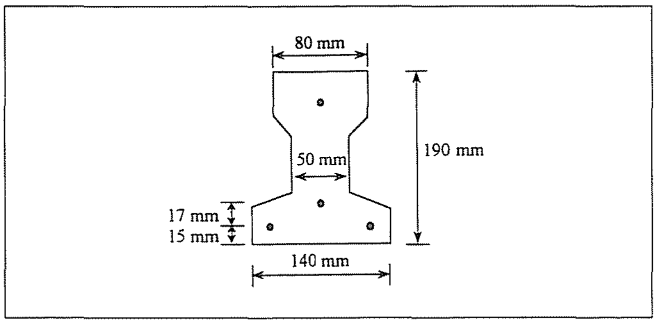


Fig. 4 Cross-section of test beams with the maximum number of tendons Symbol: either CFRP or steel prestressing wires

such effects on the durability of FRP reinforcements is very limited. Due to freezing and thawing cycles (combined with water and chloride ion diffusion) the degradation of fibres, resin and the interfacial bond is possible. According to micro cracking of concrete under freezing and thawing cycles the bond between concrete and FRP can be also influenced.

3. TESTS ON CFRP PRESTRESSED MEMBERS

An experimental study has been carried out (at the Budapest University of Technology and Economics, Faculty of Civil Engineering) on prestressed concrete beams pretensioned either with CFRP or steel tendons.

3.1 Specimens

Test beams had an I cross-section with a relatively thin web and did not contain any other longitudinal reinforcement but the prestressing tendons (*Fig. 4*). Beams were cast with 1, 2 or 4 prestressing tendons, respectively. Beams had the same cross section, with clear concrete cover of 12 mm. The diameter of the CFRP wires was 5.0 mm and that of steel wires 5.34 mm. Each prestressing tendon was pretensioned to 26.3 kN load which means 1340 N/mm^2 initial prestress in the CFRP tendons ând 1174 N/mm^2 in the steel wires.

3.2 Material properties

In these present tests steam cured normal weight concrete was used. Water-cement ratio was 0.35 with CEM II/A-V32.5R and ordinary sand and gravel aggregate. The mean compressive cube strength of the concrete at 28 days was 65 N/mm^2.

In the CFRP prestressed members CFRP wires of 5.0 mm with a sand coated surface were used. These mechanical properties are given by the producer (NEDRI, NL):

Tensile strength, f_{fu}	2700 N/mm^2
Ultimate strain, ε_{fu}	1.7 %
Mod. of elasticity, E_f	158.8 kN/mm^2
Relaxation loss (under $0.7f_{fu}$)	1.0 %
Poisson's ratio	0.3
Coefficient of thermal expansion, α_f	
-longitudinal	0.2×10^{-6} m/m/°C
-transverse	23×10^{-6} m/m/°C

In steel prestressed members, cold drawn stabilised prestressing steel wires with a diameter of 5.34 mm and indented surface were used. Their standard properties were:

Tensile strength, f_{pu}	1770 N/mm^2
Nom. yield stress, $f_{p,0.1}$	1450 N/mm^2
Ultimate strain, ε_{pu}	3.5 %
Mod. of elasticity, E_p	195 kN/mm^2
Relaxation loss (under $0.7f_{pu}$)	2.5 %
Coefficient of thermal expansion, α_p	
-longitudinal and transverse	10×10^{-6} m/m/°C

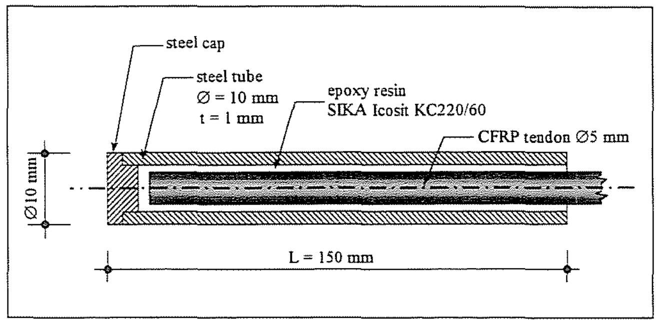


Fig. 5 Anchoring device developed for present tests

3.3 Preparation and testing of specimens

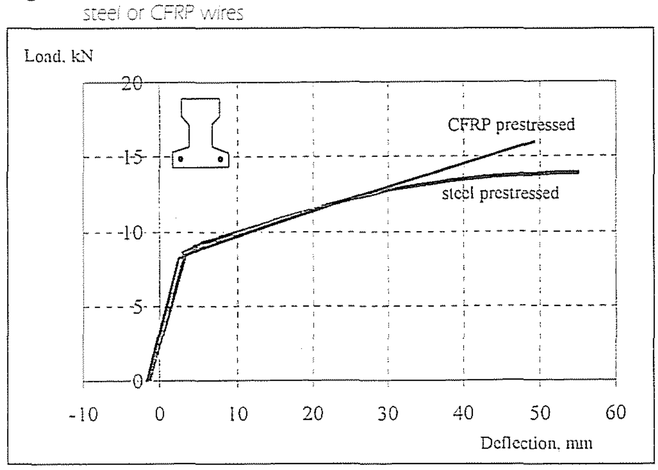
the application of prestressing force on the CFRP wires required special attention owing to their relatively low transverse strength compared to their high axial strength. The CFRP wires used for the described tests were not supplied with anchorages for prestressing. Therefore, the following technique was developed for prestressing the CFRP wires. Steel tubes (*Fig. 5*) were glued to both ends of the CFRP wires by Sika Icosit KC220/60 epoxy resin and a prestressing force was applied by a conventional prestressing jack suitable to prestress ∅ 12.8 (1/2 in) strands. Specimens were prepared by Pfleiderer Precast Company, Lábatlan, Hungary. Six beams were cast in each turn separated by hard rubber profiles. After steam curing the prestressing force was released by flame cutting of steel wires and sawing of CFRP tendons.

Our first observation after steam curing and tension release was that neither temperature difference during steam curing, nor the sudden release of the prestressing force initiated longitudinal cracks in any beam.

Beams were tested with a clear span of 3.0 m in four point bending. Deflections were continuously registered with an LVDT at mid span. All crack widths were measured at the level of the lowest prestressing tendons by a hand microscope at each load step.

4. TEST RESULTS AND DISCUSSION

Fig. 6 Load vs. deflection responses of beams prestressed with two steel or CFRP wires



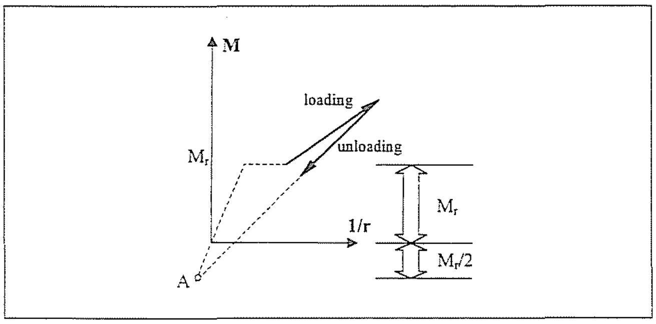


Fig. 7 Definition of pivot point according to CEB-FIP Model Code 1990, Clause 3.6, p. 109, Fig. 3.6.2.

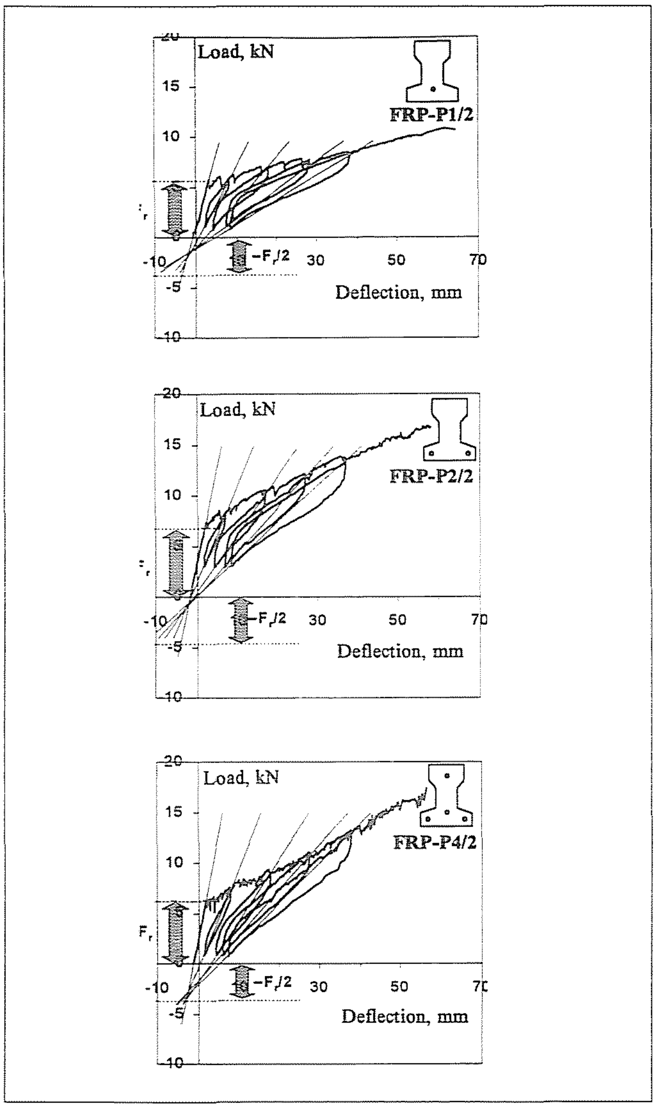


Fig. 8 Load vs. deflection responses of beams prestressed with CFRP wires

Typical load-deflection responses are presented in *Fig. 6* for beams prestressed either with 2 steel or with 2 CFRP wires. Load-deflection responses of both steel and CFRP prestressed members were linear before reaching cracking load. Exceeding the cracking load, load-deflection diagrams of CFRP prestressed members remained linear with lower stiffness on the contrary to steel prestressed members, which showed slight non-linearity. Bilinear load-deflection behaviour was due to the linear elastic behaviour of CFRP prestressing tendons.

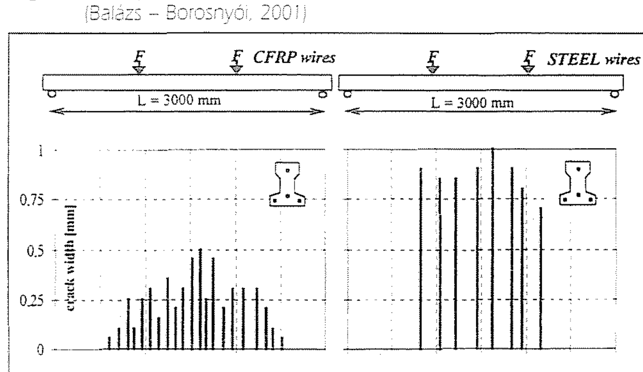
In general, the cracking load and load-carrying capacity of CFRP prestressed members could be calculated based on conventional theories of prestressed concrete. In the experimental program measured cracking load as well as the mid span deflection at failure did not depend on the type of prestressing material. However, the mode of failure changed from rupture of tendons to crushing of concrete due to the increase in reinforcement ratio. Most of the beams failed by shear failure because of the lack of any shear reinforcement in the members. While all of the CFRP wires ruptured during failure, steel wires kept the two parts of the failed beam together. So the dowel action of CFRP wires was found to be negligible.

Repeated loading-unloading cycles for plain concrete produce hysteresis loops (CEB, 1996). Unloading curves are not identical either with the loading envelope curve nor the reloading curves because only the elastic strain recovers during unloading. The slope of the reloading curve (i.e. the secant modulus of elasticity at a given strain) decreases with increasing strain. Lines of decreasing moduli form a pivot point that takes place on the line of the initial tangent. Pivot behaviour can be also observed on moment vs. curvature or load vs. deflection responses of flexural members under repeated loads. For which parameters have influence on the position of pivot point only limited data are available. According to the CEB-FIP Model Code 1990, the pivot point of decreasing loading-unloading moduli of moment-curvature response of non-prestressed flexural members under simple bending takes place on the initial line of moment-curvature diagram (see point A in *Fig. 7*). MC90 gives the moment co-ordinate of the pivot point as one-half of the cracking moment. Direct influences of load level, reinforcement ratio or geometrical parameters are not given (MC90 Clause 3.6, p. 109, Fig. 3.6.2).

In our experimental loading-unloading moduli of load-deflection responses have the described monotonically decreasing tendency (*Fig. 8*) with definite pivot point of moduli that takes place on the line of the initial modulus. The location of the pivot point seems to be dependent not only on the reinforcement ratio (thus prestressing force), but the prestressing material as well. In the case of members prestressed with one or two tendons (low reinforcement ratios, $\rho < 1\%$), pivot points of CFRP prestressed members take place *over* that of steel prestressed members. On the other hand, in the case of members prestressed with four tendons the pivot point to the CFRP prestressed member takes place *under* that of the steel prestressed member. Since the geometry of all test beams was equal, the effect of the size of elements could not be studied. It can be also realised that the proposal of MC 90 gives an upper boundary for the position of pivot points to all cases. The location of the pivot point cannot be fixed with only one parameter, i.e. the cracking load. The influence of other parameters (e.g. reinforcement ratio, Young's modulus of prestressing material, etc.) has to be taken into consideration as well.

The CFRP prestressed members had a higher number of cracks, leading to smaller average crack spacing and smaller average crack width due to the sanded surface of tendons. This phenomenon was the most remarkable on members prestressed with four tendons. *Fig. 9* shows the crack pattern of beams

Fig. 9 Crack pattern for test members prestressed with four tendons (Balázs – Borosnyói, 2001)



prestressed either with four CFRP or with four steel wires close to failure (mid span deflection = clear span/75).

The results of cracking and deflection of these members are discussed in Refs. Balázs – Borosnyói (2001) and Balázs et al. (2000), respectively. Detailed analysis of results is presented in Ref. Borosnyói (2002).

5. OPEN QUESTIONS

Flexural and shear behaviour of concrete members reinforced or prestressed with non-metallic reinforcement are fields requiring a lot of research because the many questions which so far are unanswered. These are summarised in the following.

The linear elastic-brittle behaviour of FRPs can be considered as a disadvantage. FRP reinforced members do not have plastic deformation capacity due to the lack of any yielding of the reinforcement. Therefore, the ductility of structural elements can not be ensured in spite of the appropriate moment capacity (here *ductility* means the capability of plastic deformations before failure without reduction in load bearing capacity). Due to the linear elastic behaviour of reinforcement the following design considerations are advised.

The moment of resistance of FRP reinforced cross-sections can be determined using the conventional theory of composite cross-sections, i.e. analysis of internal forces taking into account the hypothesis of plane cross-sections. Failure has to occur with crushing of the compressed concrete zone – rupture of reinforcement has to be avoided due to its sudden energy release. Concrete crushing is reached if the reinforcement ratio is *more* than the so-called balanced reinforcement ratio (whenever reinforcement rupture and concrete crushing occurs at the same time).

Plastic redistribution of bending moments can not be taken into account in FRP reinforced members (of course, reduction in stiffness due to cracks has to be taken into account).

As FRPs are linearly elastic and brittle materials ordinary definitions of ductility and ductility indices can not be applied (definition of ductile behaviour requires plastic behaviour of structural materials). However, due to the lower Young's moduli of FRPs, the deformability of these structures is considerable. A new definition of ductility index is required.

Several proposals exist with the subject of quasi-ductile behaviour of FRP reinforcement or FRP reinforced members. Quasi-ductile behaviour of reinforcement can be reached by applying *hybrid fibres or special braided configuration of bars* (Tamužs et al., 1994; Tamužs – Tepfers, 1995; Somboonsong et al., 1998). Concerning the stress-strain response of a special braided hybrid FRP reinforcement, *Fig. 10* can be introduced (Somboonsong et al., 1998). In the diagram the initial linear elastic part is followed by breaking of fibre bundles of limited strain capacity (part a) and the re-elongation and load bearing of fibre bundles of lower Young's modulus (part b).

Another proposal for quasi-ductile behaviour is the use of partially bonded (staggered bonded and unbonded lengths) reinforcement (Lees – Burgoyne, 1996).

Minimum concrete cover in reinforced concrete members is required mainly for durability of reinforcement. However, in the case of FRPs this requirement can be neglected because the reinforcement is non-corrosive although there are other important parameters that have to be taken into account such as fire protection, cover against splitting in the anchorage zone (due to high bond forces) or different coefficients of thermal expansion. For these reasons the concrete cover of FRP reinforced members can not be reduced too much. It is advised to

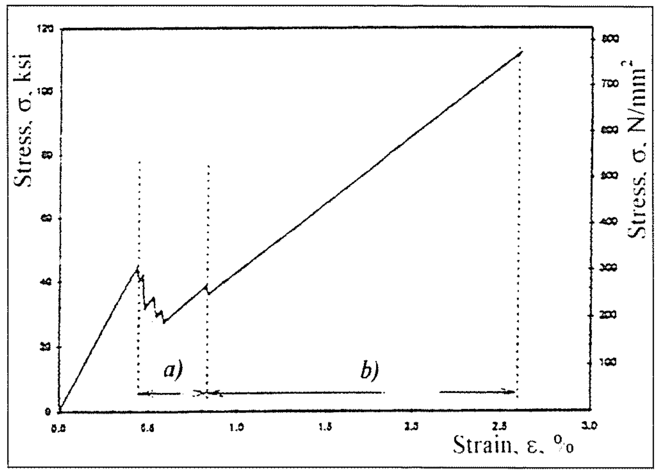


Fig. 10 Stress-strain response of braided hybrid FRP bar (Somboonsong et al., 1999)
(a) breaking of fibre bundles of limited strain capacity
(b) re-elongation and load bearing of fibre bundles of lower Young's modulus

keep concrete cover requirements of ordinary steel reinforced members before detailed studies are carried out.

Because of the relatively low relaxation and creep as well as lower Young's modulus of FRP tendons, the majority of losses of prestressing force are lower than that of steel prestressed members. However, due to the lower Young's modulus of FRP tendons, a greater travel length of the prestressing jack is needed to develop an adequate prestressing force. On the other hand, FRPs have much lower strength in the transverse direction than longitudinally and thus the prestressing of FRP tendons can not be carried out by using conventional anchoring devices. There are several anchoring systems developed for non-metallic prestressing tendons, but they have a restricted use.

The most economic application of FRP reinforcement can be in bridge engineering, especially for prestressed highway bridge girders. Cyclic and long term loads are of great importance for bridge girders but at the moment limited data is available for the structural behaviour of FRP reinforced or prestressed members subjected to cyclic or long term loads (e.g. Abdelrahman et al., 1995; Braimah et al., 1999).

6. CONCLUSIONS

Non-metallic reinforcement is a potential alternative to steel reinforcement. Furthermore, research on non-metallic reinforcement was stimulated by the observed deterioration in concrete members reinforced with steel.

Most of the non-metallic reinforcements show high strength, high fatigue strength, low relaxation and low creep in addition to corrosion insensitivity.

With regard to the experimental study which was carried out at the Budapest University of Technology and Economics, Faculty of Civil Engineering on concrete beams prestressed either with steel wires or with CFRP wires, our test results indicated:

- The load vs. deflection relationships of CFRP prestressed members are practically bilinear due to the linear elastic behaviour of CFRP prestressing tendons
- The cracking load and load-carrying capacity of CFRP prestressed members could be calculated based on conventional theories of prestressed concrete
- That dowel action of CFRP wires seemed to be negligible
- The location of the pivot point of load-deflection responses

cannot be fixed with only one parameter, i.e. the cracking load

- That due to the sanded surface of tendons, the CFRP prestressed members have a higher number of cracks, leading to smaller average crack spacing and smaller average crack width.

7. NOTATIONS

E_f modulus of elasticity of FRP tendon
E_p modulus of elasticity of steel prestressing tendon
f_{fu} tensile strength of FRP tendon
$f_{p,0.1}$ nominal yield stress of steel prestressing tendon
f_{pu} tensile strength of steel prestressing tendon
F_r cracking load
L span
M_r cracking moment
$1/r$ curvature
α_f coefficient of thermal expansion of FRP tendon
α_p coefficient of thermal expansion of steel prestressing tendon
ε strain
ε_{fu} ultimate tensile strain of FRP tendon
ε_{pu} ultimate tensile strain of steel prestressing tendon
σ stress

8. ACKNOWLEDGEMENTS

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CONCRETE WITH HIGH STEEL FIBRE DOSAGES – AN EFFECTIVE CEMENTITIOUS MATERIAL FOR STRUCTURAL REPAIR


Zoltán Orbán – Prof. György L. Balázs

The paper presents cost-effective and long-term solutions for infrastructure repair using High Performance Fibre Reinforced Concrete (HPFRC) with high steel fibre dosages. As a repair material, HPFRC has superior characteristics (strength, toughness, impact and fatigue resistance, crack control, impermeability, etc.) over conventional cement composites forming an ideal material suited for use in the rehabilitation of deteriorated reinforced concrete structures.

Experiments and numerical analyses on flexural members have shown that an optimal combination of material ductility, strength and cross sectional arrangement by appropriate use of HPFRC repair materials yields improved properties with regard to resistance, energy absorption, deformation capacity and structural ductility.

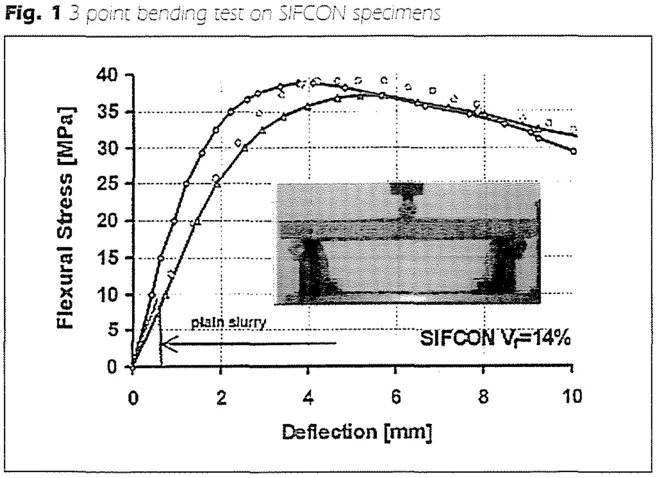
Keywords: HPFRC, SIFCON, rehabilitation, resistance, ductility

1. INTRODUCTION

There is an increasing demand for durable, reliable and economic solutions in structural rehabilitation works which calls for repair materials that answer ever-growing requirements. HPFRC opens a new horizon for the rehabilitation of concrete and reinforced concrete structures thanks to the extremely high potential in tailoring its properties to specific structural needs.

Several structural elements exist, where ductility is an essential requirement. In such elements the use of cementitious repair materials with a relatively low level of material ductility can lead to an early loss of bond accompanied by a brittle-like structural behaviour. An appropriate way of enhancing material ductility is the addition of fibres to the cementitious matrix which results in a more ductile material called High Performance Fibre Reinforced Concrete. HPFRC has superior characteristics (regarding strength, toughness, impact and fatigue resistance, crack control, impermeability, bond performance, etc.) over conventional cement composites forming an ideally suited material for the rehabilitation of deteriorated structures. Besides these many-sided benefits the higher volume of fibres usually entails significant extra costs that apparently discourages structural applications. A number of research efforts are therefore directed towards finding an optimal compromise between cost and performance. The expense of improved performance provided by fibre addition is generally compensated for by the reduction in maintenance costs.

Fig. 1 3 point bending test on SIFCON specimens



A computer-based calculation model has been developed in order to describe the non-linear flexural behaviour of RC members combined with HPFRC repair layers. The model is appropriate for demonstrating the effect of material non-linearity, strength, ductility, etc. on flexural response even in post-peak stages. Arbitrary chosen cross-sections with different levels of conventional and fibre reinforcement were analysed with respect to ultimate moment capacity, inclination for brittle failure and sectional ductility.

2. EFFECTIVENESS OF HPFRC REPAIR SYSTEMS

A common technique for enhancing the flexural capacity of reinforced concrete flexural members is the casting of a new concrete or reinforced concrete layer either on the compressive or tensile side of the member. This technique is widely used in the repair of damaged elements. The behaviour of the new composite element is expected to be superior to the sum of the behaviour of the two separate elements. To achieve this purpose the two individual members must be reliably connected at the interface level, resulting maximised monolithic behaviour. Thus, the interaction at the interface level is the critical parameter in the behaviour of the new composite member. In case of HPFRC repair layers, however, the deviation from the monolithic flexural behaviour is much smaller than for solutions with conventional concrete or Fibre Reinforced Concrete (FRC).

The amount of fibre added to the concrete has a significant effect on the stress-strain response especially in tension. The main difference between ordinary fibre reinforced concrete (FRC, fibre volume fraction: $V_f \leq 2\%$) and HPFRC (V_f is usually above 4%) is that *HPFRC exhibits a substantial strain-*

hardening type of behaviour with a considerably higher failure strain capacity leading to a large improvement in both strength and toughness. Fibres in the repair layer have the ability to stabilize crack growth and transform the fast, brittle failure type of concrete into a slow, stable fracture process with post cracking ductility and high energy absorption capacity prior to failure. In conventional FRC repair layers, first cracking is accompanied by the development of a localised fracture process zone. In this process zone the bridging fibers can partially transfer the stress across the crack. However the magnitude of the transferred stresses decreases as the crack enlarges. In contrast, using a strain hardening HPFRC material, first cracking is accompanied by a strain concentration at the mouth of the crack. Due to the stress transfer capability of the reinforcing fibres in a *strain hardening material*, stress redistribution will occur so that localised fracture will be delayed. Consequently, an expanded zone of cracking will develop prior to fracture resulting in more effective and durable repair layers with extremely high bond performance. Furthermore, the presence of fibres in high volume significantly improves the bond strength between the exposed reinforcement of the old structure and the repair material, and consequently has a major contribution to a better anchorage capacity. Another important result of the multiple cracking mechanism is the lowered permeability of the material. Furthermore the thermal expansion coefficient and the modulus of elasticity are similar to those of conventional concrete, consequently HPFRC represents a repair system that is highly compatible with the substrate concrete.

There is a practical limit for fibre content in HPFRC beyond which proper mixing of the fibres is not possible with conventional procedures. By the application of SIFCON (Slurry Infiltrated Fibre Concrete) the fibres are pre-placed and the fibre network is then infiltrated with a low viscosity cement based slurry. The achieved material has extremely high ductility and high crack controlling capacity with a fibre content even above 10-20 V%. *Fig 1.* shows the results of bending tests that have been carried out on SIFCON flexural specimens under monotonic loading (*Orbán, 2002*). The specimen dimensions were 350*100*30mm and was subjected to 3 point bending over a 300mm span with a 0.2mm/min loading rate. The applied slurry for the infiltration of the fibre bed had a compressive strength of 60 MPa and a flexural strength of 10 MPa. The viscosity of the slurry was adjusted so as to be able to achieve proper infiltration within 3 minutes of vibration. The fibre bed consisted of V_f = 14% thin hooked fibres with an aspect ratio of 60 (KSF 30/0.5: l = 30mm, d = 0.5mm). The load-deflection curves clearly demonstrate the strain hardening response of SIFCON specimens and the beneficial effect of fibres on the load-carrying capacity. The ductility of the specimens is provided by the multiple cracking phenomena developing in the plastic hinges (*Fig. 2*).

3. STRENGTH AND DUCTILITY OF REPAIRED STRUCTURES

The deterioration of reinforced concrete strucures typically involves cracking of concrete, spalling of cover and corrosion of the reinforcement. In general, corrosion of the reinforcement is of most concern since the associated reduction in area of the steel and loss of bond will, in time, lead to loss of load bearing capacity, loss of structural ductility and unserviceability. Maintenance measures should ensure safe and

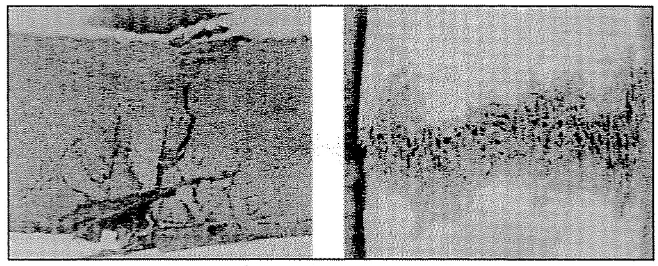


Fig. 2 Multiple cracking of SIFCON specimens in the plastic hinges

durable service at the same time. Thus repair solutions have to verify that the strengthened structural elements possess both the required resistance and the corresponding ductility.

3.1 Ductility versus safety

According to recent practice assessment of structural safety is generally based on the load-bearing capacity: safety refers to the probability that the structural loads will not exceed the structure's capacity to resist them. In case of an insufficient degree of safety, strengthening operations are completed in order to provide an acceptable risk level by enhancing the load-bearing capacity. These measures often commit the error of concentrating only on the increase in resistance. Since risk is normally defined as a combination of the likelihood of occurrence of a defined hazard and the magnitude of its consequences, a similarly effective way of reducing the risk is the mitigation of the consequences. Structural robustness is a measure of the structure's ability to mobilise alternative load paths and bridge over damaged parts in the event of overloading or under exceptional circumstances. The failure of a structure is usually preceded by the loss of robustness as a result of material deterioration or local damage making the structure susceptible to a mechanism-like collapse. A key element of structural robustness is the structural ductility that implies the structure's ability to undergo inelastic deformations and absorb large amounts of energy without substantial reduction in its resistance. In case of a mechanism-like failure structural ductility relies on the deformation capacity of plastic hinges or yield lines. Thus repair techniques should focus more on the restoration of ductility rather than strength increase only (*Orbán, 2000*).

Ductile behaviour of repaired flexural members is essential for ensuring satisfactory behaviour in the ultimate limit states. This plastic deformation capacity is indispensable for example to provide warning signs of impending collapse by the development of large deformations, resistance against imposed deformations (e.g. due to temperature, support settlement, shrinkage, creep, etc.), ability to withstand unforeseen local impact and accidental loading without collapse and for appropriate energy dissipation capability under cyclic loading.

3.2 Numerical analysis on flexural members

Different plain concrete and reinforced concrete rectangular beam cross sections combined with FRC and HPFRC repair layers were analysed in the present study with respect to the ultimate moment capacity inclination to brittle failure after the first cracking and sectional ductility. The arrangement of the beams and the varied parameters are shown in *Fig.3*. The

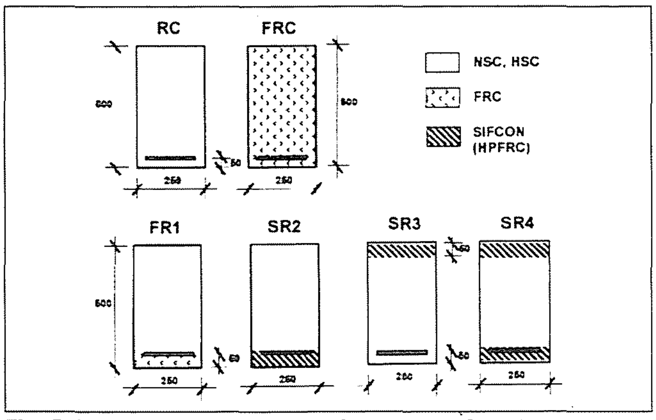


Fig. 3 Cross-sectional arrangements of the analysed flexural members

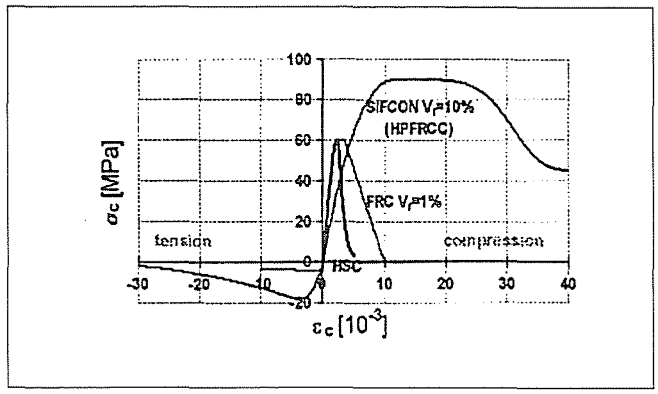
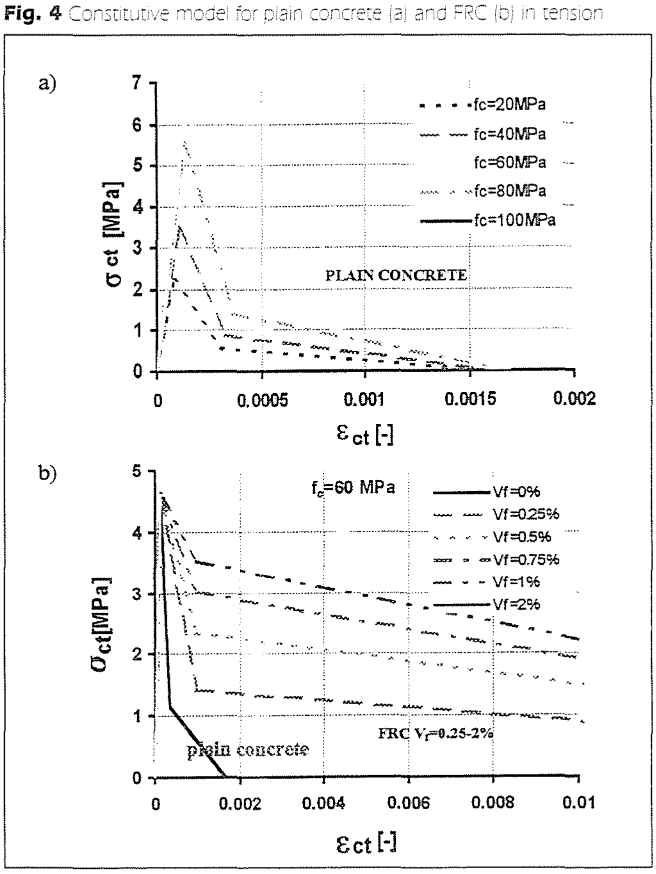


Fig. 5 Comparison of the stress-strain diagrams of plain concrete (HSC), FRC and HPFRC (SIFCON) with $f_{c,matrix}$=60 MPa

response of the members were compared to that of reinforced concrete without and with fibre reinforcement. The calculations were carried out by means of a computer program based on the moment (M)–curvature (k) diagrams of the cross-sections. The method is suitable for taking into account the whole stress-strain response of concrete both in compression and tension. The constitutive material models used in the calculations are seen in *Figs. 4.* and *5.*

The applied diagram for plain concrete in compression was adapted from the literature (*Collins, 1993*). A combined model was used for fibre reinforced concrete, where the compression side of the model for plain concrete was extended by a more ductile descending branch to reflect the higher strain capacity. For the tensile side of the diagrams the stress-crack opening (s-w) relationship in the softening stage determined by fracture energy values (G_F) was transformed into stress-relative strain (s-e) formula by a "fictious damage zone model" (*Orbán, 2002*). For SIFCON (HPFRC) a fixed value of a high fibre content (V_f=10%) was applied. The stress-strain diagram proposed by (*Naaman et al, 1995*) was used in the calculations with a small modification and with model-parameters defined by experiment. For reinforcing steel an elastic-plastic s-e diagram was used similarly to *EUROCODE-2, 1992* while ignoring the strain hardening effect. The characteristic value of yield strength and ultimate strain capacity was f_y=500 MPa and ε_{su}=2.5%, respectively. The effect of confinement and tension stiffening have been neglected in the calculations and there was no bond failure assumed between the HPFRC repair overlays and the old concrete surface. All geometrical and strength characteristics were taken into account with their actual values. As a fundamental assumption the Navier-hypothesis was accepted to be valid.

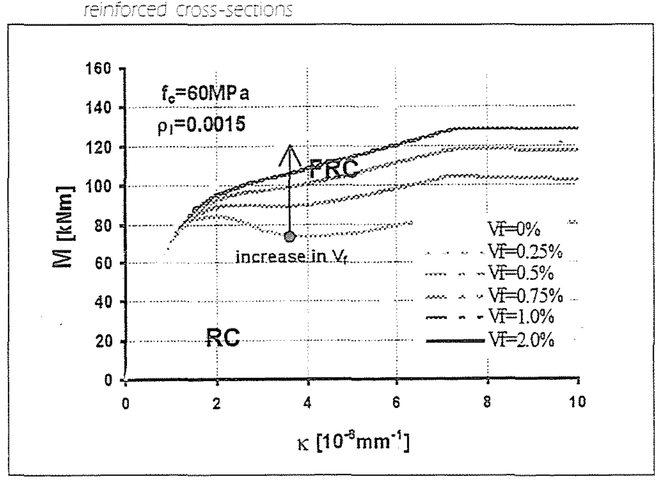
Fig. 4 Constitutive model for plain concrete (a) and FRC (b) in tension



3.3 Avoiding brittle flexural failure by HPFRC overlays

Brittle flexural failure of an under reinforced or deteriorated RC beam means that it fails before the reinforcement resistance capacity can be activated. The first bending crack runs into the compression zone with almost no warning or rotation of the beam critical cross-section. An important prerequisite for ductile failure of RC beams is the minimum reinforcement ratio requirement in order to provide stable behaviour after first cracking has occurred. As indicated in *Fig. 6.* the stabilised behaviour after first cracking can also be achieved by a concrete material with higher fracture energy. While increasing the amount of fibre reinforcement in the tensile segment of the cross-section the flexural response after first cracking is becoming more and more stabilised. Therefore HPFRC in the repair layer with high energy absorption capacity can provide this stability together with a substantial increase in load bearing.

Fig. 6 Effect of fibre addition on the flexural behaviour of under reinforced cross-sections



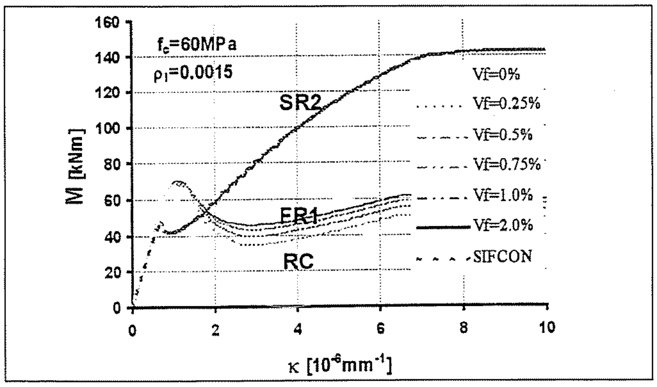


Fig. 7 The effect of FRC and HPFRC repair overlays on the flexural behaviour of under reinforced cross-sections

3.4 Increase in strength and ductility

Fibre addition in high dosages considerably enhances the moment capacity of under and normally reinforced cross-sections (Fig. 6., Fig. 7. and Fig. 8. (left)) and substantially increases sectional ductility in case of an over-reinforced member (Fig.8 (right)). Fig. 8. demonstrates the effect of HPFRC repair overlays on cross-sections having longitudinal reinforcement (r_l) of 3%. The presence of HPFRC in the compression zone of the flexural members increase strength and strain capacity of the compression zone, consequently the lever arm increases and yields to an overall increase in load-bearing capacity, energy absorption and curvature at maximum moment.

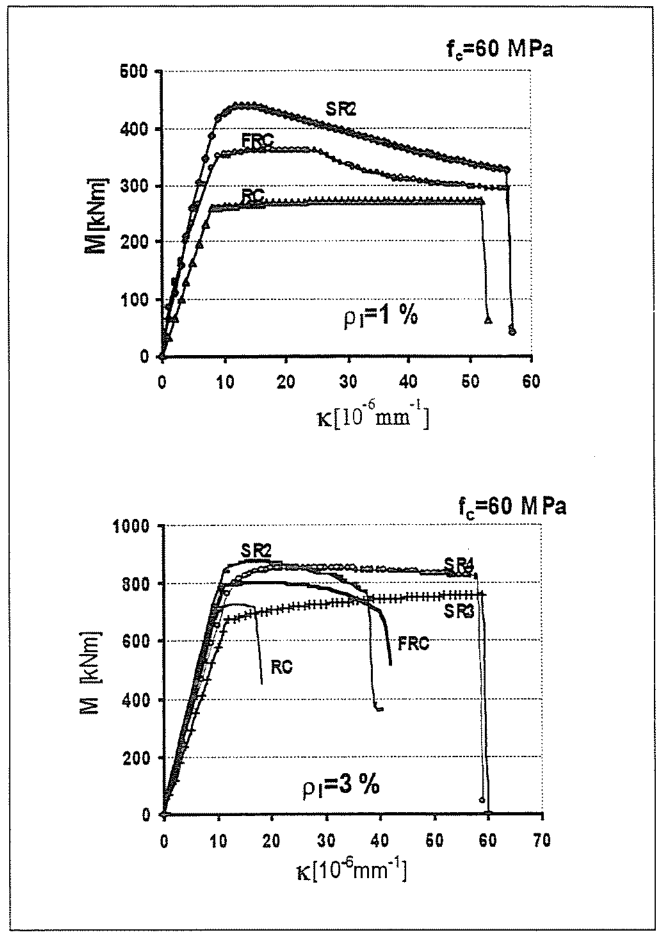


Fig. 8 Effect of FRC and HPFRC repair layers on the behaviour of normally reinforced (top) and over-reinforced (bottom) cross-sections

4. CONCLUSIONS

High Performance Concrete with high steel fibre dosages offers cost-effective and long term solutions to the rehabilitation of deteriorated flexural members and are suitable for achieving tensile and damage tolerant properties in repair systems. Even as a thin repair overlay it will substantially improve both mechanical (strength and ductility) and durability performance of a structure.

Analytical and experimental investigations have shown that HPFRC as a repair material can succesfully interact with existing structural elements, substantially increase flexural capacity, energy absorption capacity and ductility of reinforced concrete flexural elements.

5. ACKNOWLEDGEMENTS

The authors would like to offer their thanks to Techno-Wato Kft, SKW-MBT Hungária Kft. and Stabiment Hungária Kft. for their valuable assistance and co-operation by supplying basic constituent materials for the experiments, and greatly acknowledge the financial support of the Hungarian research Found (OTKA, Grant Nr. 032 525).

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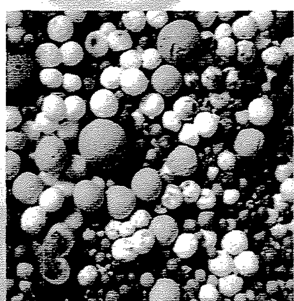
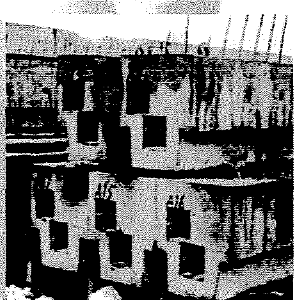
GEOFIL-BUBBLES

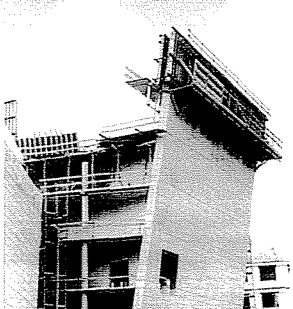
One of the greatest challenges and opportunities facing the industry is to reduce its burden on the environment while maintaining its profitability and growth. The GEOFIL Ltd accepted this challenge and is dedicated to finding creative solutions to today's toughest environmental problems in Hungary. Since its foundation (1991) the company has concentrated on environmental activities, e.g. environmental consulting and recycling.



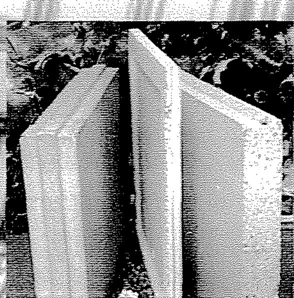
Under the name "*Geofil-Bubbles*" a pellet product from industrial waste material of high glass content is manufactured by recycling technology. This product has good bonding capability if embedded in cement, gypsum or resin matrix, and has primarily heat and sound insulating properties. The usage of lightweight aggregates produced out of waste materials has an enormous environmental advantage.



This product can be used for structural lightweight concrete as lightweight aggregate. Use of concrete with lower density and same load bearing capacity requires a smaller cross-section than in case of normal weight concrete. Some types of Geofil-Bubbles can not absorb water under pressure. Water absorption is less than 1 percentage. This property offers the possibility to use lightweight aggregate concrete (made of Geofil-Bubbles) as pumped concrete.



The most representative example is the comparison of structures (for example a bridge) made of normal weight concrete and lightweight concrete using Geofil-Bubbles as lightweight aggregate. The conclusion is that we can make a reduction of the slab thickness by 25-30%, so in the quantity of concrete by 25-30% and the amount of reinforcing steel by 30-35%. Further savings can be realized in the transportation and in the foundation costs.



Concrete with lower density has also better thermal insulating properties and the sound insulating properties are good enough. Prefabricated lightweight concrete elements can be used for sound insulating wall structures. Such walls can be constructed along highways, railways and around urban areas. It is light, easier to transport and lift into the final place.

Geofil Ltd. is open to make business both in Hungary and in foreign countries. The business relations can be licence selling, cooperation, etc.

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ISO 9001

TÜV CERT DIN EN ISO 14001

ÉPÍTŐ IPARI NÍVÓ DÍJ 2000

HÍDÉPÍTŐ

ÉPÍTŐIPARI MESTERDÍJ 1996

TÜV CERT DIN EN ISO 9001

HÍDÉPÍTŐ RÉSZVÉNYTÁRSASÁG

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Extension of the South-Pest Wastewater Treatment Plant

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

The initial purpose of establishing Hídépíto 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 "Hídépítők (Bridge Builders)" was to introduce and to make general the new construction technologies.

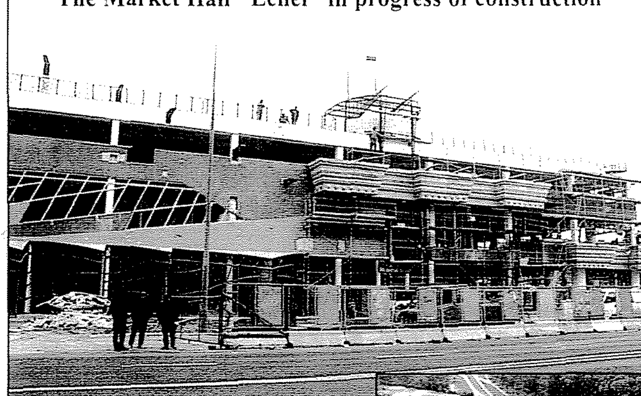
Even among these can be judged to outstanding the bridge construction by balanced cantilever method, the experts having participated in it were awarded the State Prize. By this technology were constructed five bridges in the region of rivers Körösök and this was applied at the flyover of Marx square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

The next big step was the introduction of the so-called cast-in-situ cantilever bridge construction method. This technology was applied for four bridges constructed over great streams, among them can be found the bridge with largest span (120 m) in Hungary made of stressed reinforced concrete, the road bridge over river Tisza at Szolnok.

An important result of the technologic development in the bridge construction was the introduction of the so-called incremental launching method. In the period from year 1989 up to now yet 22 bridges were constructed by this method, mainly on the base of the designs prepared by the Company's own Technical Department.

Among them distinguishes itself the viaduct made of stressed reinforced concrete in length of 1400 m on the Hungarian-Slovenian railway line, near the Slovenian State Border, constructed in one year using the incremental launching method,.

The Market Hall "Lehel" in progress of construction



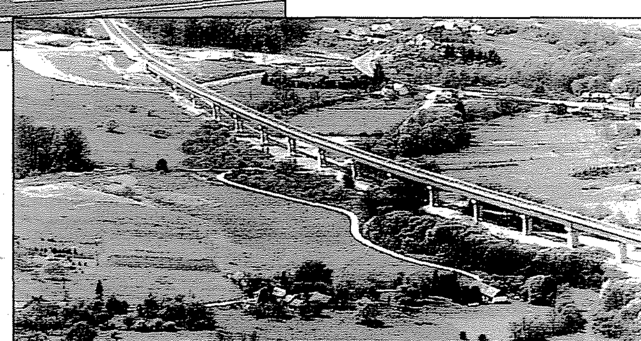
Beside the bridge construction, important results were achieved by the "Hídépítők (Bridge Builders)" in the field of foundation's technological development as well, in the introduction and general use of the bored piles with large diameter, of jet grouting and of CFA (Continuous Flight Auger) pile preparation,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

The Company was privatised (bought by the French Company GTMI) in year 1993. Following the multiple merger of the foreign interest parent Company, today Hídépítő Rt. belongs to the multinational Company "VINCI".

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

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

Nowadays, beside the high level activity in the field of bridge construction, the Company has extended its scope of activity by taking part in winding up the backwardness in infrastructure, construction of drinking water treatment plants, wastewater treatment plants, solid waste spoiling areas and sewers as well as by the introduction of the architectural engineering profile.



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

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

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