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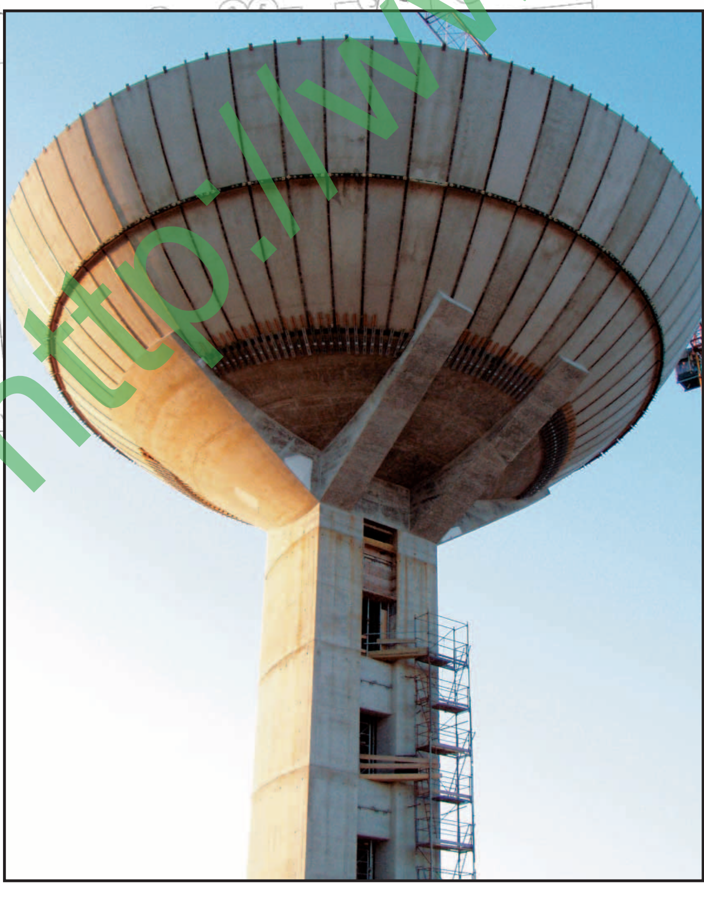

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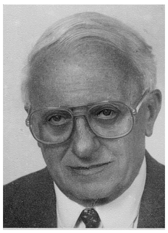
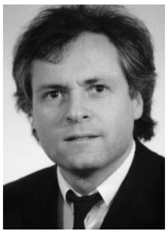
Cover:
Viaduct of Kőröshegy on M7 motorway in Hungary during constructions
Photo: János Barta
Water tower at Budafok, Hungary
Photo: Tamás Kovács

CONTENT

HISTORICAL CONNECTIONS BETWEEN DUBROVNIK AND HUNGARY - CONCRETE AND THE *fib* SYMPOSIUM

Prof. Géza Tassi – Prof György L. Balázs

1. INTRODUCTION

The congresses and symposia of *fib* are always noteworthy events for civil engineers and architects from all over the world dealing with concrete. This was also a tradition of CEB and FIP for our predecessors of *fib*. In discussing the benefits of such meetings (Balázs, Tassi, 2005), we pointed out that apart from the exchange of technical achievements, it is also our aim to familiarise ourselves with the historical and cultural links between the host city and country and that of foreign participants.

No doubt, speaking about Dubrovnik, Dalmatia and Croatia, Hungary has a distinguished position. It is our noble duty to revive these close historical connections, and to make use for our existing common technical activity.

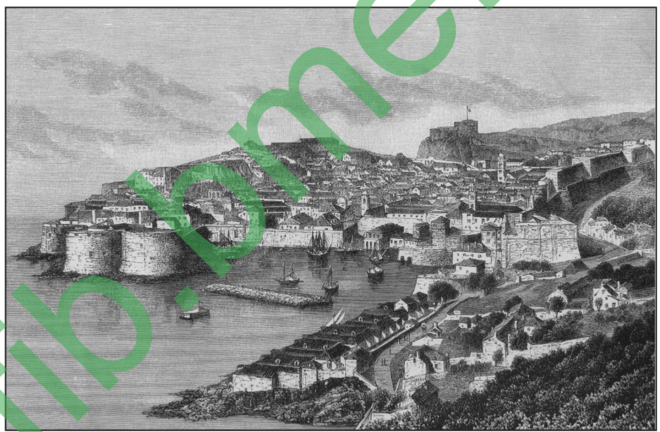


Fig. 1: Dubrovnik, engraving, 19^{th} century.

2. DUBROVNIK AND HUNGARY IN THE LIGHT OF HISTORY

2.1 The antecedents

Dubrovnik and Ragusa: Two names for this gem of the Adriatic coast known as Dalmatia, which in early chronicles was mentioned as „the home of the poor but brave men" (Gelcich 1887).

The name Ragusa is of Italian origin (or more probably Illyrian) relating to the meaning „wine", while the other name is the Croatian Dùbrōvnīk a composite word which may be "dùbrava", meaning „oak grove" (Kiss, 1988), Both names occurred in ancient times as in the early history of the city now known as Dubrovnik.

Dubrovnik was settled on a small rocky island in close proximity to the continent and its densely constructed houses were surrounded by well defensible, high stone walls and bastions *(Fig. 1)*.

This excellent coastal position destined Dubrovnik for maritime trade and due to the Italian part of its mentioned Janus-faced character; the records of living commercial activity are documented from the early centuries of the last millennium partly in Italian and partly in Latin. These statements of writers are probably not fully precise. Namely a part of population (at the beginning larger, later minor) was speaking in Latin art, and they are called in historical resources "Romanized inhabitants". They were probably of Illyrian origin. While the family names of the high dignitaries of the city are of Slavic origin the name of the city used in these documents is always referred to as Ragusa. E. g. the 18^{th} century's patrician family of the famous poet and Rector of the Ragusan Republic, Ivan Gundulić was referred to in these early records in the Italian form „de Gondola" (Gelcich, 1887).

2.2 The history till the middle of the 16^{th} century

Maritime trade in the Mediterranean region brought Ragusa early in history into a severe and enduring conflict with the Venetian Republic. Hostility concluded with the surrender of Ragusa in the beginning of the13^{th} century. The young Hungarian kingdom under the eighth Árpádian king, Saint Ladislas (László), due to his familiar connections became, after the death of the prince Zvonimir, also the king of both Croatia and the Dalmatian coast, but not of the part of it that belonged to the future Republic of Dubrovnik-Ragusa.

Coloman Beauclerc (Könyves Kálmán) King of Hungary occupied Croatia in 1096-97, and he was crowned as King of Croatia in 1102. The close connection between Croatia and Hungary was established for a very long time. He annexed also the important costal cities of Trau (Trogir), Zara (Zadar) and Spalato (Split) to the Hungarian Kingdom. These annexations were merely a formality and did not curb the hegemony of the Venetian Republic over Dalmatia (Pauler 1893). This situation was realised after 1205. Thus, the Venetian pressure upon Ragusa over maritime trade remained as strong as in previous times.

In the middle of the 13^{th} century the Hungarian Kingdom

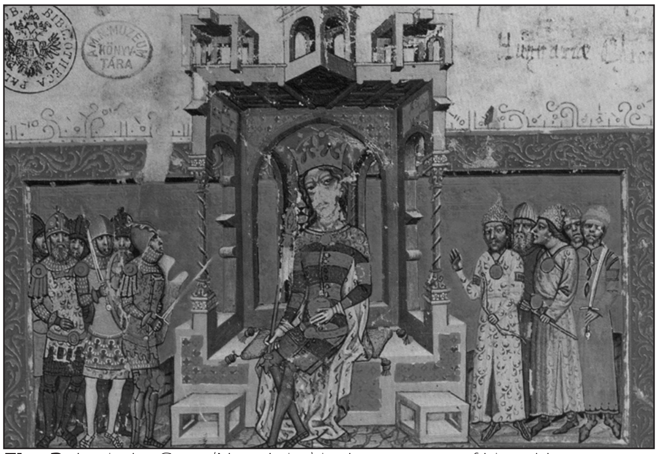


Fig. 2: Louis the Great (Nagy Lajos) in the company of his noblemen, Chronicon Pictum, 14th century

suffered a heavy disaster from the Mongolian invasion of that country. This necessitated the flight of King Béla IV. along with the mortal remains of the first Hungarian king, Saint Stephen (István). Refuge for the king could only be found in Trau (Trogir).

The Franciscan friars accompanying the king also rescued the right hand relic of Saint Stephen, the so called Holy Right, which was then deposited for safe keeping in Ragusa until 1771 (Hóman, Szekfű, 1935). Dating back from these events there remains to this day another relic of Saint Stephen in the reliquary treasury of the Saint Blaise (Vlaho, Balázs) (Armenia, ?–316 A. D.) Cathedral in Dubrovnik and to this day there exists the relic of the hand of the great Hungarian king Saint Ladislas (László) in the treasury of the Franciscan monastery of Dubrovnik *(Fig. 2)*.

A close and enduring connection between Hungary and Ragusa commenced with the reign of the Anjou king of Hungary, Louis the Great (Nagy Lajos). After his Dalmatian war the peace-treaty was concluded in Zara (Zadar) on 18th February 1358 resulting gaining independence from the influence and power of Venice.

Ragusa was immediately ready to acknowledge the supremacy of the Hungarian king and accordingly a delegation of local noblemen were sent to the royal court of Hungary in Visegrád to discuss the relevant conditions. These conditions were in fact privileges with a few minor obligations. Thus Ragusa was able to retain its original laws and legislation and to elect its own magistrate headed by the new dignitary of the "rector", and above all achieved free maritime trade under the protection of the Hungarian king. As recorded in a document from 27th May 1358 (Gelcich 1887) along with the above privileges, some of the obligations cited were: the oath of allegiance, a tax 500 gold ducats annually and the general use of the royal Hungarian flag.

Although the veneration of Saint Blaise – named Vlaho by the Dubrovnik population – in Ragusa was documented from the end of the 13th century, cult of the patron saint of the city only became wide-spread from the reign of Louis the Great. It may thus be worth clarifying whether the life scenes of Saint Blaise in the famous Hungarian Anjou Legend (Levárdy, 1975) were in any way connected with the distinct veneration of the patron saint of Ragusa (Magyar Anjou Legendárium, 1975). In fact, the cult of Saint Blaise is much older. The church named after Saint Blaise – Vlaho was built and sacrificed in late 9th century.

It is also noteworthy, that the beautiful reliquary of Saint Blaise´s foot, kept in the treasury of the cathedral Saint Blaise, is adorned by a coat-of-arms containing only the heraldic silver and red fesses of the first Hungarian royal dynasty *(Fig. 3)*.

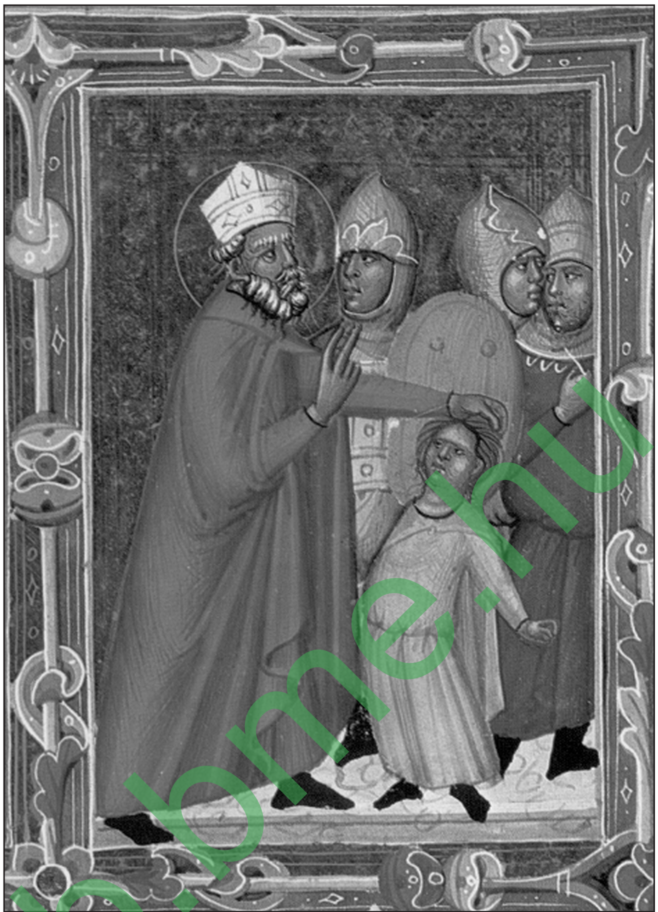


Fig. 3: Saint Blaise healing a child with a sore throat, Hungarian Anjou Legendar, early 14th century.

Relations between the Hungarian Kingdom and Ragusa remained excellent even also after the death of Louis the Great. In the year 1432 the seafaring Ragusans, while congratulating his imminent imperial coronation in Rome, delivered to Sigismund (Zsigmond) of Luxemburg, king of Hungary, the first news of the threatening appearance of the Turks in Europe. At the same time they applied to the king to facilitate free maritime trade with the region of Levant, which request was granted within two years to the complete satisfaction of Ragusa.

Not only was news about the military movements of the Turks reported regularly to the Hungarian royal court, but Ragusa even offered 2000 gold ducats to assist with the war effort against the Turks being waged by the Hungarian governor, János Hunyadi. (In the Croatian literature named Sibinjanin Janko – as in the ballade of the famous Hungarian epic poet János Arany, in the 19th century: Szibinyáni Jank.)

Ladislas V. (László) king of Hungary granted Ragusa the privileged use of a red seal-wax and additionally a heraldic extension of the arms of the city.

Some years later, due a special favour granted by the great Hungarian renaissance King Mathias (Mátyás) Corvinus (son of the late János Hunyadi), the rector of Ragusa in honour of his high dignity, was awarded the ceremonial right to have an unsheathed sword carried in front of him.

King Mathias Corvinus facilitated trade between Ragusan citizens and the Kingdom of Sicily, but by obligation, Ragusa had to deliver sufficient victuals to the nearby frontier fortress of Počitelj. This fortress had been erected by order of King Mathias Corvinus as protection against the imminent Turkish threat (Gelcich, 1887).

Connections between the Hungarian Kingdom and Republic of Ragusa came to an end when the Hungarian capital Buda was occupied by the Turks in the year 1541. Nevertheless, the previous two centuries were beyond doubt a highly successful time in the history of both Hungary and Ragusa.

2.3 The relation of Ragusa to Hungary at the time of Turkish invasion

From this time onwards Ragusa had to pay a high price for its independence, in part for the favour of the Turks occupying the region lying next to the narrow territory of the Republic of Dubrovnik-Ragusa, in part for the favour of the Venetian Republic. The city suffered in 1667 when a very powerful earthquake claimed the lives of many victims and devastated buildings. This remains a reminder to engineers of today how vital is the careful and accurate calculation of seismic effects on structures.

After driving out the Turks from the region in 1684 the maritime trade in the Mediterranean along with the importance of the Ragusan Republic declined. Ragusa, then aligned with Venice, became formally absorbed into the Austrian Empire and remained until the Napoleonic wars.

As the Austrian emperor was in person also the Hungarian king, Ragusa managed to conclude its allegiance with the latter, the formal conditions having been received from Louis the Great.

After brief French supremacy lasting from the peace of Campoformio until 1814, Ragusa again became a provincial town of the Austrian Empire (Gelcich, 1887).

2.4 The state of Croatia and therein Dubrovnik after World War I and World War II

Significant change occurred with the advent of World War I, changing its name from Ragusa to Dubrovnik and annexing to Serbia – Croatia – Slovenia, which later became known as Yugoslavia. When Hitler's Germany invaded Yugoslavia in 1941, they created a state, the Kingdom of Croatia under a puppet government. Subsequent to Hitler's defeat in 1945, Croatia (including Dubrovnik) became a member of the Federative People's Republic of Yugoslavia, which was renamed Socialist Federative Republic of Yugoslavia in 1963.

2.5 Dubrovnik – a pearl of the independent Republic of Croatia

The Southern Slav War of 1991 caused much suffering and casualties for Dubrovnik. Some 500 buildings of the old city were seriously damaged by the military actions of the Yugoslav army and inimical paramilitary militias (Chetniks). Many people both civilians and soldiers, also died. However, due to traditionally good relations, Hungary rendered significant help Croatia both during and after the war. The independent new Republic of Croatia embarked on a program of rapid development *(Fig. 4)*.

Thus the wounds of the war healed quickly and now Dubrovnik is again − and hopefully ever so − the gem of the Adriatic coast, frequented regularly by businessmen and many thousands of tourists both from Hungary and through the world.

Particularly over the last couple of years engineers have enjoyed and appreciated the splendid lines and shapes of the beautiful cable-stayed bridge Dr. Franjo Tuđman at the western gate the city Dubrovnik (Dubrovnik Turistička Naklada, 2006) *(Fig. 5)*.

Fig. 4: The old city of Dubrovnik today

The magnificent hotels and different conference centres with their excellent facilities have served well venues of international scientific conferences end congresses.

Dubrovnik is truly a real pearl of Dalmatian coast of Croatia, and among the many other activities within the city, it is an excellent place for meetings of people coming from different parts of the world.

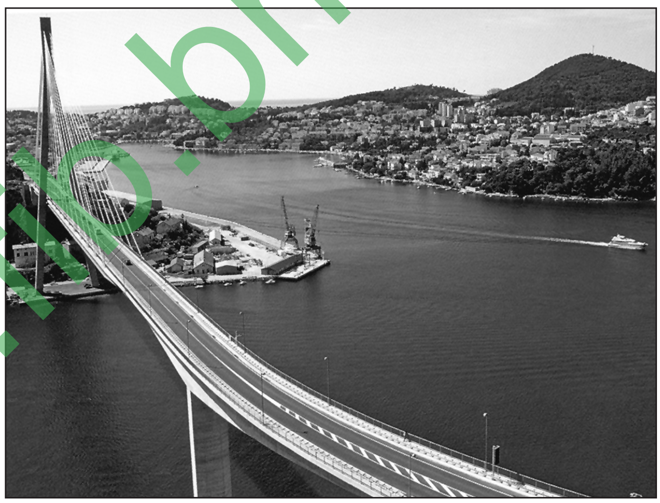


Fig. 5: The new Dr. Franjo Tudman Bridge

3. CULTURAL CONNECTIONS

The two neighbouring nations had wide cultural connections in the field of art, architecture and literature, both ecclesiastic and secular. In this paper, we can mention only a few examples:

Croatian playwright and poet, Marin Držić (1508-1567), whose comedies are popular in Hungary to this day, is to be mentioned. His play „Dundo Maroje" was performed in 2004 in the Hungarian cities of Szolnok and Pécs.

The Zrínyi Family – in Croatian version Zrinski –, whose progeny had Croatian and partly Dalmatian origins. Initially their name was Šubić (in Hungarian orthography Subics). In 1347 King Louis the Great donated to them the fortress Zerín, in Croatian Zrin. Since that time the family used the name Zrínyi, in Croatian version Zrinski. The most notable member of the family was Miklós Zrínyi − Nikola Zrinski (1620-64), who was a poet, politician, strategist, and from 1647 viceroy of Croatia. His main work is the heroic epic "A szigeti veszedelem" = "The Sziget siege". The hero of this epos was his great-grandfather Miklós Zrínyi (1508-66). Péter Zrínyi – Petar Zrinski (1621-71), the brother of the poet, translated the epos to Croatian and it was published under the title "Vazetje Szigetszko". The original was written by an eye witness, Brne Karnarutić (later entitled "Obsada Sigeta"). The

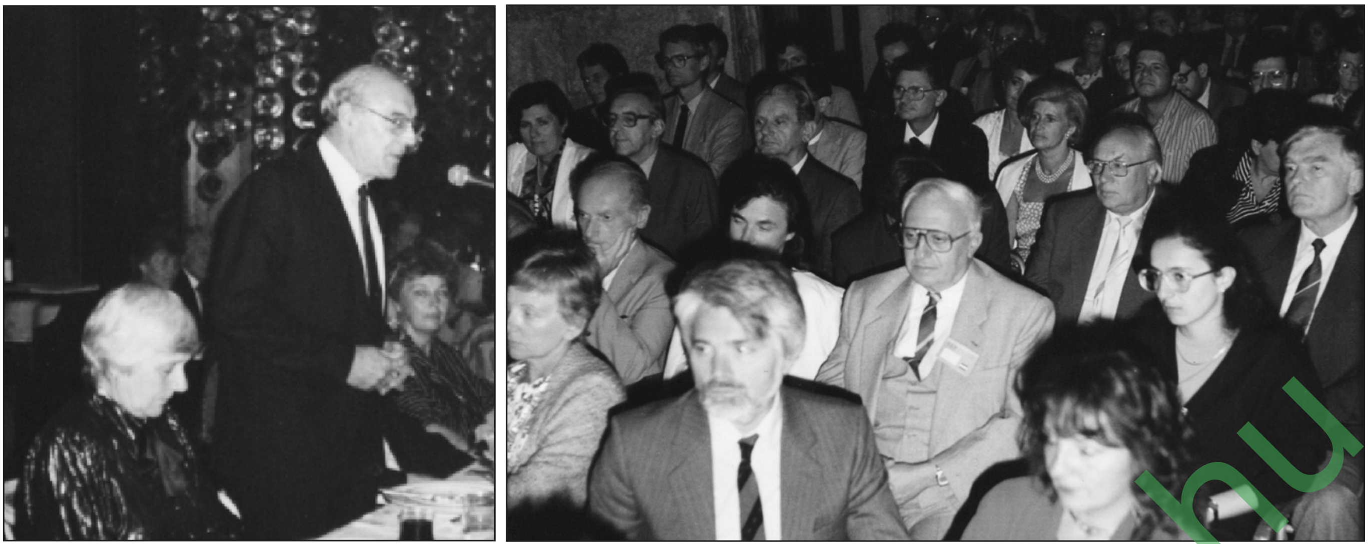


Fig. 6: Professor R. E. Rowe, President of CEB welcomes the participants of the Plenary Session Dubrovnik, 1988

translation gave a Croatian character to the epos (Varjú, 1927; Berei, 1962.).

It is worth mentioning that the most popular Croatian national opera "Nikola Šubić Zrinski" was a great success in 2001 on the stage of the Hungarian State Opera.

Miroslav Krleža (1893-1991), the internationally famous Croatian writer and poet finished his secondary education in 1911 at the cadet school in Pécs. He continued his education at the Budapest Ludovica Military Academy but departed for Serbia before World War I, where he was imprisoned. Miroslav Krleža had a very close affection for Hungarian literature. The poetry of the acclaimed Hungarian lyric poet, Sándor Petőfi (mid of 19th century) had a deep influence on his work, and he translated Petőfi's work „The Apostle". Krleža regarded highly appreciated the poetry of Endre Ady, and published a long essay on Ady, who was the most famous Hungarian poets of the early 20th century. More than ten works of Krleža were published in Hungarian between 1952 and 1980 (Glatz, 2000, Köpeczi, Pók, 1978). It should be noted that Krleža's play "Saint Stephen day festival" is currently being performed in the Budapest Radnóti Theatre.

These few examples or only flashing points of the Croatian-Hungarian cultural links which have existed for over a thousand years.

We must emphasize again that these examples show only a small part of Croatian-Hungarian cultural connections spanning a millennium and it is gratifying that these good links continue to develop during our time.

We would like to note that during the period Professor Zvonimir Marić was active as Consul General of Croatia in Pécs, he contributed greatly to, and encouraged the development of both cultural and technical cooperation.

4. TECHNICAL SCIENTIFIC RELATIONSHIP BETWEEN HUNGARY, DUBROVNIK AND CROATIA

Hungarian businesses undertook several important civil engineering construction projects in Croatia. One example is the reconstruction of the port in Ploče at the Neretva river mouth (1997).

The main task was to strengthen the 507 m long and 23 m wide jetty with a 40 m deep Benoto pile foundation for a concrete slab. The jetty was built in 1966 and was damaged because of soil slipping due to overloading. Additionally some piles had been broken.

Hungarian engineers solved the problem by constructing a new Benoto foundation and a 22 m wide, 0.60 m thick slab on 46m long diaphragm piles, and it can be said that it also supported the slipped dam behind the jetty at the seashore. The damaged jetty was tied by post-tensioning tendons to the new structure (Wellner, Lipót, Szabó 1999). This task could only be performed under a very high level of cooperation between Hungarian and Croatian specialists.

A further example is the bridge across the Una River on the Zagreb-Sisak-Banja Luka-Bihać highway, close to Hrvatska Kostajnica. This project took place in 1966. It is coincidence that close to the bridge a Zrínyi fortress is located, and that the original bridge was constructed by Hungarian soldiers in 1875.

Several tender works on bridges across the Sava River had Hungarian participation.

There are many other fields where Hungarian engineers contributed to structures in Croatia, in great part realizing concrete structures.

In field of concrete science and education, there were valuable results of cooperation. The Technical University of Budapest (today named Budapest University of Technology and Economics) hosted young Croatian people to carry out doctoral research work or to cooperate with Hungarian teachers furthermore students under items of IAESTE.

Professor S. Šram, head of construction of the monumental Krk Bridge, lectured in Budapest. Many other Croatian engineers participated at scientific symposia and other meetings in Hungary, e.g. CEB plenary session (1980), FIP Symposium (1992), *fib* Symposium (2005), many bridge conferences and conferences organized by the Hungarian Group of *fib*.

Croatian students came to study tours to Hungarian construction works. Croatian engineers published papers in Hungarian periodicals and Hungarians in Croatian ones.

Hungarian concrete engineers enjoyed on many occasions the hospitality of their Croatian colleagues, gaining much of the experience. Over many years they visited the construction sites of the Šibenik Bridge, the Krk Bridge and the engineering structures of the Učka motorway many years ago among others. Recently Hungarian experts studied many other structures: Mala Kapela and the Sveti Rok tunnels, the Maslenica bridges (the new one, built on the Zagreb-Split motorway, as well as the reconstructed old one), the bridges over the Krka river at

Skradin and over the Guduča Gorge (all of them on the Zagreb-Split motorway) as well as two bridges over the Cetina river (on the Split-Dubrovnik motorway) during construction phase.

Hungarian educators visited the Zagreb and the Osijek universities and lectured there. The University of Pécs and the University of Osijek have a close ties under the leadership of Prof. P. Lenkei and Prof. Z. Marić. A multi member Hungarian delegation participated at the CEB plenary session in Dubrovnik *(Fig. 6)* in 1988. (Tassi, Lenkei, 2003). Also Dubrovnik provided the venue to the conference on durability of concrete structures in 2004 which included Hungarian participants. Conferences in Zagreb, Plitvice, Brioni and other places were locations where the exchange of opinion and experience enriched the knowledge of Hungarian engineers.

We discussed the benefits of the international scientific conferences before the *fib* Symposium Budapest, 2005 (Balázs, Tassi, Borosnyói, 2005). We are confident that the *fib* Symposium Dubrovnik 2007 will contribute to our mutual benefit and to all participants of the meeting.

5. CONCLUSION

There is more than a one thousand year period during which Hungary and Croatia lived as a very close community and in neighbourhood. There was mutuality of the nations in traditions, religion, art, architecture, literature, and science. Recently, the technical cooperation comes in foreground along with other problems to be solved in Europe and all over the world. A meeting, such as the *fib* Symposium Dubrovnik 2007 will reach all participants and they will profit in knowledge, in developing good cooperation between the engineers of different nations, and it will improve the excellent cultural links between Hungary and Croatia.

6. ACKNOWLEDGEMENTS

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Prof. Z. Marić deserves the highest merit. The excellent cooperation existing between the Croatian and Hungarian *fib* Groups, the technical and cultural friendly links are owing to him in lion part. The authors express their thanks for rendering assistance to this article.

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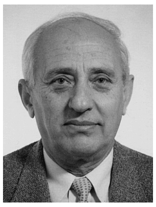
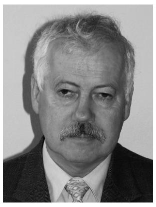
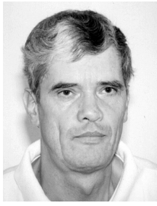
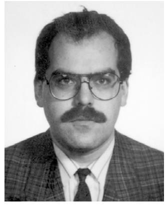
VIADUCT OF KŐRÖSHEGY, THE LARGEST PRE-STRESSED CONCRETE VIADUCT IN HUNGARY – DESIGN AND CONSTRUCTION

Péter Wellner – László Mátyássy – Tamás Mihalek – János Becze – János Barta

The 1872 meters long, 23.8 meters wide continuous, prestressed concrete structure is being built on the motorway M7. This motorway leads from Budapest, the Hungarian capital, to the Hungarian-Croatian and Hungarian-Slovenian border. On this section, this viaduct is the largest engineering structure. Its significance is, from technical point of view, marked by the special constructing method, the short construction time, and the continuous relationship of design and construction, realized in international co-operation. From the middle of year 2007 the road traffic can use the motorway, except for a short section, as far as to the coast of the Adriatic Sea.

Keywords: viaduct, prestressed concrete superstructure, free cantilevering technology, Advancing Shoring System, prestressing tendon, expansion joint, bored pile, pier, precasting, hydraulic lifting jack

1. INTRODUCTION

The design works of the viaduct started in the spring of 2004, after winning the tender. The construction works were started in June 2004, with the site preparation works. The construction of the superstructure started from both abutments, right after the completion of the structures of the abutments and the first piers. So, the construction of the structure is progressing from two directions, and the two parts will be closed together in the middle of the viaduct.

Until the end of the year 2006, three fourth of the structure was built. Because of the unusually short specified time, in the interest of faster construction, we needed to change the construction technology underway. Of course, the extent of this change had to be harmonized with the requirement of not affecting the appearance or the static system of the engineering structure.

In this article, a report is given on technical issues of an interest beyond average, that we came across in the area of design and construction of the viaduct.

Fig. 1: Longitudinal section

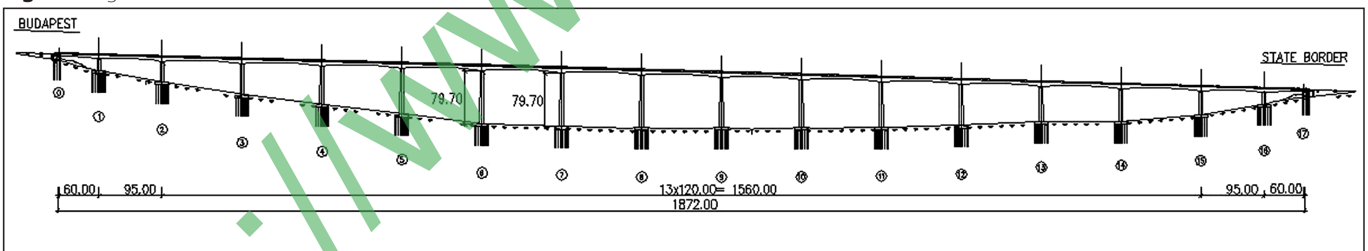
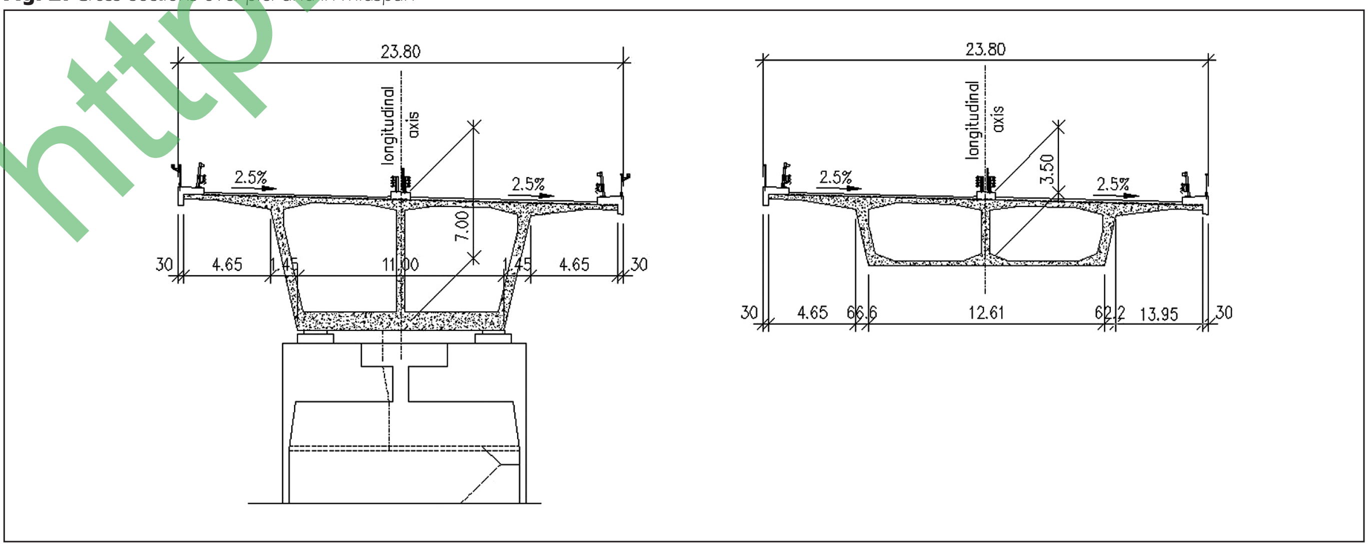


Fig. 2: Cross-sections over pier and in midspan



2. GEOMETRY OF THE VIADUCT

The longitudinal profile of the structure can be seen in *Fig. 1*, the cross-sections of the superstructure above the pier and in midspan are shown in *Fig. 2*.

3. SUBSTRUCTURES

3.1 Foundation conditions

The geotechnical conditions of the valley in the respective section are characterized with layers of silt, rockflour, clay and, in some places, marl. On the slope of the hillside towards Budapest, the soil conditions are slightly more favourable than on the side towards the state border. In the flat valley of the Séd creek of Kőröshegy, high ground water and loose soil can be found in the upper layer. In the process of designing the foundation, it was an important point of view that, given the significant loads, relatively low settlement values had to be reached. The selected solution for the foundation has, first of all, been meant to reach this goal.

Considering the soil conditions, the above-described requirements are met by the large diameter, bored, reinforced concrete pile foundation used. The diameter of the piles is 1.50 m at the abutments, and 1.20 m at the piers. A total amount of 19 000 m of bored reinforced concrete piles were made for the foundation of the viaduct.

It was insufficient to design the technical configuration and structural analysis of the special size foundation in accordance with the solutions used so far in domestic practice. The reason of this was that the calculation of the piers couldn't be fulfilled independently, but only as part of the entire bridge structure, because the interaction of the superstructure and the substructures cannot be neglected.

Therefore, the internal forces and deformations of the abutments, piers and the superstructure were studied on a complex model containing the behaviour of the superstructure and also the elastic restraint of soil. Also secondary effects were taken into consideration in the calculations. The behaviour of the piers due to earthquake was also studied. Special care had to be taken of the analysis of stages during construction.

3.2 COMPLEX INVESTIGATION COVERING BOTH THE SUBSTRUCTURES AND THE SUPERSTRUCTURE

For the structural analysis of the viaduct, several finite element models and programs were used. The models were elaborated to the detail corresponding to the design task.

For calculating the complex model of the entire structure, the TDV RM2000 program was used. The interaction of the superstructure, the 16 piers, the abutments and the foundations was investigated on this model. The loads for the more detailed models were taken from this model, and the construction stages and earthquake effects were also investigated on it.

The wind load has to be highlighted from the point of view of both the foundations and the substructures. Given the 80 m long arm of force on the highest piers, the transversal force arising on the lateral surface of the superstructure represents a huge flexural load at the foot of the piers and on the foundation. As the viaduct lies in a horizontal curvature of 4 000 m radius, there is no unequivocal transversal and longitudinal direction from the point of view of the surface of the entire structure, exposed to wind, therefore, when determining the design loads, the wind load was applied in 5 degrees steps. Examining the construction stages, the surfaces of the main girders and the formwork wagons of the Advancing Shoring System (ASS) were also significant in the point of view of the wind load.

It was also possible to study the effect of the number of the fix bearings on the complex model. From the point of view of the horizontal forces parallel with the axis of the viaduct, it is more favourable to take these loads by more piers with fix bearings than to take them by the friction of the movable bearings, but movable bearings used in more piers are more favourable to take the forces arising from the creep and shrinkage of the concrete of the superstructure and from the effect of the variation of temperature. Taking the various points of view into consideration, 4 bearings fixed in longitudinal direction were used in the central section of the viaduct, and for the remaining 12 piers, as well as for the two abutments, movable bearings were applied.

3.3 Detailed calculation of the substructures

The considerable size of the viaduct required that the dimensions of the structural elements should be determined with a multilevel modelling much more detailed than the usual calculations.

The piers were also studied, one by one, with more detailed structural systems, using the programs AXIS VM and TDV RM2000.

The large sizes of the foundation bodies (20 m×27.20 m ×3.5 m) and the great, concentrated forces acting on them made it necessary to perform additional analysis of these structural parts in even more detail. This was executed by using the finite element procedure of the TNO DIANA program. For studying the highlighted parts, the design loads were taken from the detailed pier model.

On the pier caps, the concentrated reaction forces of 2 bearings (load capacity 65 000 kN each), has to be distributed on the pier walls. This distribution was demonstrated by the spatial investigation also performed with the TNO DIANA program.

3.4 Abutments

The horizontal and vertical loads acting on the abutments are received by 15 piles with 1 500 mm diameter. The pile caps, the walls and the slabs of the structures are made of reinforced concrete.

Behind the retaining walls, rooms were configured to allow placing electric, hydraulic and telecommunications equipment, and also to facilitate the performance of maintenance works. The equipment of internal energy supply is also located here. The collector basin of the rain-pipe system and the energy dissipation reinforced concrete troughs are located in the abutment towards the state border.

When designing the foundation of the reinforced concrete abutments, it had to be held in mind that, during the construction of superstructure, up to the very last closing, the abutments are considered as fix supports. The horizontal force arising in this way significantly exceeds the loads arising during service state.

3.5 Piers

Between the abutments, 16 piers are built, the height of which, in accordance with the longitudinal profile of the viaduct and the features of the ground, is varying between 17.70 m and 79.70 m. The piers are constructed on foundation bodies leaning on large diameter bored reinforced concrete piles. The dimensions of the foundation bodies are unusually large: 2×2 pieces are of the size 16.40 m×23.60 m×2.60 m, 2×3 pieces are 20.00 m×23.60 m×3.50 m, and 6 pieces are 20.00 m ×27.20 m×3.50 m. The largest foundation body is a 1 904 m^3 reinforced concrete slab, which is concreted in a single phase. When concreting such a massive concrete block, an important point of view was to use a special concreting technology and to monitor the temperature of the concrete, in order to avoid the damaging of the concrete caused by the amount of heat generated by concrete setting.

The piers are configured with closed, two-cell, box cross-section, stiffened by horizontal diaphragms in every 20 meters. The external walls of the piers are flat-faced, parallel in crosswise direction, the sides are slightly slanting in the bridge axis direction. The walls incline towards each other with a tilt of 1:140, while approaching the pier cap. This kind of configuration of the piers basically influenced the construction technology of the walls.

For building the ascending walls, climbing formwork were used. The starting pier sections and the concreting sections belonging to them are of varying height, but in the upper part, regular, 5 m long sections are built. The thickness of walls at the starting section is 0.80 m, further on upwards: 0.45 m and 0.35 m. When preparing the final drawings and the arrangement of reinforcement, a solution accommodated to the specifics of the technology was arrived at. From the upper part of each pier downwards, concreting sections of identical configuration and armature are made allowing the wall sections to be made of prefabricated reinforcing elements. The reinforcing elements of the size of the individual wall sections are placed in one piece, and the connecting reinforcement of the corners is installed on site.

As mentioned, the walls are stiffened by horizontal diaphragms in every 20 metres, made with prefabricated structure. After placing, they are connected to the pier walls using in-situ concreting and re-bars placed in the walls previously.

The corrosion resistance, constant quality and equal appearance of the concrete walls has been reached with the appropriate concrete technology.

The configuration and construction technology of the structural beams were also unique. The shuttering assembly work of a 3.1 m high structural beam in 80 m height is a serious engineering achievement. Prefabricated FP bridge beams were used for this purpose. These bridge beams serve as formwork in holding the concrete weight of the 1.9 m high first concreting phase. The already completed lower part of the structural beam also takes part in supporting the second concreting phase.

3.6 Conjoining structures of the substructures

Spherical cap bridge-bearings are designed for supporting the superstructure. Longitudinally fix bearings are to be placed on four piers in the middle of the viaduct, and movable bearings are designed on the remaining supports.

The special, 1 872 m long superstructure is built in one dilatation section, therefore the movement of the expansion joints is also unusually large, and in total it reaches 1 200 mm.

As regards the bearings and the expansion joints, it is worth mentioning a special solution ensuing from the unusual length and the 2.86 % longitudinal slope of the viaduct.

It is easy to see that, if the movable surface of the bearings on both ends of the viaduct were built in horizontally as usual, the longitudinal profile of the carriageway at the length of the expansion joint would change its slope according to the movement of the expansion joint. This error of the longitudinal profile might even endanger the traffic of the motorway. To avoid this phenomenon, the sliding surfaces of the bearings were placed parallel to the slope of the carriageway, and the ensuing load effects were taken into consideration in the calculations of the substructures.

4. CONSTRUCTION OF THE SUPERSTRUCTURE

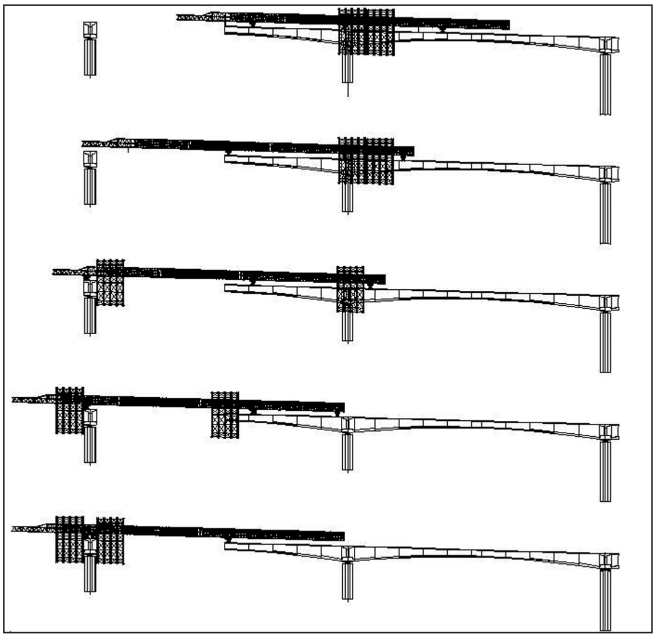
4.1 The technology

As mentioned above Hídépítő Co. had decided to use a rarely applied technology to execute the construction procedure of the Kőröshegy viaduct quicker, than the traditional free cantilevering method. This construction method was offered and provided by the German company Peiniger RöRo. The main point of this technology is that the formwork wagons (travellers) are not fixed onto the completed concrete cantilever part, but they hang on a pair of 160 m long and 4 m high steel girders, which are supported always on three points: on the two ends of the just constructed balanced cantilever and on the end of the last completed concrete cantilever. The pair of girders, its supports and the formwork wagons together are called Advancing Shoring System (ASS).

What are the advantages of the application of the ASS?

- More, than twice as long segments can be produced as in case of the traditional technology. In Kőröshegy this length is generally 11.25 m.
- After completing and closing a balanced cantilever, it is not necessary to disassemble the ASS, transport it to the next pier on the ground, and after lifting it up onto the next pier to re-assemble it. The ASS is able to move onto the starting segment on the next pier 'in the air', using its own hydraulic jacks (*Fig. 3*).
- The ASS can stabilize the constructed cantilevers over the pier by its own weight.

Fig. 3: Typical phases of the moving sequences of the ASS from one pier to the next one



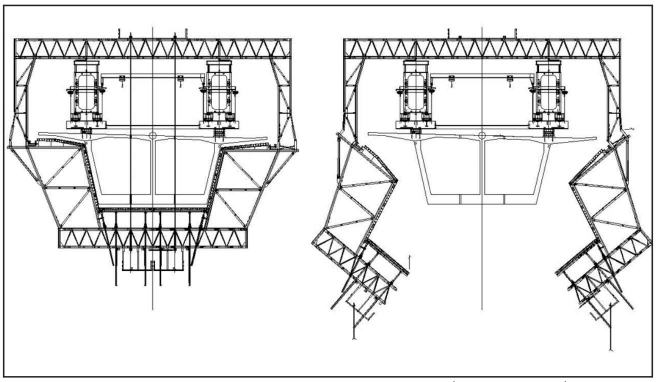


Fig. 4: Formwork wagon in closed and open state (cross section)

- The materials (including even the fresh concrete and the big formwork plates) can be transported to the working place (to the formwork wagons) along the ASS.

The ASS consists of three main parts: the girders, the wagons, and the main supports; and many complementary structures: two help supports, cranes, engine to move the wagons, etc. Though the superstructure is curved (with a 4000 m radius), the axis of the ASS is straight. The main girders have two parts: a cca. 140 m long main part (partly web plated, partly truss structured beam), and a cca. 20 m long leading nose (truss structure, lighter than the main part, not able to carry the weight of the wagons). The formwork wagons are 12 m long. They have 5 frames (*Fig. 4*) 3-3 metres from each other with stiffening trusses between them. The three main supports have high load capacity: they transfer the cca. 1600 tons total weight of the whole ASS to the superstructure. The horizontal moving devices (both in longitudinal and in transversal direction) of the ASS are connected to the main supports, which have sliding plates on the top, to make possible the movement of the main girders on them. However, they also have brakes to make possible the fixation of the ASS, when needed. The role of the help supports is to substitute the main supports when they are taken out and moved to another position (both in construction stage, and during moving the ASS from one pier to the next one; see *Fig. 3*). The main girders mustn't be moved, when the help supports are active and any of the main supports is inactive!

The detailed working process is the following:

Two main supports (called A and B) of the ASS are standing on the actual starting segment, the third (C) on the end of the previously completed cantilever of the superstructure (in case of the first pier: on the abutment). The two formwork-wagons are closed on the ends of the starting segment. The prefabricated reinforcing armatures of the bottom slab and the webs (*phase 1*) are lifted up and placed into the wagons by the own cranes of the ASS. After placing the inner shuttering, phase 1 can be concreted. The total weight up to this point, including the weight of the fresh concrete of phase 1, is carried by the ASS. After the hardening of the concrete of phase 1 it is tensioned to the starting segment with the help of 2-2 tendons led in the webs. As the result, phase 1 will not only be self-sustaining, but will also be able to carry the weight of the concrete of the paving slab (*phase 2*). After placing the shuttering and the prefabricated reinforcement, the top slab is concreted and after its hardening the tendons in it are tensioned. Preparing to produce the next segments the main supports A and B are replaced onto the ends of the fresh-made elements, using the help supports; the main-girders are pushed forward a segment-length together with the front wagon; the rear wagon is pulled back with the same distance. This is the end of the constructing cycle of one pair of segments. One balanced cantilever generally consists of five pairs. After completion of a branch the closing segment is constructed with the rear formwork wagon. With this action the total constructing cycle of one balanced cantilever has reached to its end. After this a 25 phase moving sequence has to be executed (*Fig. 3*), as a result of which the ASS will be in the starting position of the constructing process of the next branch over the next pier.

As a matter of curiosity we mention that in order to be able to move beside the piers the formwork wagons are opened, and then after passing by the pier are closed back with the help of hydraulic jacks (*Fig. 4*).

To reduce the construction time the superstructure is built from two sides: one-one ASS (*Fig. 5*) have started from both abutments, and they will meet in the middle of the viaduct after completing 8-8 balanced cantilevers.

Fig. 5: ASS in work

4.2 Interactive design of the ASS

The ASS was designed (Saul Ingenieure GmbH) and manufactured (Peiniger RöRo) in Germany, while the calculation of the superstructure was executed in the Technical Department of Hídépítő Co. in Budapest, Hungary. The two constructions have significant effect on each other. Beside the self-weight of the concrete superstructure and the effect of the prestressing, the reaction forces of the ASS had to be taken also into consideration. In comparison with the cca. 200 tons weight of a formwork wagon in case of the traditional free-cantilevering technology the total weight of the ASS is almost 1 600 tons. This obviously means higher stresses in the superstructure during construction. That's why the Hungarian and German designers had a permanent and interactive contact during the design works, so that both the prestressed concrete superstructure both the steel ASS should be convenient in all building stages and in final state as well. They furnished the reaction forces in the construction stages and in the different moving sequences, and we checked the superstructure. When we suggested any changing that had effect on the ASS, they checked their structure.

The most critical situations occur during moving the ASS from the completed and closed balanced cantilever part to the next pier. The highest tensile stresses appear in the top slab of the fresh-completed closing segment in 3 different stages: when the main-girders of the ASS are pushed forward cca. 60 m without supports under their nose, when the first, and finally when the second formwork wagon is being moved towards the next pier and they are located directly over the main support, which is on the end of the 57 m long cantilever. We can declare that as a result of a long common designing work we could find the solution that is satisfactory for both parts.

4.3 Measurements during construction

In order to get the precise shape of the superstructure at the end of the constructing procedure the height position of the constructed segments have to be adjusted to a previously calculated precamber value. To get these values we asked the help of the German design office from Stuttgart, Leonhardt, Andrä und Partner Gmbh (LAP), who have great international prestige and experience in this field. Their task was first to execute the independent analysis of the structure (we can say proudly that they found everything satisfactory); and then to calculate the precamber values of the segments. The height of the formwork wagons at the tips of each segments are adjusted according to these values before casting them. After pouring the concrete of phase 1, the position of the tips of the actually constructed segments and (if any), the previously completed segments of the same balanced cantilever are measured, similarly after the segment is prestressed to the already completed superstructure part. These measurement values are always given to LAP, who recalculate the precamber according to the reality, and give us the modified values, if necessary.

4.4 Settlement of the piers

To reach the correct shape of the completed structure it is not enough to determine the deformation of the balanced cantilevers, but it is also needed to know the settlement of the substructures. For this purpose 4-4 measuring points were placed on the top of the pile caps of each pier, and another 4-4 points in the wall of the first segments of the piers and the abutments. The geodesic heights of these points are measured after every action, which causes extra load on the certain foundation. By the time the pier-cap is completed, the behaviour of the foundation and the soil under the pier is quite well-known. In cooperation with geotechnical experts from the Budapest University of Technology and Economics as well as Geoterra Ltd. we could estimate the future settlement of the substructures under the loads that would be acted on them later (*Fig. 6*).

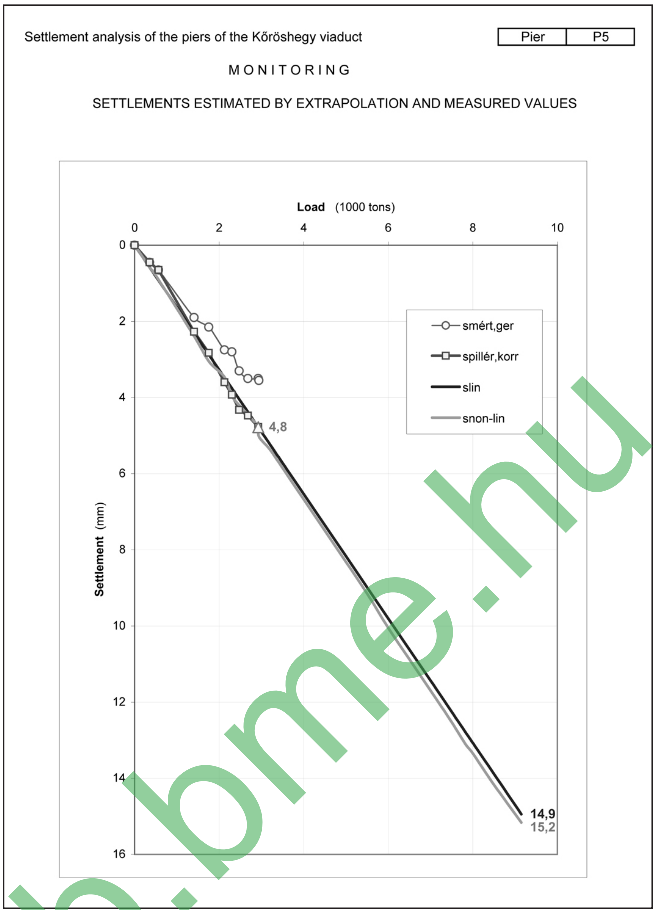


Fig. 6: Linear and non-linear settlement curves of a pier, with the measured values (TU Budapest)

Meanwhile the elastic shortening of the RC walls of the piers were also calculated considering even the long-term effects. The necessary extra height of the bearing-tables and so the cambered level of the bearings could be determined from these data, taking the actual temperature into consideration, too.

The maximum necessary correction of height due to later settlement was 40mm. However the monitoring of the settlement of each substructure is continuing after placing the bearing on the bearing-table and during the construction of the starting segment and all other segments of the superstructure. Having more and more measuring data the estimation of the future settlements were more and more precise.

5. ANALYSIS OF THE SUPERSTRUCTURE

5.1 Structural system

The viaduct rests on 18 supports. Its total length is 1 872 m with the following span-spacing: (1.0 m)+60.0 m+95.0 m +13×120.0 m+95.0 m+60.0 m+(1.0 m). The axis of the viaduct, in plan view, lies in a circular curve with a radius of R=4 000 m. The longitudinal slope of the carriageway level is

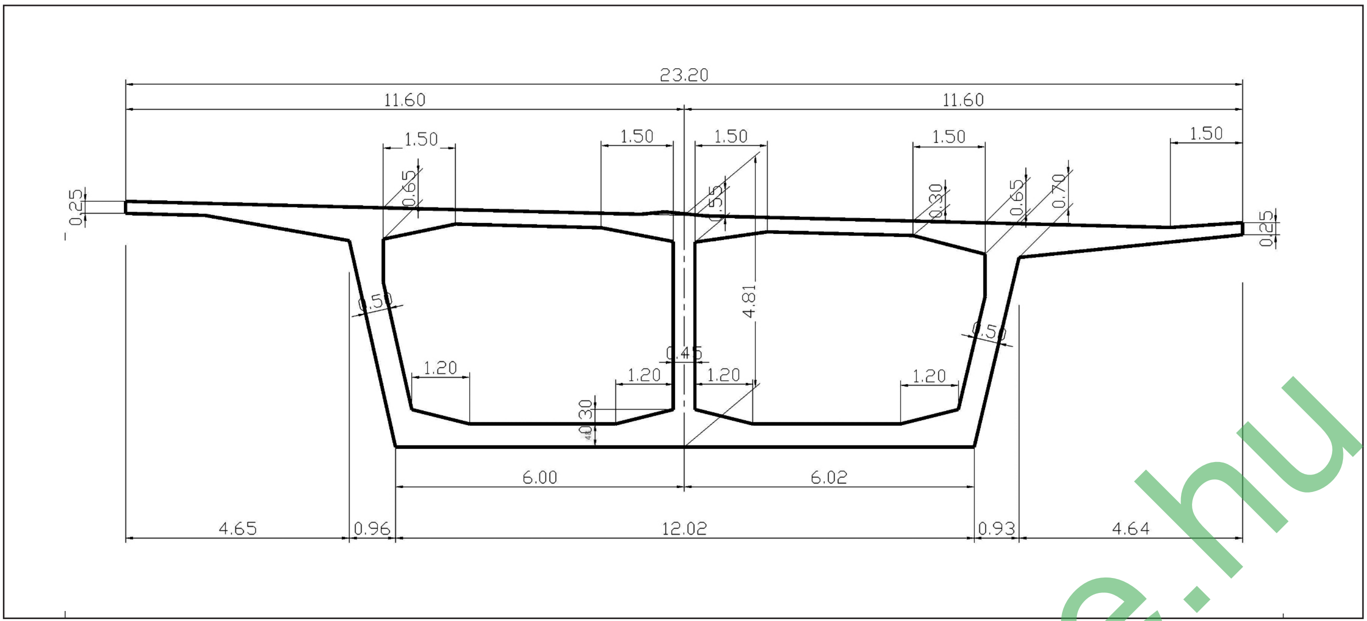


Fig. 7: Dimensions of the box-girder

constant, 2.86%, the carriageway has a transverse decline of 2.5%. A level difference of 53.50 m can be measured between the abutments marked 0 and 17. The superstructure is made with a single box, two-cell cross section with cantilevers of the slab on both sides.

On the viaduct, the traffic lanes of opposite directions are separated by a 1.0 m wide curb; the total width of the top slab of the superstructure is 23.80 m. The carriageway is divided as follows: exterior service footway with handrail (1.40 m) + safety lane (1.50 m) + traffic lanes (2×3.75 m) + lateral distance from the mid curb (1.0 m). The division of the carriageway in both traffic directions is identical (*Fig. 7*).

The height of the prestressed concrete box superstructure above the exterior piers marked 1 and 16 is 5.50 meters, while above the rest of the piers is 7.00 m. In the midspans, the structural depth reduces to 3.50 m, the bottom level of the superstructure changes in parabolic line between these values.

The top slab of the box girder is built with constant width (23.20 m) and thickness. The thickness of the cantilevers varies between 0.70 m and 0.25 m, the thickness of the slab between the webs is 0.30 m; with a 1.50 m wide haunch next to the three webs, (0.25 m on both sides of the inner web, and 0.30 m next to the outer webs).

The thickness of the bottom slab of the box varies from the supports to the midspans in linear manner between 1.00 m and 0.25 m (*Fig. 8*).

The superstructure is constructed in sections. A 115 m long balanced cantilever is made on each pier, with 5.0 m long connecting sections at midspans between them (closing segment). The parts of a balanced cantilever: there is a 6.0 m long basic element (starting segment) above each pier to which, on both sides, a 9.5 m long segment-pair is connected. The remaining segment-pairs are 11.25-11.25 m long.

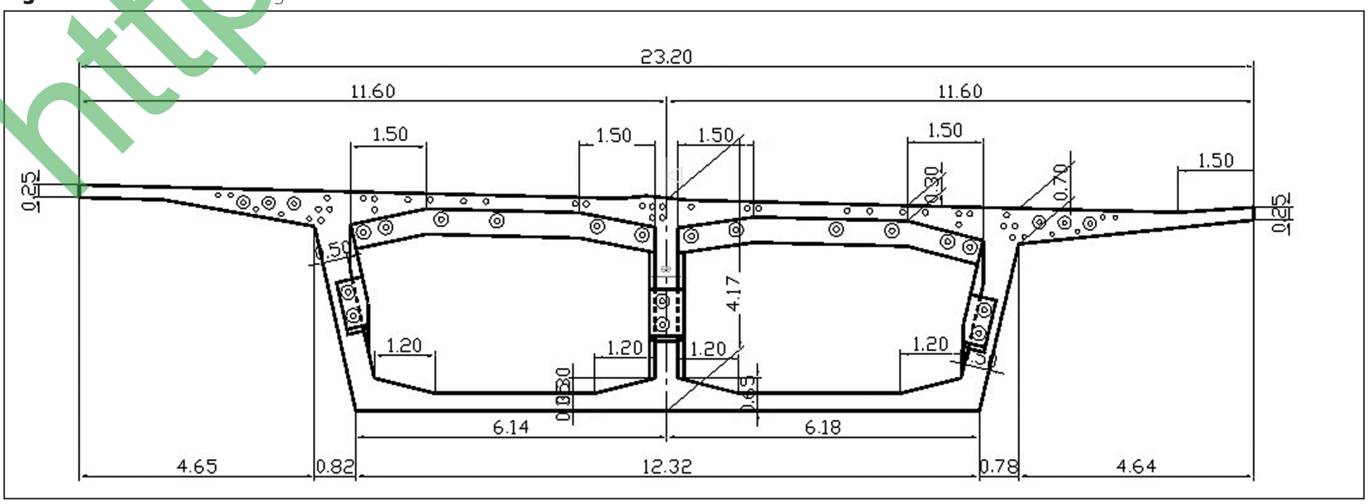
The load-bearing capacity of the superstructure of the viaduct (both in construction and in final state) is ensured with post-tensioning tendons. During construction, tendons led within the concrete structure (top slab + webs) are used, while the final state is reached by making the structure continuous (with closing tendons in the bottom and the top slabs), and finally, the full load-bearing capacity necessary for carrying the traffic load is provided by external sliding tendons led inside the box girder.

The outer webs of the box girder are made with a leaning of constant inclination, narrowing downwards. The base width of the bottom slab of 11.0 m at the supports will change to 12.62 m up to the midspans.

The middle web is 0.45 m thick all over the entire viaduct, while the two outer webs are 0.70 m thick in the starting segments over the supports and in the first segment-pairs (on a total length of 9.5 m+6.0 m+9.5 m=25.0 m), and in the remaining segments they are 0.50 m thick.

The starting segments of the three webbed box girder above

Fig. 8: View at the end of a segment



the piers, are supported by two bearings, therefore a 1.50 m thick cross-wall is provided here, in which – in both cells – a 1.85×2.50 m opening had to be left, that will accommodate the service footways and pipelines for draining rain water.

There are also cross-walls in the fourth segments, as deviating points of the external sliding cables. The thickness of these is 80 cm. The anchoring points of the external tendons are placed in the 5.0 m long closing segments in the middle of the spans. The tendons are anchored on both sides of the 1.3 m thick cross-wall, configuring an overlapped extension.

Cross girders with 2.0 m width, thickened to 0.80 m, are configured on the ends of the segments, for anchoring the prestressing tendons led in the top slab, while the 3×2 cables led in the webs are anchored in the webs thickened at the segment-ends (*Fig. 9*).

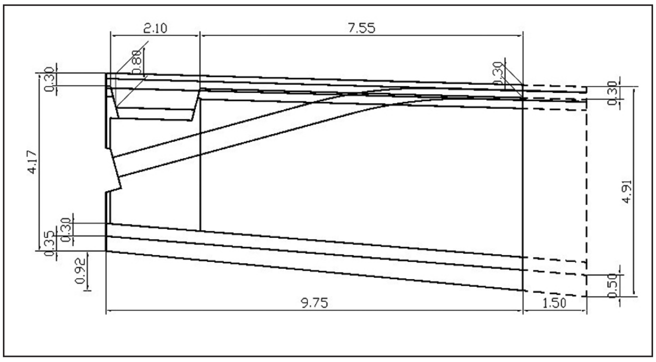


Fig. 9: Longitudinal section of a segment with prestressing tendons

5.2 Loads

In the process of the structural designing of the viaduct, the norms ÚT 2-3.401:2002 "Designing road bridges" (Loads and actions) and ÚT 2-3.414:2002 "Designing rules of road bridges IV" were taken into consideration. The viaduct was dimensioned, in accordance with the Load Class "A" of the above-mentioned rules, for a motor vehicle load of 800 kN. Among the self-weight loads of the superstructure, the weight of all the internal installations and equipment existing in the final state, which are necessary for the service and maintenance, were also taken into account. Such are the service footways in both cells over the full length of the bridge, the rainwater pipeline and also the flush-water pipeline.

The weight of the ASS was also a significant self-weight character load in the construction state.

5.3 Structural systems

a) Construction stages: The superstructure is built as a balanced cantilever, progressing in two directions from the starting segment over the pier. Two 11.25 m long segments are made on each end of the balanced cantilever, poured into the hanging formworks, which are held by an auxiliary steel structure leaning on the two ends of the balanced cantilever being built (A and B supports). The third (C) support leans on the cantilever-end of the already completed part of the viaduct.

The starting segment is to be made on the two spherical cap bridge bearings. The balanced cantilever can rotate and slip on the bearings. Its stability is ensured by the ASS, because during turning the two reaction forces (A and B) equalize each other. This equalization starts at the time of the construction of the second segment-pair because the starting segment is kept tied down up to the time when the first segment-pair has been completed, and the tying will only be released after this. The completed balanced cantilever is connected (closed) backwards, to the completed viaduct part, so longer and longer continuous girder parts with more and more supports are made.

b) Final stage: Finally, the two half-viaducts with 9-9 supports, are closed in the middle to each other, and the continuous superstructure with 18 supports gets configured.

During service, the longitudinal static equilibrium of the bridge is ensured, and horizontal forces are received by the fix bearings built in on the 4 central piers.

5.4 Analysed states

The structural analysis was performed with the finite element program packet Ponti of the German company RIB.

a) Construction stage: The construction of the segment-pairs on both ends of the balanced cantilever took place in two phases, in formwork wagons. In the first phase the bottom slab of the box and the three webs were poured, up to the upper level of the webs. The weight of the reinforcement and the concrete of the first phase is carried by the hanging formwork. The shuttering of the top slab was shored up on the completed bottom slab, so the self-weight of the part of the top slab between the webs is passed over to the bottom slab, while the cantilevered part of the top slab on both sides is carried by the formwork wagons. Due to the above facts, the hardened first phase concrete structure had to be made capable of bearing these loads, which was achieved with prestressing 3×2 pieces tendons in the webs, consisting of 19×0.6" strands.

The prestressing could be started after the result of the 36 N/mm^2 ultimate stress of the control test cubes was available. After the hardening of the second phase concrete, the prestressing of the further tendons lead in the top slab could follow.

Examining the construction stages, it was pointed out that no tensile stress greater than allowable would arise in the top of the first phase concrete:

$$\sigma_{FI} < (f_{ctd}/1.2)/2 = (2.3\ \mathrm{N/mm^2}/1.2)/2 = 0.96\ \mathrm{N/mm^2}.$$

After the completion of the full cross-section, nowhere arose tensile stress:

$$\sigma_{FII} < f_{ctd}/1.2 = 2.30\ \mathrm{N/mm^2}/1.2 = 1.92\ \mathrm{N/mm^2}.$$

During construction, when the ASS is moved over the next pier, in the mid cross-section of the last span of the completed viaduct-part, a tension did arise in the top, but its value also remained under the allowable value ($\sigma_{fl} < f_{ctd}/1.2$ = 1.92 N/mm^2).

b) Modified construction stage: In the process of the construction, a modification became necessary, on the basis of which the segments were not poured in formwork with monolithic concreting in their final position, but in a shuttering, down on the ground, with prefabrication. Their length became shorter by 1.50 m, and they were lifted up with lifting equipments moving on the existing ASS. The lifted precast segment-pairs were connected to the ends of the balanced cantilever completed so far, by making a 1.50 m wide monolithic concrete strip.

The monolithic strip was also made in two phases: the segment-pair became self-bearing by prestressing, after the hardening of the concrete of the full cross-section, the 3×2 prestressing tendons led in the webs, and then, it was possible to release the suspension. At last, the prestressing of the remaining tendons followed. Also by employing the above method it was ensured that no tension arose in the concrete of the balanced cantilever (σ_i< 0 N/mm²).

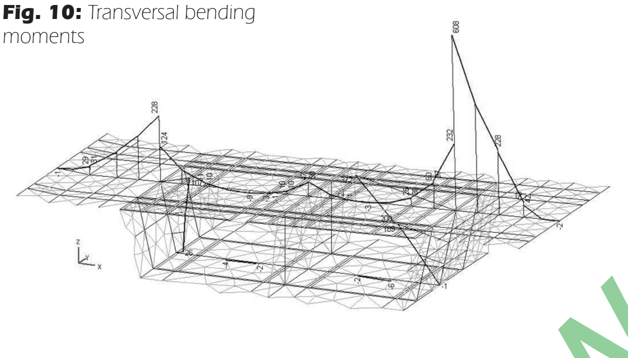
c) Final stage: The external sliding tendons, inside the box girder, are prestressed in the completed superstructure. Over the full length of the viaduct 2×(4+2)=12 pieces of tendons consisting of 19×0.6" strands are used. These tendons ensure that, taking the service value of the specified load class into consideration, nowhere in the superstructure arises tensile stress in the concrete.

5.5 Analysed main structural elements

Detailed investigations were performed with the Hungarian finite element program AXIS VM 8.0.

a) The top slab: The analysis of the upper area of the box girder (the full top slab) was performed on a space model built up of shell elements and containing three spans of the viaduct. In this process, the dimensioning of the anchoring girders (forming crosswise lower rib), configured on the segment-ends was also fulfilled. In all cross-sections of the top slab such a bi-directional reinforcement was applied which ensures the w<0.2 mm crack-width in the structure for the service load (*Fig. 10*).

Fig. 10: Transversal bending moments



b) Webs: The shear investigation of the webs of the box section, both in construction and final stage, was performed with the shear and principal stress testing module of the RIB Ponti program. As a result, ø20/15 main reinforcement was applied on both the right and left side of the webs, which, approaching the supports, had densification from segment to segment and later were placed in two rows, with the following allotment: ø 20/15 + ø 20/15, then ø 20/15 + ø 25/15.

c) Cross-girders: They provide the support of the middle web because it has no direct support; only two bearings were applied under the outer webs. In the process of the analysis, a diaphragm wall with rectangular openings for the service footways was studied, and the structure was provided with reinforcement that will ensure significant shear resistance and suspension strength.

d) Deviating points of external sliding tendons: For receiving the vertical and horizontal friction forces arising at the deviating points of external sliding tendons, a full height wall was configured between the bottom and top slabs, and it was also tied in the webs. This way it was possible to avoid exaggerated loads arise in the bottom slab due to the components of tendon forces.

e) Cross-wall at midspan: In the closing segments at midspan, 1.30 m thick cross-wall was applied in which it became possible to anchor the sliding tendons, and to extend them with overlapping. The walls - in accordance with the prestressing sequence of the tendons, - were dimensioned to bear tendon forces from only one side (after all tendons have been prestressed, the loads get equalized almost entirely). The dimensioning of the wall was calculated on a space wall-model fix-supported at three sides. A crack width of w<0.2 mm can be ensured with the reinforcement used (ø 20/10 + ø 25/10),

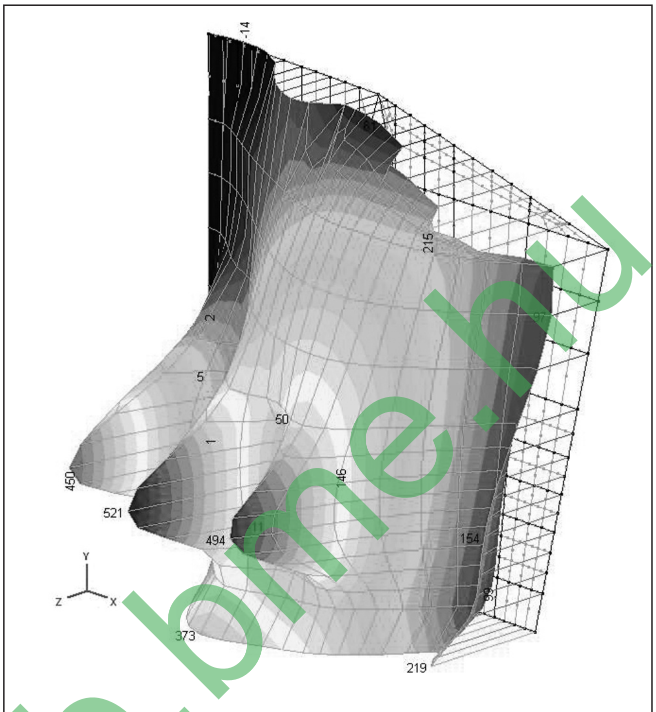


Fig. 11: Bending moments in the diaphragm wall

satisfying the prescription (*Fig. 11*).

5.6 Independent Analysis

The independent analysis of the design and the calculations of the viaduct was performed by the experts of the German company Leonhardt, Andrä und Partner.

The calculations also covered, beyond the checking of the final state, the analysis of each phase of the construction. According to their findings, the superstructure satisfies, in all its details, all respective criteria of the Hungarian regulations.

They examined the whole viaduct - in its entirety - on a finite element space model. The normal stresses of the complete main girder were checked, along with the shear and principal stress states. Their calculations also covered the review of the reinforcement of the top slab and the webs, according to which they found the quantities of the reinforcing bars sufficient, and the configuration optimal.

6. MOVEMENT-LIMITING STRUCTURE IN THE ABUTMENTS FOR RECEIVING LONGITUDINAL TENSILE AND COMPRESSIVE FORCES

The construction of the superstructure is performed starting from the two abutments. The completed parts of the viaduct are fixed to the abutments, ensuring in this way the receipt of pier forces induced by longitudinal movements (changes of length resulting from temperature changes, shortenings resulting from creep and shrinkage) (*Fig. 12*).

For fixing, 2 + 2 pieces of tendons consisting of 19×0,6"

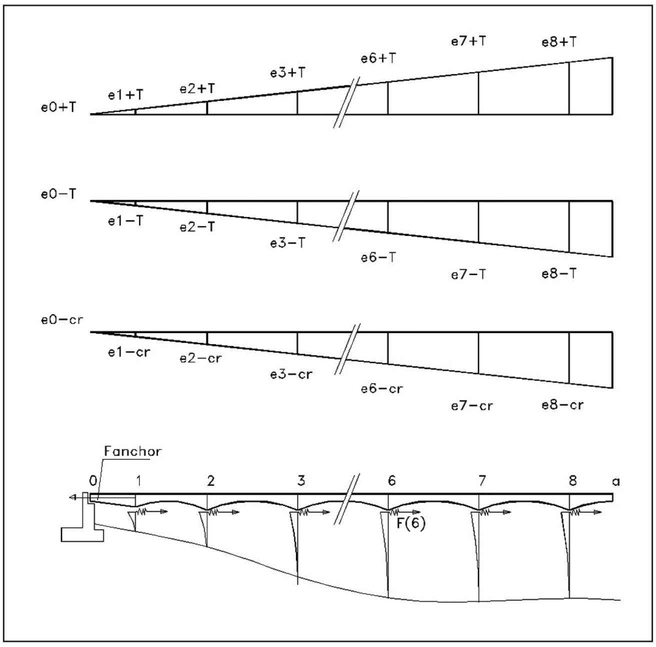


Fig. 12: Forces on the pier-head caused by natural effects

strands led inside the box girder are used. One ends of the tendons are anchored inside the superstructure, in the cross-girders (diaphragm walls) of the starting segments over piers 16 and 1. The other ends of the tendons are anchored in anchoring plates concreted into the thickened (strengthened) section of the retaining walls of the abutments.

NEOPRENE bearings are built in between the superstructure-end and the abutment-wall for ensuring flexible horizontal supporting and occasional rotations. The NEOPRENE bearings (3 – 3 pieces under the fixing tendons) are placed on auxiliary bearing frames, and are built in vertical plane, with the contingent gap filled in with steel plates. These plates are underlayed in such a way that, when the value of the pier forces exceeds the force in the fixing tendons, during the further elongation of the cables (accompanied with the increase of the distance of the superstructure from the abutment) the plates, falling out, give a warning that the abutment is receiving a non-admissible force, and it is necessary to decrease the force in the tendons (releasing) (*Fig. 13*).

The prestressing tendons consist of Fp 150/1770 type strands. Their length is 62 m.

See the characteristics of fixing tendons in the following table:

Applied prestressing force (before anchoring):	F_{anchor} = 2800 kN ,
Calculated elongation of cable:	324 mm
Compression of the NEOPRENE bearing:	2 mm
Compression of the superstructure part due to prestressing:	2 mm
Cable elongation, total:	328 mm
Cable force – after wedging:	2730 kN / cable

The tendons are anchored in MA6819 anchoring devices concreted into the anchoring block formed in the inner side of the retaining wall of the abutment. For checking the value of the tensioning force in the tendons, force-measuring cylinders are built in between the anchoring head and the anchoring device, at two - two cables per abutment, whose electrical signals allow us to continuously check the value of the tensioning force in the tendon.

The anchored half-viaduct will be longer and longer during construction connecting the completed balanced cantilevers to it. Due to these connections, the longitudinal movements of the superstructure (shortening – elongation) will affect more

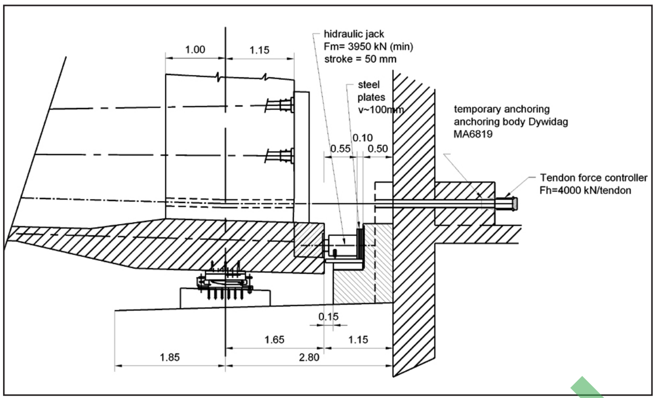


Fig. 13: Anchoring at the abutment

and more piers and, through the bearings, the changes in length will cause the displacement of more and more pier caps. And the displacement of the pier caps can only come about with the resistance of the piers, namely, the resistance value of the connected piers gets higher and higher.

Taking the time schedule of construction into consideration, we plotted the cumulative intensity of the resistance of piers which is shown in *Fig. 14*.

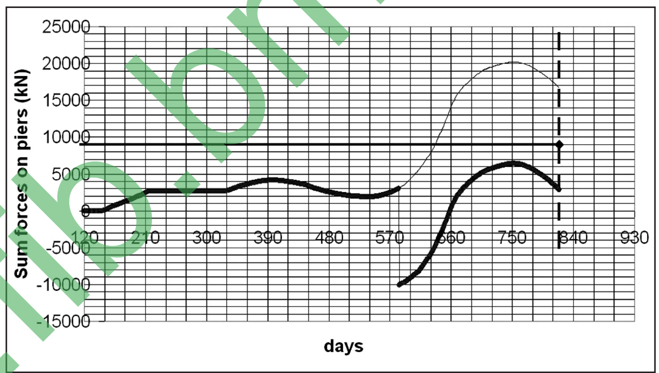


Fig. 14: Cumulative forces from the piers

The diagram shows the maximum value of force ($F_{h\text{-abutment}}$ = 9 000 kN/abutment) which the foundation (piles) of the abutment can bear (assuming also that the backfill in full width has also been completed).

It can be seen from the diagram that at about the ¾ of the construction time (between 600 and 630 days) that is, in September – October 2006, the resistance of piers (the force pulling the abutment) would gradually reach, then exceed the $F_{h\text{-abutment}}$ = 9 000 kN value (the piers more and more incline towards the abutment - they are drawn as a bow).

In order to keep the tensile force on the abutments under the above described limit values, the pier caps are displaced artificially towards the centre of the viaduct (their “updraw” is reduced). This is reached by shifting the half-viaduct that is, the superstructure is pushed with the help of hydraulic jacks placed between the abutment and the end of the half-viaduct, then, after it has been moved, the increased gap between the abutment and the superstructure is filled up again, and the jacks are removed. Of course, this shifting elongate the fixing tendons and also increases the tensioning force.

The enclosed diagram shows that, assuming different bearing frictions, to what extent the cumulative forces could increase – the date of exceeding the limit value would not change.

On the basis of the evaluation of the figures, it can be seen that the shifting had to be taken place before the date described above (September 2006). The designed value of the shifting is 50 mm. The above operation was performed on both half-viaducts successfully.

7. MODIFIED CONSTRUCTION TECHNOLOGY

The cycle time ensured with the original technology, in practice, was 14 – 15, sometimes 13 workdays. With this construction speed it was not possible to observe the deadline set for the completion of the construction.

7.1 New building technology

7.1.1 Shortening of the cycle time

The most acceptable way of shortening the cycle time was the change-over to prefabrication. If the segments are precast on ground level, right under their final position, they can be built in by lifting them onto their final elevation. After these, it is only the technology of lifting of the elements that had to be elaborated.

7.1.2 Preparing of prefabricated segments on ground level

The 5-5 superstructure segments of each balanced cantilever were prefabricated on ground level. Preserving the original superstructure division, the segments were divided into two parts. The prefabricated elements were prepared 1.50 m shorter. On the 1.50 m long section of the monolithic connection, it was possible to carry out the tendon connections properly. The individual segments were prefabricated in the position corresponding to the curved geometry of the carriageway. As to their height position, the segments are independent from each other, so it became possible to form the fabrication area following the varying ground level of the valley. Naturally, when prefabricating the elements, besides the original accessories, also the connecting elements for the lifting spreader beam pair had to be placed.

7.1.3 Utilization of the existing auxiliary structures

Prior to the introduction of the new technology, we were able to decide on the adaptability of the equipments used for the in-situ free cantilevering work, after a structural checking analysis.

The steel main girder pair was adaptable without changes. It was possible to shorten one of the originally 12.00 m long formwork wagons by splitting it into two 3.00 m long parts. The length of the other wagon was reduced to 6.00 m for constructing the closing segment.

The construction process of the superstructure according to the new technology was started on pier 5 on the Budapest side, and on pier 13 (fourth from the abutment) on the state border side (*Fig. 15*).

7.1.4 New auxiliary structures and equipments

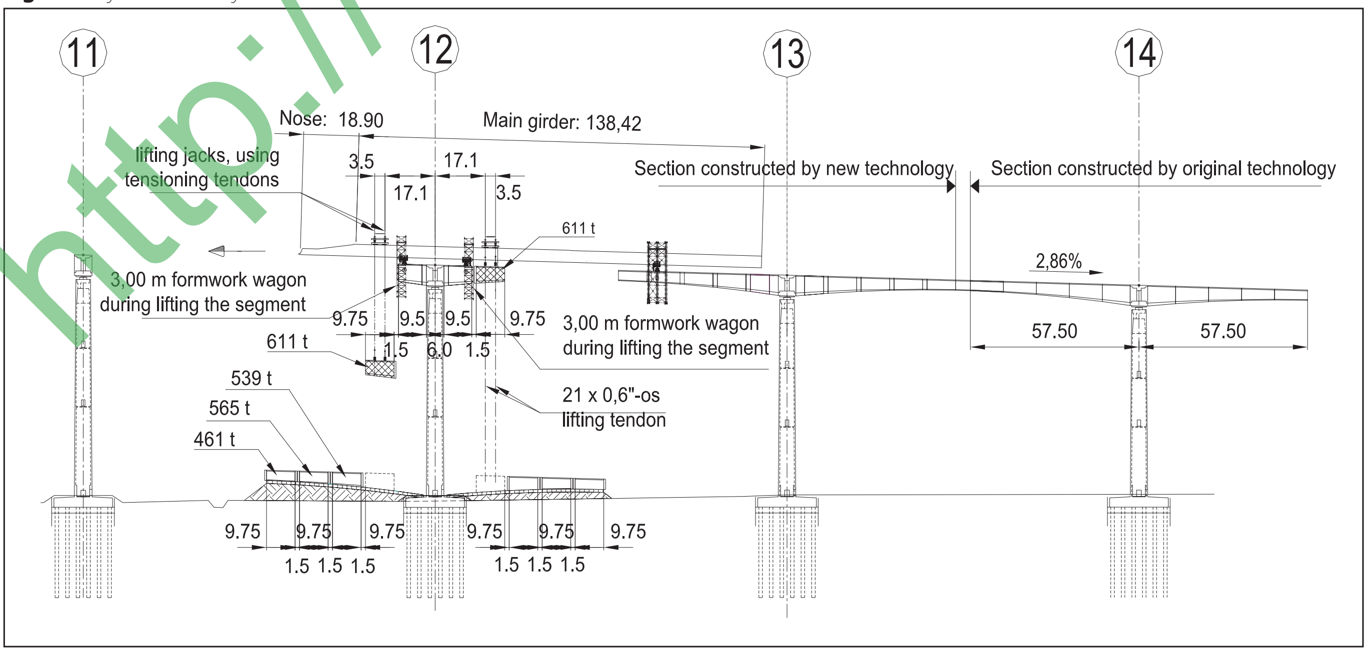
For placing the lifting jacks and tendon coilers we had to manufacture and install so called lifting frames. The lifting frames were placed on the steel main girders next to the formwork wagons. The four hydraulic lifting jacks with the capacity of 210 tons each, their control equipment, and the coilers of the lifting tendons were placed on the lifting frame (*Fig. 16*).

For each of the lifting phases, the lifting frame pairs can be adjusted to the lifting and fixing position of the next segment together with the connecting formwork wagons. The lifting can be started with tearing up the segments. In all cases, the lifting operation had to be started with tearing up the segment towards the abutment. The lifting was performed in 1.00 m steps. The time required for lifting an element is 6 to 8 hours. Special load-distributing spreader beams were applied to lift up the prefabricated segments from the ground level (*Fig. 17*).

7.1.5 Horizontal and vertical adjustment

The independent analysis of the superstructure and the values of the vertical adjustment (precamber values) for each precast segment are also calculated and specified by the company Leonhardt, Andrä & Partner. The prefabricated elements are adjusted in accordance with the calculated precamber values. For concreting the 1.5 m long monolithic closing part, the formwork wagon is closed to the completed superstructure part and the lifted segment hanging on the lifting tendons.

Fig. 15: Layout of auxiliary structures – side view



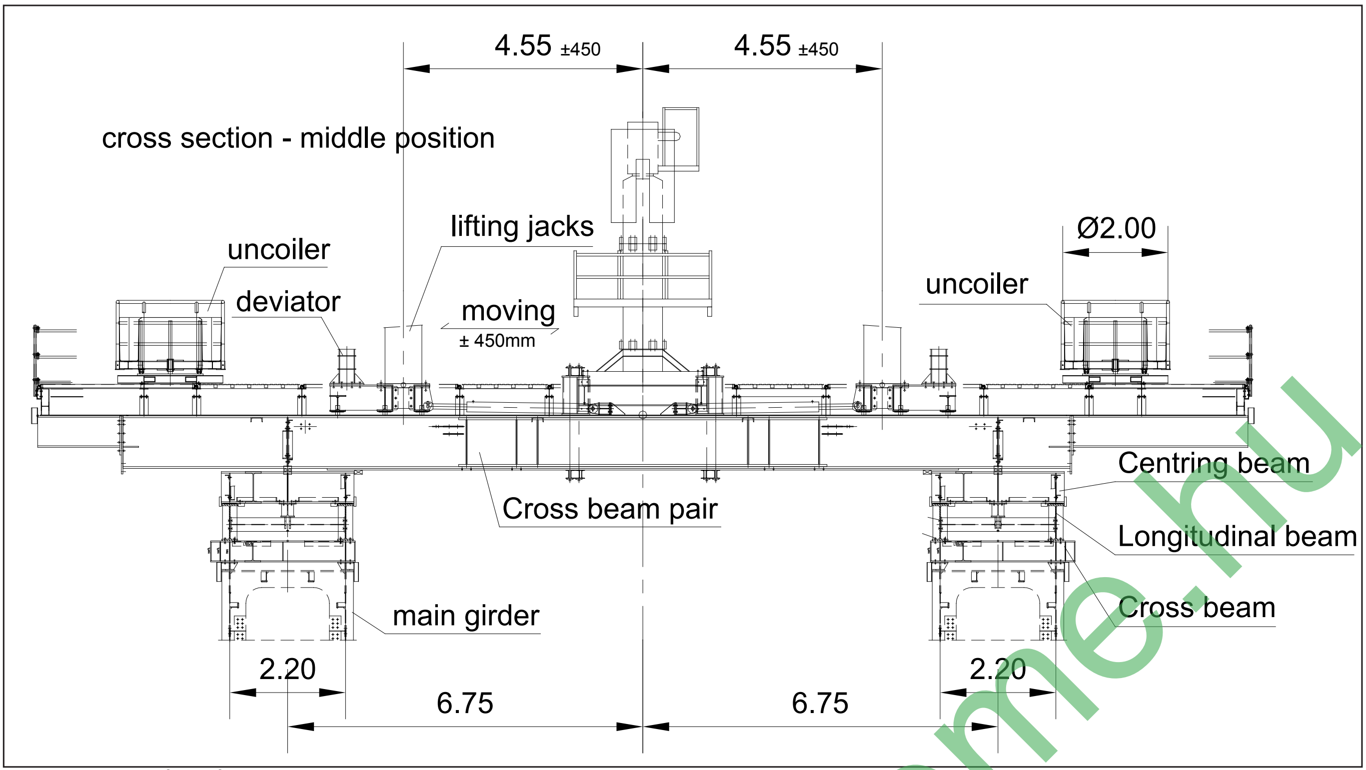


Fig. 16: Layout of the lifting beam - cross-section

Horizontally, the segments are adjusted according to the country-wide coordinate data corresponding to the actual balanced cantilever.

7.1.6 Shortening of the construction cycle

With the application of the new technology the construction process of the superstructure was divided into two activities, which could be performed in parallel. In the process of the prefabrication at ground level, it was possible to observe the 2 to 2.5 week long fabrication cycle per segment pairs. It was possible to adjust the starting time of prefabrication on the basis of the time required for the fabrication and erection works.

The cycle of the erection of the cantilevering – along with the lifting of the segments, adjusting of the formwork wagons, concreting of the monolithic strip, prestressing – was shortened to one week.

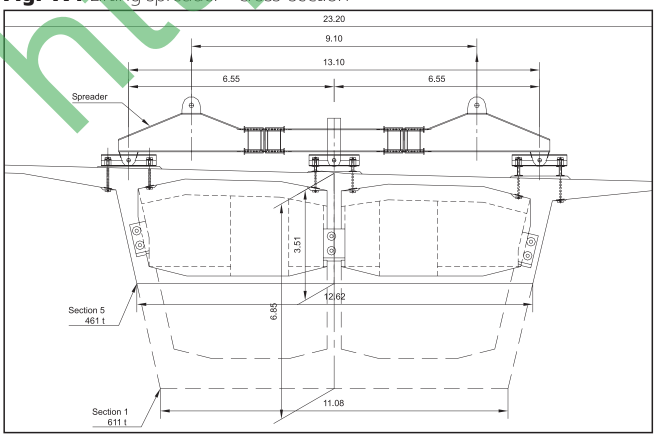
8. CONCLUSIONS

The Helsinki corridors constitute a very important part of the European traffic network. Several of them cross the territory of Hungary. Hídépítő Co. is constructing Central Europe's longest viaduct as a part of one of them, the M7 highway. In consequence of its extraordinary sizes and the short deadline we chose a rarely used technology to erect the structure, which is faster than the traditional method: balanced free-cantilevering concreted in-situ with Advancing Shoring System. Its fastness is the result of not only the longer produced segments, but also the short time needed to move the ASS to the next pier, and the easy way of transporting the building materials to the construction site. Meanwhile, to speed the construction even more up, the technology was changed: precast elements are lifted up from the ground with the help of hydraulic lifting jacks, and are connected to the previously completed part with concrete wet joints and prestressing.

It is a great professional challenge and even greater pride to take part in a job like this. Both the design and the construction works proved to be a task that needed a wide range of domestic and international cooperation.

According to our hope the big work will be completed in 2007, so from the middle of the summer only a short section will be missing to be possible to travel on highway from Budapest to the Croatian and Slovenian border, moreover as far as the Adriatic Sea.

Fig. 17: Lifting spreader - cross-section



Péter Wellner (1933), M. Eng. is Head of Technical Department at Hídépítő Co. Designing of prestressed reinforced concrete bridges and the associated institutions involved in their technology in Hungary indicates his successful professional background. He received a State Prize for his involvement in the first bridge built with the cantilever mounting method. He also took part in the launching of the method of cantilever concreting in Hungary. The incremental launching technology was initiated in Hungary under his direction. Such structures are now continuously used. At the moment he is the responsible designer of Köröshegy Viaduct. He is a member of the Hungarian Group of ***fib*** and the Palotás prize was awarded to him.

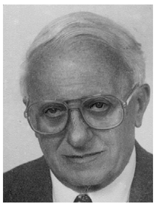
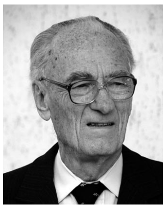
László Mátyássy (1949), MSc. structural engineer. Graduated from the Technical University of Budapest in 1972. General Manager of Pont-TERV Co. Specialized in bridge design. He participated in the design of several major bridges, including prestressed concrete viaducts built by the free cantilever or incremental launching method and steel/prestressed concrete bridges over the Danube and Tisza. President of the Bridges Section at the Hungarian Chamber of Engineers. Member of Hungarian Group of ***fib***.

Tamás Mihalek (1950), MSc. Structural Eng. He started his designing profession 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 with 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 reinforced concrete bridges, the influence of the structural materials and the applied building technology on the structure and considering these influences during statical calculations. Nowadays he is dealing with the design works of the Köröshegy Viaduct. He is a member of the Hungarian Group of ***fib***.

János Becze (1948), MSc. Civil Eng. He started his designing profession at the Bridge Department of the Road and Railway Design Co. (UVATERV). He took part in the technological design works and the design of the temporary structures of many great bridge structures in Hungary. Since 1987 he has been working at the Technical Department of Hídépítő Co. His basic task was the developing of the adoption of the incremental launching technology in Hungary, and the design of the necessary temporary structures. Since 1988, the design of the Berettyó bridge near Berettyóújfalu, he designed the complete technological process of several structures. He also worked in the technological design of the St. Stephen bridge at Szolnok, where he had lion part in solving foundation problems, too. In addition he deals with the design of special steel structures and technological tasks. Recently, he designed in frame of his department the first extradosed concrete bridge in Hungary, and now he is working on the technological solutions of the Köröshegy Viaduct. He is a member of the Hungarian Group of ***fib*** and the Palotás prize was awarded to him.

János Barta (1968), MSc. Civil Eng. has been design engineer at the Technical Department of Hídépítő Co. since 1997, after working for five years at a statical design company (designing building structures consisting mainly of office buildings and blocks of flats). At Hídépítő he took part in the design works of the sub- and superstructures of several bridges, such as the Viaducts on the Hungarian-Slovenian railway line, the Homokkert overpass in Debrecen and two viaducts on the M7 highway; and also a pier structure of the Port of Ploče, Croatia. He was the statical designer of Hungary's first extradosed bridge structure at Letenye, and took part in the design works of the Köröshegy Viaduct. Member of the Hungarian Group of ***fib***.

CONCRETE BRIDGES TO RIVER ISLANDS

Prof. Géza Tassi – Dr. Herbert Träger

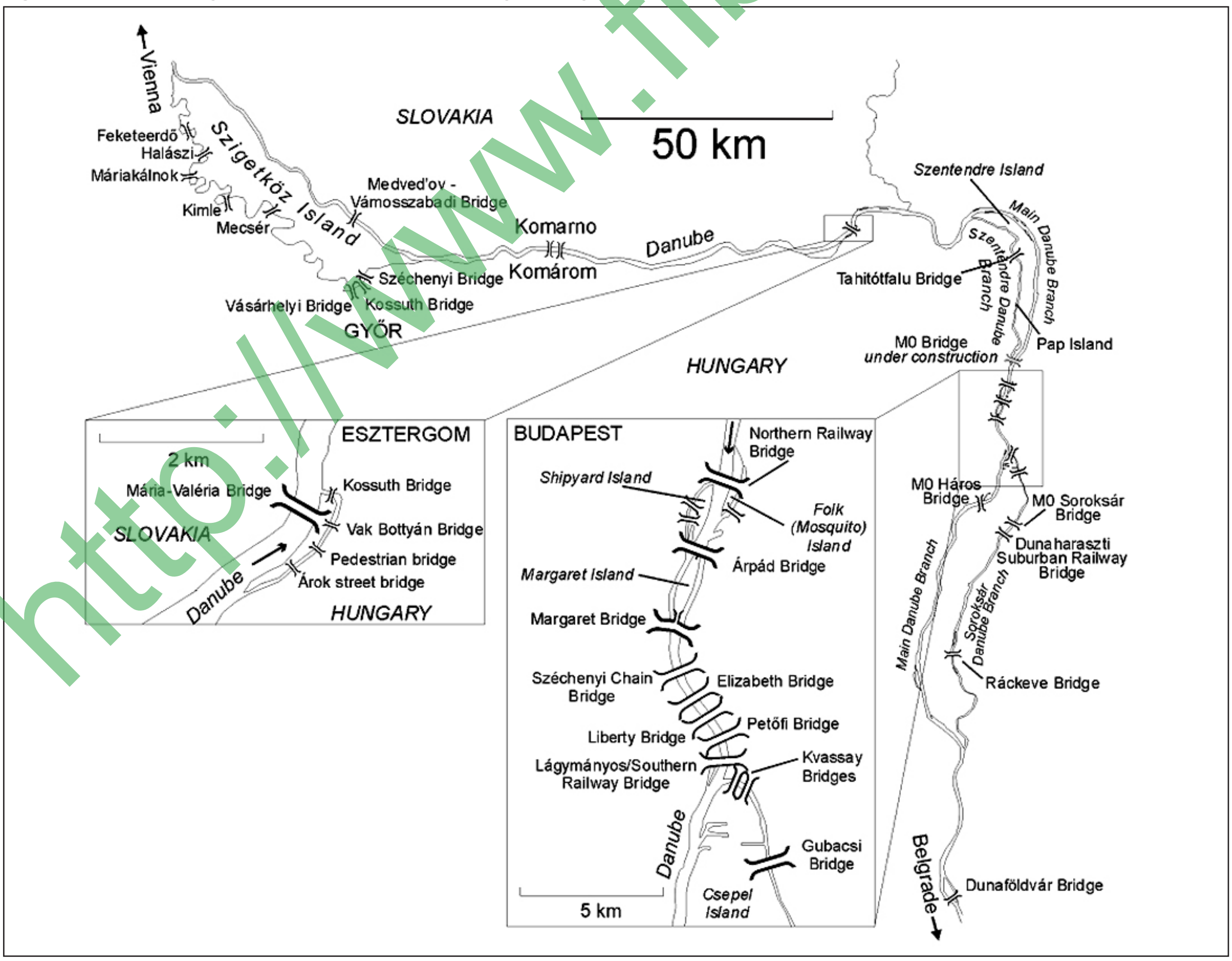
Hungary is one of the few European countries having no seashore. The fib Symposium 2007 in Dubrovnik appointed as one of the themes the connection between mainland and island. This paper shows that within the proposed topics it is worthwhile to deal with concrete bridges leading from the Danube river shore to major and minor islands. Concrete and composite bridges are discussed showing some specialties and presenting some bridges of other type.

Keywords: Danube, river island, arch and girder bridge, concrete and composite structure

1. INTRODUCTION

The Hungarian section of the Danube River is 417 km long. A part that is interesting from point of view of islands is shown in *Fig. 1*. The main feature of this section is that the river leaves the Alpine region not far of the Austrian/Slovakian border. The channel slope decreases, the speed of the current is slowing down, the bedload sediment is depositing. This induces braided, downstream anal branching planform that is ideal for island forming (Timár, 2003, 2005; Timár, Telbisz 2005; Pišút, Timár, 2006).The major islands in the Hungarian territory are the followings (in parentheses the length of the smaller branch): The Szigetköz (70 km), Szentendre Island (32 km) and Csepel Island (50 km). There are several minor islands. The ones which are connected by bridges to the river shore are the followings: The Prímás Island at Esztergom (2.5 km), the Shipyard Island (3 km), the Folk (Mosquito) Island (now peninsula, 2 km) and the Margaret Island (2.5 km). About 70 km southwards from Budapest there is a small island between Dunaföldvár and Solt. This island is crossed by main road Nr. 52 the part of

Fig. 1: Islands of the Hungarian section of the Danube and bridges leading to them



which is the Dunaföldvár Danube bridge leading over the small unsettled island, and another bridge is connecting the island to the right side river shore. The Mohács Island is close to the Serbian and Croatian border. Its eastern branch is actually an artificial channel with unimportant small bridges.

There are many steel bridges leading to islands. In this paper the steel structures will be shortly mentioned and a more detailed description is given about the concrete and composite structures. Among the bridges which are discussed we find different structural systems: arch, single, compound and multi bay girder, continuous composite structure, special arrangement of concrete and steel parts.) The destination is various: highway, railway, pedestrian and pipe bridges.

2. THE BRIDGES FROM THE DANUBE RIVER SHORE TO ISLANDS

2.1 Bridges to the Szigetköz

2.1.1 Bridges to the Szigetköz in general

The Szigetköz is situated between the main river bed of the Danube and the Moson Danube branch. The majority of the bridges across the Moson Danube are steel structures, these are the following. After the locality the main feature of the bridge is given. Feketeerdő: Main bay structure is steel, the other is of concrete. Halászi: Temporary steel bridge Máriakálnok: Steel structure. Kimle: Similar to the bridge at Feketeerdő. Mecsér: Steel-concrete composite bridge. Győr: The "Vásárhelyi Pál" pedestrian bridge, a very early cable stayed steel structure, another Moson Danube crossing in Győr is the Kossuth Bridge, a steel Langer type arch. It is noteworthy because of its very early welded connections. The Széchenyi highway bridge at Győr under the heaviest traffic is a prestressed concrete structure (see Chapter 2.1.2.). The Szigetköz is connected to the left side shore, across the main river bed by the continuous steel truss bridge Medvedov (Slovakia)-Vámosszabadi.

2.1.2. The prestressed concrete bridge across the Moson Danube at Győr

The Széchenyi Bridge in Győr is the first Hungarian bridge constructed by balanced free cantilevering resulting in a monolithic structure (*Fig. 2*). It was completed in 1979 (Varga, 1980). The spans of the continuous box girder structure across the river bed are 50+90+50 m. The superstructure consists of twin box girders. The traveller system and post-tensioning (Freyssinet) was applied. The full width of the carriageway is 14m, and there are sidewalks at both sides with a useful width of 1.60 m. The depth in the middle of the main opening is 2.43m, and above the intermediate support 4.63 m. There are access bridges at abutments, 230 and 115 m long. They are composed of 22 m long precast post-tensioned I-shape girders. This bridge holds the highest traffic to the Szigetköz and to the direction Vámosszabadi-Medvedov (Slovakia) across the main Danube branch.

2.2 Bridges to the Esztergom Island

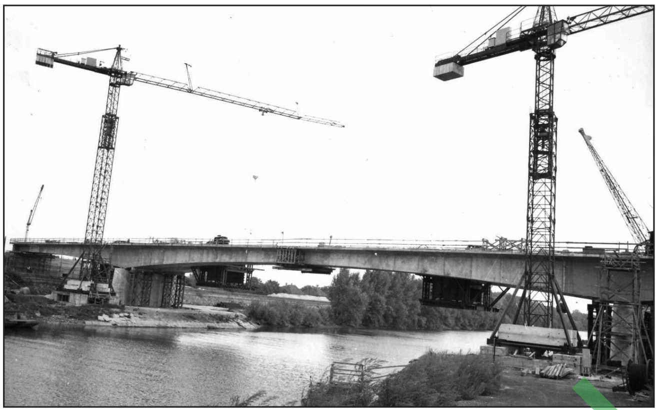


Fig. 2: The Széchenyi Bridge at Győr

The Mária Valéria Bridge at Esztergom has its right bank abutment at the island named after the city. It is called Prímás (Archbishop) Island, too. The steel truss bays of the main branch bridge connecting Esztergom to Štúrovo (Slovakia) was constructed at the end of the 19th century destroyed late 1944 and reconstructed in 2001.

A new steel frame bridge was constructed at the southern part of the island to unload the city centre of the traffic coming from the main branch bridge,

There is a concrete pedestrian bridge leading to the island.

The access way to the Mária Valéria Bridge crosses the eastern narrow Danube branch. From the city to the island the Vak Bottyán Bridge leads to the island. It is a compound concrete structure. The double cell girder serves for a 2.4+7.0+2.4 m wide deck. The spans are 12.0+40.5+13.0 m, with two hinges in the middle bay. Under one side bay there is a walking way. The other side opening is closed and filled by earth.

There is a municipal road bridge, the Kossuth Bridge connecting the city to the island. This steel structure serves now only the pedestrians.

2.3 Bridges to the Szentendre Island

2.3.1 The Szentendre Island bridges in general

There is a single bridge existing nowadays from the right river shore to the island, across the Szentendre Danube branch at Tahitótfalu. The first bridge was constructed in 1914. It was a steel truss structure, which has been destroyed during World War II. After the war it was reconstructed in the same manner, but later this structure was not wide and strong enough for the traffic. In 1978 a new composite structure was built, as described in Chapter 2.3.2.

Let us mention here that there is a small island, the Pap (Priest) Island in the Szentendre branch of the river, at Szentendre City. The bridge to this island is a small timber structure with steel parts. The significance is rather because of the recreation area on the island.

There is a bridge under construction close to the southern peak of the island. This will be the northern M0 motorway (circular motorway around Budapest) across the Danube. The access bridge bays (under construction in 2006-7) will be concrete girders. The openings across the main (Vác) branch will be cable stayed structures with steel stiffening girders and concrete towers with the main span of 300 m and the Szentendre branch superstructure is designed as a steel girder. Because of environmental protection aspects, although this bridge will have supports on the island, no access to the island will be constructed. Therefore, as designed, an individual, probably concrete bridge will be built between the city of Szentendre and the Szentendre Island.

2.3.2 The composite girder bridge at Tahitótfalu

The bridge at Tahitótfalu was constructed in 1978. It is a three-span (60+80+60m) steel-concrete composite construction on old piers. The two main girders, the steel bottom plate and the reinforced concrete plate form a box section. The concrete plate was prefabricated, in parts of 11.0 x 2.5 respectively 3,0 m. The gaps between these parts (15 respectively 65 cm) were cast in situ. Near to the piers the concrete plate was prestressed (Darvas, 1979).

2.4 Bridges to the Folk Island

The Folk (Mosquito) Island is now a peninsula because a dam was built at the northern peak across the Újpest Danube branch.

The Budapest-Esztergom railway line crosses both the main (western) branch of the Danube and also the Újpest river branch, leading on dam across the island. The original 1896 built steel truss structure was destroyed late 1944. Since 1953 a three storey bolted steel military “K” bridge crosses the main branch, and a continuous three bay steel truss structures the Újpest branch constructed also in 1953, providing pedestrian and bicycle crossing to the island from both riversides (Gáll, 1995). Not far from this structure, in 1973 a steel bridge was built at the southern end of the island for a fresh water pipe and for pedestrians.

In 1997 an interesting communal bridge was constructed across the Újpest branch. A hybrid bridge for a wastewater pipeline was constructed. The double reinforced concrete piers and abutments support steel cantilevers which work together with the self carrying tube. (Csíki, 1997).

2.5 The bridges to the Shipyard Island

At the northern part of the Shipyard Island a 98 m span three storey K (military) steel truss highway-railway bridge leads from the right (Óbuda) river side to the island. The bridge was constructed in 1955, the sidewalk cantilevers in 1973. This bridge leads to the famous yearly “Island Festival” attracting many dozen thousand young enthusiasts.

At the southern peak of the island, the Budapest Árpád Bridge has supports, but no access bridge is leading to the island.

To the North, close to the Árpád Bridge there is a concrete bridge leading to the island. Originally, there was a lift able steel structure at this place to serve the ship traffic at time of high water level. After 50 years use the lifting was no more needed, the structure was fixed. In 1968, instead of reconstruction, the steel bridge was demolished. A new concrete compound girder structure was built. The opening is 55.6 m.

2.6. The bridges to the Margaret Island in Budapest

Fig. 3: The interchange structure of the Árpád Bridge at the northern peak of the Margaret Island

Fig. 4: The Margaret Bridge with access to Margaret Island

The Árpád Bridge (built in 1950 and widened in 1984) is a continuous steel girder structure. It touches the northern peak of the island. There is a concrete access bridge system which enables the undisturbed traffic to and from the island (*Fig.3, Google Earth*).

At the southern peak the Margaret Bridge crosses the river connecting also the island to both parts of the Hungarian capital. The bridge was originally constructed in 1876, widened in 1937. The specialty of the bridge is in connection with the island. Namely at the southern peak the bridge has a 30° turn in ground plan. It is interesting that the island access side branch of the bridge was only constructed later, in 1900. The bridge was partly destroyed by the German troops under traffic, in November 1944, fully in January 1945. The island access bridge avoided the catastrophe and it shows even now the original structural system of the bridge, while the steel arches of the main bridge received a new two-hinge form during the reconstruction in 1947 and 1948 respectively.

The steel structure has reinforced concrete cantilevers for the sidewalks. The originally steel bays above the embankments were replaced by a structure with precast prestressed pre-tensioned girders. *(Fig. 4, Google Earth)*

2.7 The Csepel Island bridges

2.7.1 The Csepel bridges in general

At the northern peak of the island two bridges serve the traffic, a bridge for suburban railway and a highway bridge.

Another bridge is the Gubacsi Bridge, which has a steel truss, for four highway lanes and a railway track.

The southern section of the M0 circular motorway is led across the island. There is a composite continuous bridge (steel – prestressed concrete box girder) over the main branch of the Danube at Háros. This has a length of 770 m. The main spans are 108 m long. Over the Soroksár-branch a prestressed concrete bridge was constructed (see Chapter 2.7.2.).

An interesting concrete arch bridge was built for suburban railway (see Chapter 2.7.3.) At Ráckeve, a steel truss bridge is crossing the Danube branch.

2.7.2 The bridge of the southern section of the M0 motorway across the Soroksár Branch

The bridge is situated at the southern part of the motorway around the capital (*Fig. 5*). It was completed in 1990. The river bed spans are 37+75+37 m. The structural system is similar to the Moson Danube bridge at Győr (see Chapter 2.1.2.). The width of the superstructure is 17.5 (without sidewalks). There are two access bays at the left, and twelve at the right side abutment. These bays have 25 m span each. The access bridge structures consist of factory made I-shaped prestressed concrete girders with a continuous cast-in-situ slab (Tassi, Ódor, Fáy, 1993).

Fig. 5: The Soroksár Bridge

2.7.3 Suburban railway bridge at Dunaharaszti

A steel truss bridge with a 50 m bay existed where the Ráckeve suburban railway line crosses the Soroksár Danube branch. This single track bridge was destroyed during the war.

A new type of concrete arch railway bridge with ballast was built in 1949. (*Fig. 6*). This is also for one track and a sidewalk is situated as cantilever at one side out of the arch. The span is 52 m, the rise is 8 m. The structural system is arch with stiffening girder. A relatively high cube strength of that time for cast-in-situ concrete was produced. The main specialty was the construction procedure. At the top of the arch a hinge, under it, in the stiffening girder a gap was left, and also free tie-like reinforcement has been running along the track level. This way, under dead load, the structure worked as three-hinge-arch with tie. After concreting the temporary hinge and stressing the longitudinal reinforcement, then casting concrete into the gap of the stiffening girder and producing bond between the girder and the previously free reinforcement, concluded to a prestressed concrete structure for live load. In this means, this was a very early application of prestressing (Evers, Forgó, 1980).

Fig. 6: The Dunaharaszti suburban railway bridge

3. CONCLUSION

Although the distances between the river shores and river islands are generally not as long as those between mainland and sea islands, it is interesting to give a survey of river bridges in such situation. There are cases when the bridge layout is defined especially by the situation that it leads to a river island.

There are valuable bridges at the Hungarian section of the Danube. Several times, to find the optimum area for river crossing, the consideration of environmental protection play a definitive role for the designers. It is worthwhile to study river side–island bridges concerning bridge construction history.

4. ACKNOWLEDGEMENTS

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APPLICATION OF HIGH PERFORMANCE CONCRETE IN THE BRIDGE DECK OF A PEDESTRIAN BRIDGE WITH BASKET HANDLE



Dr. János Farkas – Imre Németh – Rudolf Korpás

The article discusses the static and dynamic problems of a prestressed, high strength/high performance (HS/HP) concrete bridge deck with basket handle. Working for Vegyépszer Co. and Mahíd 2000 Co. we have gained experience in the construction of basket handle bridges and in the planning and construction of HS/HP concrete bridges. In this paper we are presenting our experience in construction, planning and execution for a bicycle track bridge for a local government. Apart from static issues, problems related to structural dynamics are also discussed.

Keywords: high performance concrete, bridge with basket handle, prestressed concrete main beam, dynamics

1. INTRODUCTION

The site of the planned bicycle track is an old railway dam out of service. The bridge to be implemented crosses the river Zala at Zalaapáti, and shall be constructed on an existing foundation of a demolished railway bridge. The longitudinal and transversal tracks were pre-defined by the abutments to be used. The central span of 42 m could be spanned with an arch bridge without new piers and without disturbing the existing dams. The lower level of the bridge structure is 110.72 mBf (metre over the Baltic sea), crossing the axis of river Zala at an angle of 90º. The crown width of the bicycle track is 3.00 m at the bridge. The useful width of the bridge is 2.40 m, its total width is 4,90 m. The pavement surface slope in cross section is 2.5 % in two directions, that of the longitudinal section is 1.0 %.

The arch structure is made of tube sections with a nominal diameter of 600 mm, the material used was S355NL. The arches are connected by horizontal transverse beams at the top are made of S355 NL tube sections with a nominal diameter of 300 mm. The bridge deck is connected to the arch by skew mediating bars with forked suspension bars connecting to the S460NL steel sections constructed on the streamlined, cigar shaped transverse beams. The connection of the arch and the suspension bars is similar to that of the transverse beam and the suspension bars. The suspension bars are S355 steel tube section structures with non rigid connections at the end.

The suspension bars are not parallel but have a triangular arrangement coplanar with the arches. This way the residual rigidity of the structure was increased.

The thickness of the prestressed high strength and high performance (HS/HP) reinforced concrete slab is 54 cm, its thickness/span ratio is 0.54/42, i.e. 1/77, which means it is a remarkably slender structure.

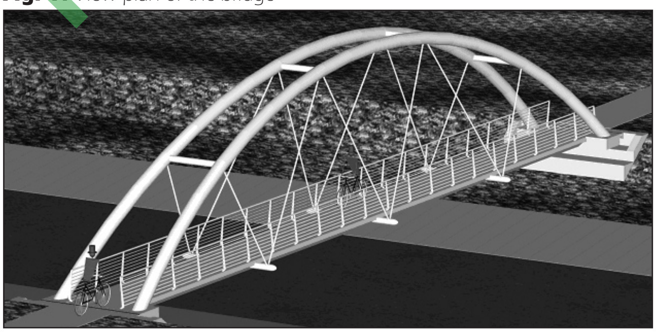
The superstructure has five transverse beams: two reinforced concrete beams at the ends and three steel beams in between. The transverse beams at the end of the superstructure are supported by two neoprene bearings each. The bridge deck is connected to the bridge abutment with expansion joints.

A 1400 mm high dip galvanized steel guard-rail is built along both sides of the bridge.

During bridge construction, prefabricated elements are transported to the site and assembled at the existing dam of river Zala. The assembled steel structure is hoisted by a crane truck, with temporary beams installed for additional rigidity. The reinforced concrete deck is constructed using temporary piers in the flood area of river Zala, and with single-phase concreting from a scaffold constructed using strong double-T beams above the river bed. After the compensation of the tension forces produced by the curve, the formwork is removed from the structure and the remaining elements (e.g. guard-rails) are installed and added to the finished structure.

Figs. 1, 2, 3 and 4 show the view plan of the bridge; the bridge model with springs representing the temporary piers; the cross section of the bridge; and the profile of the bridge is to be seen, respectively.

Fig. 1: View plan of the bridge



2. REQUIREMENTS APPLIED

The applied load chosen was 5 kN/m^2 in accordance with Hungarian Standard ÚT 2-3.401.

The prestressed HS/HP concrete bridge deck was designed on the basis of „5/2004 Építőipari Műszaki Engedély” (Construction Technical Permit) compiled for NA Rt., the National Motorway Agency.

The loading and stability tests of the arch and the suspension bars were carried out on the basis of Hungarian Standard ÚT 2-3.413 „Közúti acélhidak tervezése” (Design of public steel bridges). Owing to the chosen bridge construction additional dynamic tests had to be carried out.

3. PARAMETERS OF THE HS/HP CONCRETE AND OF THE PRESTRESSING SYSTEM

We needed a prestressed HS/HP reinforced concrete bridge deck for three reasons:

- Firstly, to avoid the excess weight of the insulation and of the asphalt pavement, which is unnecessary for the prestressed HS/HP concrete bridge deck, and this way to construct the simplest and most economical structure possible;
- Secondly, the maintenance budget of the bridge was expected to be rather low because the bridge will be managed by the local government, so the durability features of high performance concrete can be utilised to the full, resulting in the reduction of operation and maintenance expenses of the local government;

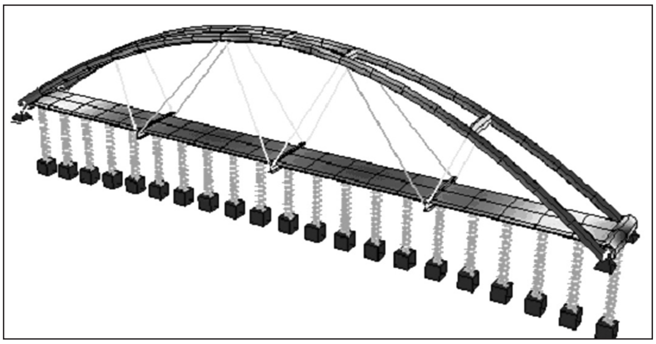
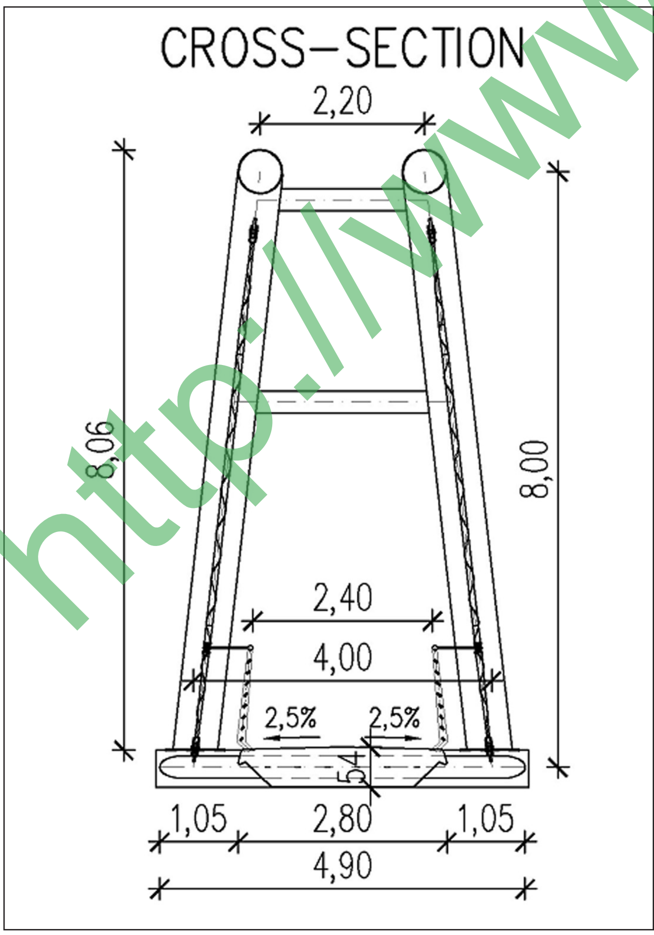


Fig. 2: Model of the bridge with spring supports modelling the entire scaffolding

Fig. 3: Cross section of the bridge



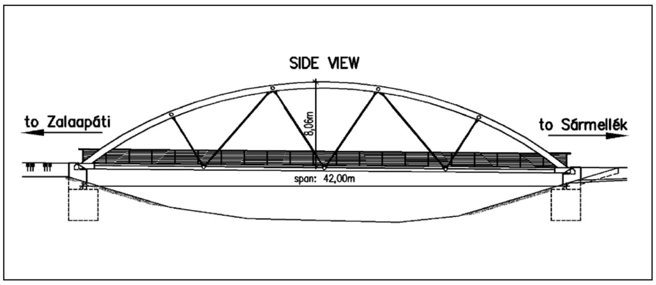


Fig. 4: Profile of the bridge

- Thirdly, as opposed to a light steel structure main beam sensitive to dynamic stimuli, the eigenfrequency (and rigidity) of the main beam is increased by the application of a reinforced concrete main beam.

Notwithstanding the above, the tensile forces in the reinforced concrete bridge deck resulting from the arch structure had to be absorbed, this is why prestressing was employed.

Table 1 shows the composition of the concrete mix on the basis of the experimental bridge No. S-65 on Motorway M7, (Farkas, Kocsis, Németh, Bodor, Bán, 2006).

The type of the concrete mix used is C50/60-XK2(H)-XF4-XC4-16-F3; to reduce cracking, 0.9 kg/m^3 FIBRIN 1832 polypropylene fibre is added.

The prestressing system uses sliding tendons comprising singly extruded strands of wire. The type of wire-strand is Fp-15/1770, its modulus of elasticity is 200 000 N/mm^2. The total number of strands foreseen is 36, comprising 9 bands of 4 strands each. The cross-section of one strand is 1.5 cm^2, its failure load is 265 kN, the prestressing force is 186 kN at 70 per cent of the nominal failure strength; we reckoned with a maximum overstressing of 10 per cent and a friction coefficient of 0.06.

The wire-strands are prestressed from one end in a straight line along the centre of gravity of their cross-section.

4. STATIC CALCULATIONS

The „5/2004 Építőipari Műszaki Engedély" (Construction Technical Permit) stipulates two specific requirements (apart from the general ones) regarding prestressed HS/HP reinforced concrete bridge structures:

- In service state, all points of the reinforced concrete structure must be stressed;
- Load bearing, deformation and crack control must be demonstrated for the limit of load-bearing capacity.

The structure supported by scaffolding is simultaneously exposed to both the g_I weight of the dead load and of the prestressing. When the scaffolding is removed, the g_{II} weight of the dead load is supported by the final structural framework.

The required calculations were made on an elastic rod model; the time- and prestressing-dependent strength losses were considered both for load-bearing and service stages.

Loads and actions:

The calculations were made for the following load components (notation and brief description of loading components):

LF 1	dead weight on scaffold
LF 14	removal of scaffolding of the entire structure
LF 71, 72	prestressing loads
LF 140	dead weight for g_{II}

Table 1: Planned composition of 1 m^3 concrete

Description	Mass %	Volume percent	m^3	Mass kg
OH 0/4 Hejőpapi	39	40	0.278	735
UKZ 5/12 Uzsa	19	18	0.125	357
OK 8/16 Gyékényes	41	42	0.292	772
Aggregate total:			0.696	1864
Cement CEM II/A-S 42,5 N			0.136	420
Silica suspension Centrilit Fume SX	6		0.036	50
Water			0.115	115
Additive				
Muraplast FK 62.30 plasticizer	1.7		0.0064	7.1
Centrament Retard 310 retarding admixture	0.3		0.0011	1.3
Air			0.010	
Total			1.000	2457

LF 91	temperature difference –15 °C
LF 95	temperature difference +15 °C
LF 240	blast of wind for 3 kN/m^2 (from upstream direction)
LF 250	wind pressure for 1.5 kN/m^2 from upstream direction
LF 260	wind pressure for 1.5 kN/m^2 from downstream direction
LF300	service load at the load-bearing limit state at a reference value of 5 kN/m^2
LF301	service load at the service limit state at a reference value of 2.5 kN/m^2
LF 410	creep + shrinkage between t_0 and t_1
LF 420	creep + shrinkage between t_1 and t_2
LF 430	creep + shrinkage between t_2 and t_3
LF 440	creep + shrinkage between t_3 and t_4
LF 450	creep + shrinkage between t_4 and t_∞

The total value of creep and shrinkage used was 1.8 and 0.0002, respectively. The creep and shrinkage processes were modelled as follows: the total period under examination was divided into 5 parts, assuming that one fifth of total creep and shrinkage takes place in each.

Load components were used to determine the standard load combinations, which, in turn, were used to calculate the bending moment and stress analyses were also completed. The calculations were made both with and without considering creep and shrinkage. In load-bearing capacity calculations the prestressing force was considered as a single external force, the ultimate moment was calculated with the stress/strain diagrams of C50/60 concrete and steel reinforcement.

After the evaluation of static calculations the following conclusions were drawn:

- the reinforced concrete bridge deck and the arch have an appropriate load-bearing capacity,
- in the service stage only compressive pressures are produced in the reinforced concrete bridge deck (*Fig. 5*),
- the steel structure arch has appropriate buckling characteristics.

5. INTRODUCTION TO DYNAMIC CALCULATIONS

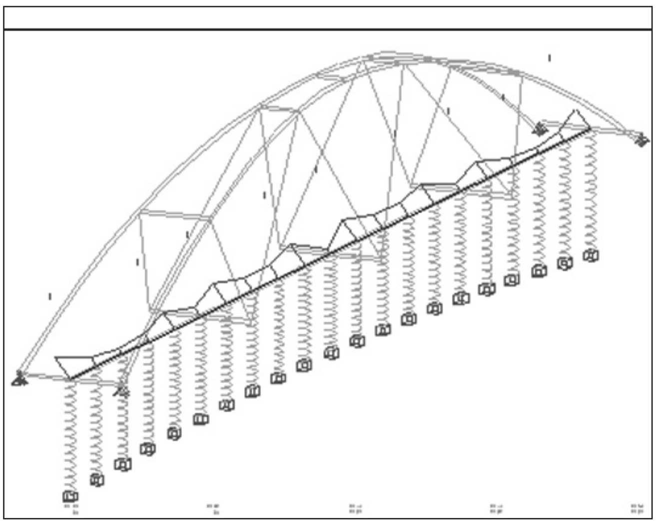
Section 3.6 of Hungarian Standard ÚT 2-3.401 specifies vibration testing of cable stayed girder (or similar) structures. A structure having basket handle, a reinforced concrete bridge deck, and skew suspension bars belong to this group. Section 4 of Hungarian Standard ÚT 2-3.413 „Kiegészítő előírások a közúti hidak tervezéséhez" (Additional requirements for design of public bridges) discusses the requirements pertaining to the cables of cable-stayed and prestressed steel bridges („Függesztett és feszített acélhidak kábelei"). These two sections underline why calculations reflecting the dynamics of the related phenomena are necessary.

In the case of the construction under review here – compared to solid bridges –dynamic calculations are essential, because very often the stresses resulting from dynamic calculations have to be applied during the design phase.

We carried out the *load-bearing check* of prestressed concrete beams in accordance with the previous section.

A traditional fatigue test for the bicycle road bridge was not carried out. (However, as far as wind load in the suspension bars is concerned, the extreme values and the variation of the cumulative stress can be determined with the following methods. Then, knowing the number of windy days – with certain assumptions – the fatigue calculations could have been

Fig. 5: Stress distribution of the reinforced concrete bridge deck in service state at $t_{infinite}$ time



carried out on the basis of cumulative deterioration principle. However, they were not made at this point.)

The stress analysis of the *service limit state* was carried out according to the previous section, too. It was revealed that in the prestressed concrete bridge deck, in the service state limit only compression occurs.

The *stability check* was performed by examining the direction of reaction forces. Results show that reaction forces always have a vertical direction.

Examination of *deformation*, i.e. motions generated by dynamic effects (and related stresses) is described below.

Assessment of the individual construction states was practically completed in the previous section by assuming that the entire concreting and prestressing will be performed with the scaffolding in place.

As far as *extraordinary* load distribution is concerned, earthquake load is determined according to EC 8 below.

Therefore, for this particular bridge the dynamic calculations meant *deformation* (with related cumulative stress) and *extraordinary (earthquake)* examinations.

In the following, brief summarized definitions of some dynamic phenomena and aerodynamic features are given.

Flutter means the effects of twisting and bending oscillation caused by wind load on a structure with an aerodynamically unstable cross-section; galloping means the effects of bending oscillation. The *von Kármán* turbulences are vortices periodically shed by aerodynamically stable (e.g. circular) structures causing lateral swing to the structure. The amplitude of the lateral swing and the shape coefficient of the structure depend on the Reynolds number. Its value is directly proportional to the diameter of the circle and to the critical speed dependent on the eigenfrequency and inversely proportional to the viscosity of air. The Reynolds value has no unit of measure. Depending on the Reynolds value, shape coefficients can be found in the diagrams prepared on the basis of the wind channel tests.

The Scruton value is an aerodynamic property of cross-section. It is directly proportional to the volume and the damping of the structure, and inversely proportional to air density and the squared value of the diameter. It has no unit of measure. Further dynamic examinations were done for the individual basic structural elements of the bridge and not for the whole structure.

Eigenfrequencies were determined assuming that the whole bridge structure was a rod structure. The effect of flutter was examined in a simplified way on the basis of eigenfrequencies.

Periodic pedestrian load was not dealt with because the eigenfrequencies of the bridge are beyond the limit of 1.5-2.5 (for more information see the Reference: *fib*, 2005).

We only describe the examination of those effects of the earthquake component that are perpendicular to the bridge axis. For the practical calculations of earthquake examination of structures see Kollár (Kollár, Sajtos, 2004).

The skew suspension bar was examined for the following phenomena of wind dynamics:

- the aerodynamically stable tube structure was examined for excitation by von Kármán's vortices on the basis of prior evaluation of the Reynolds value,
- the supposedly aerodynamically unstable structure with a degree of freedom of 1 was examined for galloping bending forces.

As both examinations required the first eigenfrequency of the suspension bar, it was first calculated with the appropriate software product.

6. DYNAMIC CALCULATION MODELS AND SIMPLIFYING ASSUMPTIONS

The various calculations were made with rod models and models with a degree of freedom of 1 (mass, spring, damping).

The following simplifying assumptions were made:

On the basis of eigenfrequency analysis we found that the bridge is not excited by pedestrian and bicycle traffic, and flutter.

The calculation of earthquake vibrations perpendicular to the bridge axis was made pursuant to EC 8 and using the response spectrum method.

The vortex shedding of suspension bars with an assumed degree of freedom of 1 was calculated in line with Petersen (Petersen, 2001) using the Scruton value, v_{kr} (critical wind speed). The critical pressure on the suspension bars was also calculated.

The pressure results in a reversible combined stress perpendicular to wind speed. According to literature (Petersen, 2001) the Scruton value should be higher than 25, otherwise a structural modification is needed, e.g. a spiral spine on the tube, concreting, etc. Based on the results of preliminary tests the tube of the vertical bar was filled with concrete, and a spiral spine was added, whereby the load, the eigenfrequency, and the damping could be increased as well.

However, with the addition of the spiral spine the circular section, which had previously been aerodynamically stable, became aerodynamically unstable, susceptible to galloping and the shape coefficient also increased. The shape coefficient can be determined either by a wind channel or a CFD (Computational Fluid Dynamic) test. The shape coefficient function is calculated on the basis of the air flow angle in the wind channel. Using this nonlinear function the $\sigma_{derivate}$ value can be determined (Petersen, 2001), and also the v_E equivalent critical speed can be calculated. In the formula of the v_E equivalent critical speed $\sigma_{derivate} = 3$ was used for the square cross section with sides equalling the diameter. Using this value the v_E (average) speed was determined and compared with the average speed of v = 30 m/s typical of the area.

7. DYNAMIC CALCULATIONS

Calculation results indicated that the twisting and bending oscillation of the superstructure was 4.00 and 9.98, respectively.

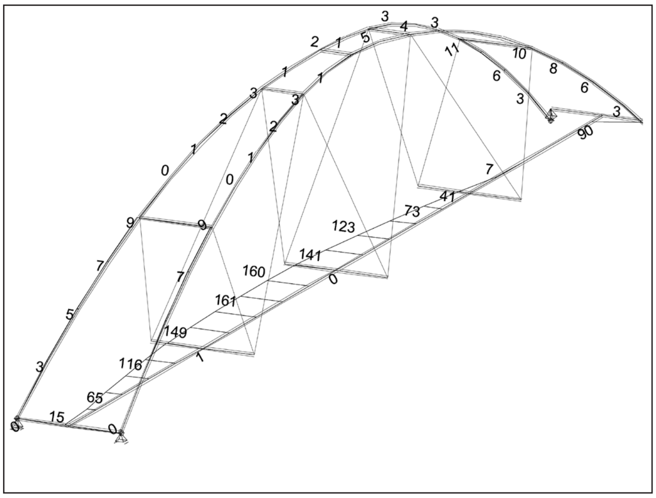


Fig. 6: Bending moment in kNm on the main beam, typical of the Zala area $a_y = 0.2$ g acceleration (not limiting, because less than the bending moment originating from 1.5 kN/m² wind)

Table 2: Analysis of vortex shedding parameters of a bicycle track bridge

Structure: Vertical stick with circular cross-section		
I. Determination of Scruton-value (Sc)		
f_1 value of the system (frequency 1)	f_1=	3.240 Hz
Diameter of the structure	d=	0.120 m
Strouhal-value	S=	0.20
Critical wind speed v_{kr}	v_{kr}=	1.94 m/s
Kinematic viscosity of air		0.0000125 m^2/s
Generalized mass of the structure	m_1=	19.50 kg/m
Supplemental generalized mass (e.g. concrete)	m_2=	6.40 kg/m
Alleviation according to A_1=log.decrement DIN 4131/4133	A_1	0.02
Supplemental alleviation of concrete: A_2=log. decrement	A_2	0.04
Air density	ρ=	1.25 kg/m^3
The calculated Scruton-value Sc=	Sc=	170
Scruton-value > 25? Yes		
II. Determination of Reynolds-value		
Reynolds-value =	Re=	15520.00
Shape coefficient (its value can be determined from the diagram Re-c_{lat}, e.g. by Petersen)	c_y=c_{lat}=	0.70

Table 3: Approximate galloping dynamics calculation of the suspension bar of a bicycle track bridge

Vertical rod = (circle cross-section+spiral line) = square cross-section		
I. Determination of Scruton-value (Sc)		
Length	l=	8.0 m
External tube diameter	D=	12.0 cm
Tube skin thickness	t=	0.7 cm
Internal tube diameter	d=	10.6 cm
Relative mass g=M/l	g=	19.50 kg/m
Frequency generated from Sofistik software.	f_1=	3.240 Hz
II. Determination of parameters:		
Strouhal-number (aerodynamic parameter)	S=	0.20
v_{kr}=	v_{kr}=	1.94 m/s
Kinematic viscosity of air		0.0000125 m^2/s
Generalized mass - average	m_1=	19.50 kg/m
Generalized mass of supplemental concrete	m_2=	10.00 kg/m
Attenuation of steel tube according to A=log. decrement DIN 4131/4133	A_1	0.02
Attenuation of supplemental concrete A2	A_2	0.04
Air density	ρ=	1.25 kg/m^3
Sc	Sc=	196.65
Scruton-value > 25 ? Yes		
III. Galloping test		
$\sigma_{\text{derivation}}$ – (according to Petersen 2001) for square cross-section	σ=	3.00
Height of cross section	b=	0.12 m
v_E=	v_E=	30.58 m/s
Average wind speed in the area	v_{ave}=	30.00 m/s
Safety parameter	n=	1.02
>1,00?? Yes		

Flutter is a very complicated dynamic phenomenon whose effects can only be estimated. It usually occurs at large-span, slender structures (e.g. Tacoma bridge) when twisting and bending forces are combined. Its effects can be reduced by installing skew suspension bars that increase the total rigidity of the bridge. Elimination requires a practical approach (Petersen, 2000), i.e. the examination of the proportion of twisting and bending oscillations. In our case it is 9.98/4.00 = 2.49 > 2.00 which means the bridge structure is suitable as far as flutter is concerned.

Earthquake vibrations parallel with (x) and perpendicular (y) to the longitudinal axis of the bridge are examined separately; combined earthquake vibrations result in loads determined in using various formulae (e.g. the square root of the sum of squares of loads perpendicular to each other). Design calculations can be based on these cumulative stresses. In the present paper only stresses developing on the superstructure with an acceleration factor of 0.2 *g* perpendicular to the bridge axis are discussed (Figure 6).

The oscillations of the suspension bars generated by the von Kármán turbulences on the model with a degree of freedom of 1 were controlled by monitoring the Scruton value. The tube section vertical bar was partly filled with concrete, and a spiral spine was added. Calculations are summarised in *Table 2*.

The tube section with the spiral spine was supposed to be aerodynamically unstable and was substituted with a square section, and a critical wind speed of v_E speed was used. Safety margin results from the difference between v_E critical wind speed and the wind speed typical of the area. For the required safety, further concreting was needed to increase mass of the structure. Calculations are summarized in Table 3.

8. RESULTS

A bicycle track bridge with basket handle was designed on existing abutments. Our experience in the field of planning and construction of large bridges was used in the case of a minor bridge managed by a local government.

For economical, maintenance and dynamic reasons, a HS/HP reinforced concrete bridge deck was designed, which was also prestressed to absorb horizontal stresses of the arch. Discussions with the client revealed that local governments are especially interested in using high performance and enduring concrete types and, consequently, in the reduction of maintenance expenses in the long run.

Standard static calculations of the bridge were made with a view to the different construction states, creep, and shrinkage.

Attention was paid to the issues of dynamics raised by a slender, crack-free and low-damping bridge. For the bridge, the effects of pedestrian excitation, wind excitation (flutter) and earthquakes were analysed, and, as far as the suspension bars of the bridge are concerned, the von Kármán vortices and simplified galloping tests were performed.

The effect of pedestrian excitation and flutter could be eliminated on the basis of the eigenfrequency. Earthquake examination results were not limiting.

Regarding the oscillations caused by von Kármán vortices, the aim was to increase the Scruton number to an empirically accepted value above 25. This was attained by partly concreting the tube and by the addition of a spiral spine.

The aerodynamically unstable tube structure was approximated by a square section. Taking the concrete filling and the addition of the spiral spine into consideration, the critical wind speed was determined and was found to be faster than the wind speed range typical of the area.

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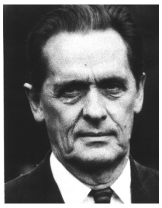
THE 3000 M^3 CAPACITY WATER TOWER AT BUDAFOK, HUNGARY





Prof. György Farkas – Tamás Kovács – József Thoma

A new, 3000 m^3 capacity water tower is under construction on top of the Tétényi-plateau at Budafok, XXII. district of Budapest, Hungary. Besides fulfilling many functional requirements in the water supply network, this concrete tower will represent a usual engineering structure with many unusual structural and architectural solutions. The structural curiosities of this tower are the eccentricities between the centres of gravity lines of the base, the shaft and the water tank.

Keywords: water tower, water supply system, eccentricity, shaft, climbing formwork, construction technique, water tank, horseshoe-shaped cross section, spherical shell, precast panel

1. INTRODUCTION

In 2001 The Water Works of Budapest as investor announced a public competition for designing a 3000 m^3 water tower onto the top of the Tétényi-plateau at Budafok, Hungary. The company expected such architectural-structural solutions that

- fully fulfil the functional requirements, such as
  - providing the necessary resources for the increasing water demands of the community,
  - equalizing the daily fluctuations of the water consumption in summertime,
  - significantly reducing the risk in the continuity of the water supply in the hilly region by providing and maintaining a constant pressure in the water supply network,
  - helping in the optimization of the energy input and allowing energy-saving operation by providing a considerable reservoir capacity at the top of the covered area,
  - providing resources for the water demands of the fire-service.
- can be the symbol of the company,
- have an aesthetic appearance,
- fit into the atmosphere and the environment of the surrounding region,
- become a permanent architectural product of the quickly developing neighbouring district.

It can be noted that the water towers as the typical structures of the water supply system have gone out of fashion and became classical objects by today. Nowadays, the above original water supply operational requirements, for whose fulfilment the water towers were the perfect engineering solutions in the past, can be assured by high performance pressure booster pumps and similar mechanical devices. Another alternative for this function could have been a water tank installed at the same place below the ground level but it would have had no architectural and aesthetical value. Choosing a tower instead of a tank and one of the possible alternatives shows the strong commitment of the investor in building a structure that combines the function with the architectural value.

The winner design (*Fig. 1*) was selected from among the plans of 22 competitors by an invited professional jury including many architects and structural engineers. The authors and all other data regarding this competition can be found in www.viztorony.hu.

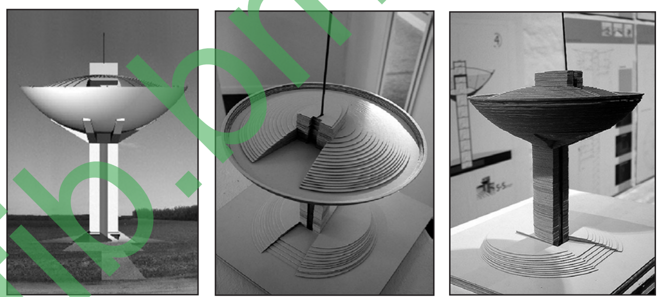


Fig. 1: Virtual and architectural models of the winner design competitive (source: www.viztorony.hu)

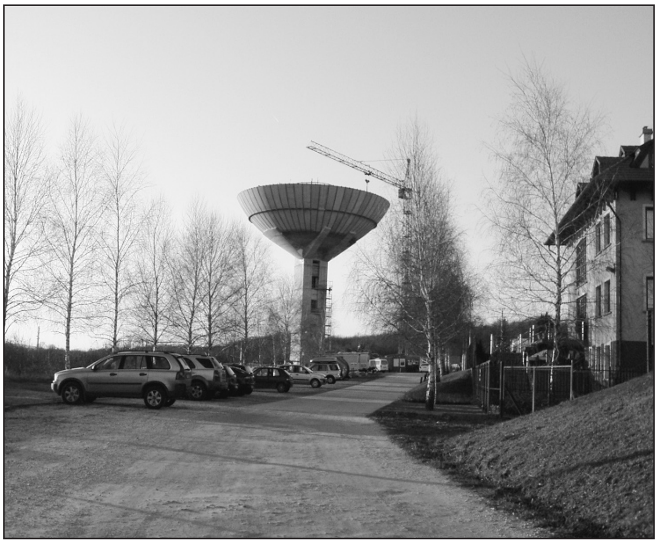
2. MAIN DIMENSIONS AND PARAMETERS OF THE LOAD BEARING STRUCTURE

The *Budapest University of Technology and Economics, Department of Structural Engineering,* as the main supervisor of the investor, played a significant role in both structural design, which was made by the *Mélyépterv CE Ltd.*, and the full execution process of the tower.

The load bearing structure of the tower is fully made of concrete. It consists of a 3.5 m thick, flat, circular base slab with a diameter of 19 m, a 23 m high shaft with a horseshoe-shaped cross section whose overall dimensions are 7.00×8.45 m and a spherical cap-shaped water tank having an outer radius of 21.4 m. The diameter of the largest horizontal section at the top of the tank is nearly 36 m. The total height of the tower is 42 m.

Approximately 1900 m^3 of concrete in total will be used for the entire load bearing structure of the tower, which results in specified concrete input of 0.605 concrete m^3 per stored water m^3.

The roof of the water tank has a large (~100 m^2) look-out terrace at approximately 34 m height above the ground level. According to the operating plans, this terrace will be open for public all year. From this place a beautiful view of Budapest



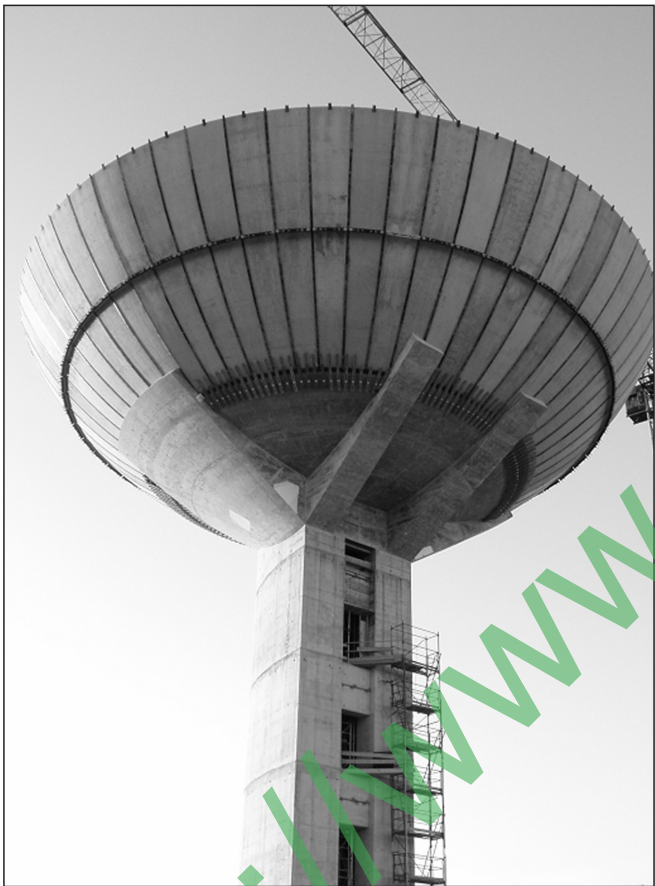


Fig. 2: Construction phase after removing the formwork

is visible. This terrace includes a sector-shaped part above the water tank linking to a circular part, which follows the top perimeter of the water tank from inside (*Fig. 5*).

2.1 The foundation

The structure has been built on a flat footing therefore the circular slab base lies directly on the supporting ground. The load bearing subsoil is situated approximately 1.0-1.5 m below the ground level and consists dominantly of limestone with medium size pieces of rocks (*Fig. 3*). The surfaces of load bearing soil lie in a slope of approximately 2.0 degrees.

2.2 The shaft

The horseshoe-shaped cross section of the shaft is closed on its front wall at each floor (at every 3 m) by 2.0 m long, 2.3

Fig. 3: Excavation of the footing

m high and 0.7 thick deep walls. This cross section does not change along the height (quasi-prismatic) but its thickness varies between 0.39 m (at connections to the front wall) and 1.2 m (at the tip of the horseshoe). The front wall is 1.4 m thick (*Fig. 2*).

The shaft includes 300 mm thick landing slabs at every 5.3 m and stairs between them along the inner perimeter of the cross section. The stairs lead via the centre of the water tank to the very top of the tower and have an exit to the look-out terrace. The shaft also includes a lift that runs between the ground floor and the mentioned look-out floor. Look into the water tank through windows are possible from the exit floor of the lift and from the look-out floor as well.

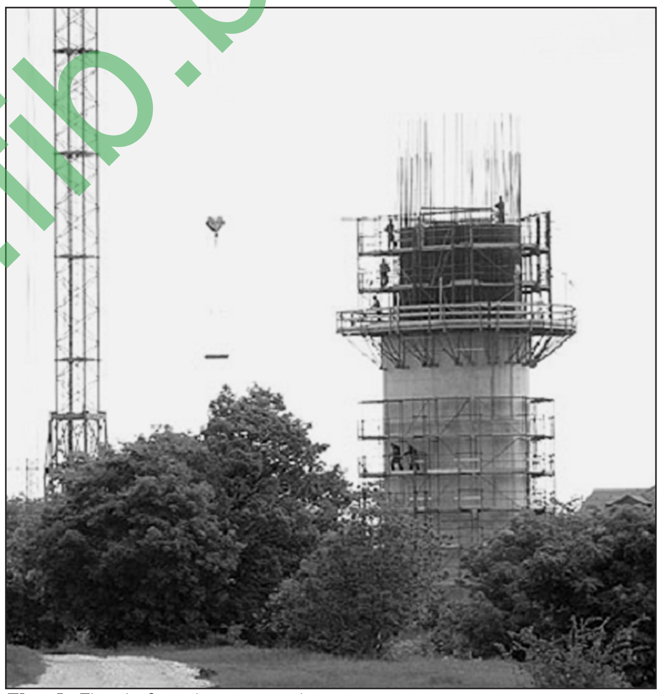


Fig. 4: The shaft under construction

Due to the prismatic cross section the shaft has been built by the use of two 5.3 m high climbing formwork segments (*Fig. 4*). The slip-form construction method was rejected due to difficulties in assembling the reinforcement into the unusual cross-section as well as the high number of openings along the front wall. The shaft has been built by a rate of two weeks per segment.

2.3 The water tank and its parts

The point symmetric, spherical shaped water tank has two water compartments with approximately 1100 and 2200 m^3 water storing capacity separated by a vertical, circular partition wall. Both compartments are point symmetric to the centre of the sphere (*Fig.5*).

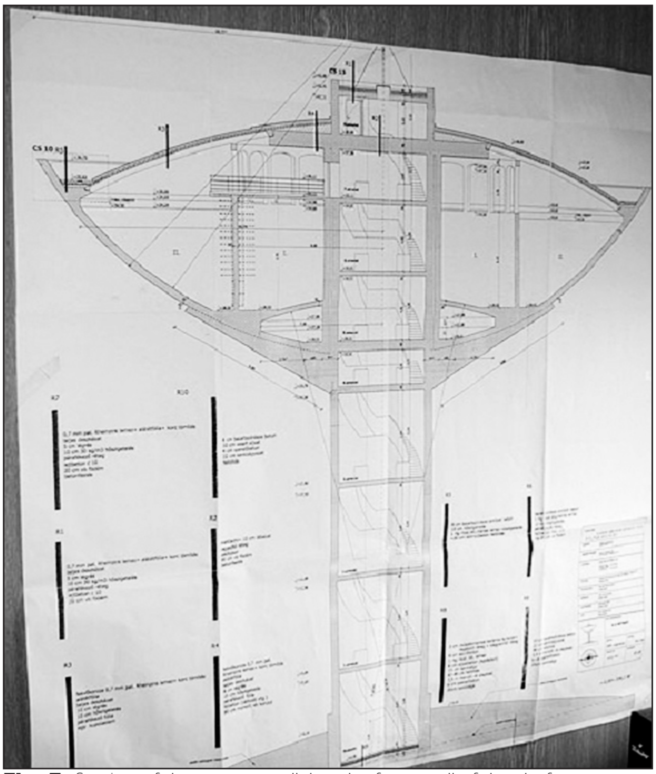


Fig. 5: Section of the tower parallel to the front wall of the shaft

The water tank is directly supported by the shaft through a monolithic connection between the shaft and the tank. Additionally, four cantilevers projecting from the shaft (*Fig.2* and *Fig. 5*) are also designed partly for providing additional, indirect support for the tank and partly for giving a special architectural view for the structure by symbolizing an imagined human hand holding the spherical water tank at the top of the tower.

Structurally, the water tank has an inner monolithic, posttensioned concrete core and an outer monolithic, post-tensioned, 0.4 m thick, concrete spherical shell (*Fig. 5*). The inner core is cast on conventional formwork in its final position while the outer spherical shell is cast to 70 mm thick, precast concrete formwork panels fixed to the inner core and the shaft before concreting. The post-tensioning system is made of individually led, circular VT-M 150 monostrands that are anchored on alettes from the water side of the core and the shell.

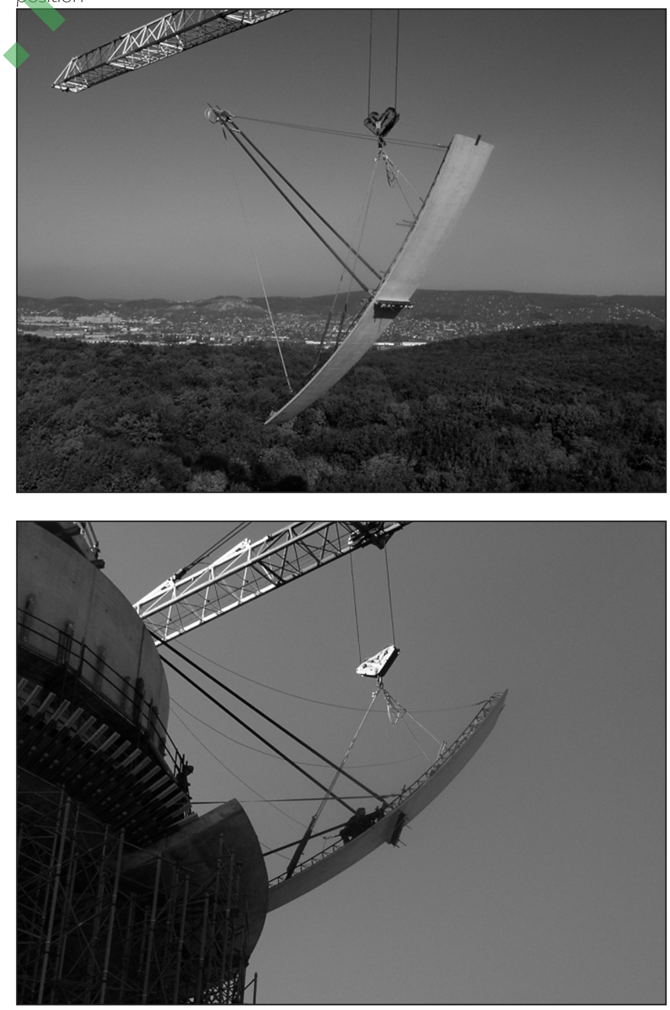
The inner core consists of a significantly strengthened ring inside the shaft, solid concrete blocks as the fixed ends of the projecting cantilevers at the bottom of the tank, another post-tensioned ring having a triangular-shaped cross section and forming the bottom of the water compartments and a posttensioned spherical shell above the assembly chamber (*Fig. 5* and *Fig. 6*).

The formwork panels and the upper concrete layer have a composite action due to three triangular-shaped truss beams per panel fully made of structural steel. The formwork panels embed the bottom flanges of these truss beams, by which the panels can be moved and fixed to the inner core and the shaft. These panels provide the spherical form of the water tank (*Fig. 7*) as well. The formwork panels also include the bottom reinforcement layer of the shell.

The upper ring of the tank is situated at the top of the tank and has many functions such as provides a stiff, structural ring

Fig. 6: The reinforcement mesh of the bottom slab and the formwork of the upper slab for the assembly chamber under construction

Fig. 7: An individual formwork panel during lifting and after fixing its final position



at the top for the shell, provides support for the roof system and the look-out floor and represents a circular footway as part of the look-out terrace.

The mentioned look-out floor is arranged above the water tank as part of its cap and is supported by the shaft itself, the partition wall between the inside and the outside water compartments and the upper ring on the shell of the water tank.

The roof system of the water tank consists of 70 mm thick precast concrete panels as segments of a sphere with an outer radius of 33.7 m. These segments cover 7.5° from the plan view of the total roof area. These panels are supported on their longer sides by arched, reinforced concrete roof purlins arranged radially along the surface of the above sphere. The roof purlins are supported by the upper ring of the water tank and the top slab of the shaft. The mentioned sectoral look-out terrace breaks this spherical roof system and provides support for the roof purlins along the connection lines.

3. DESIGN

The speciality of the tower is that the centre lines of gravity of the water tank, the shaft and the base do not coincide with each other (*Fig. 2*). The eccentricity of the water tank itself is 1.46 m measured positive from the centre of gravity line of the shaft, this eccentricity is 1.43 m for the weight of the water in the inside compartment and 1.17 m for the weight of the water in the outside compartment. The shaft also has an eccentricity of -1.20 m from the centre of gravity of the base. All the above eccentricities are considered in the symmetry plane of the shaft. The whole structure is symmetrical to this plane.

The other curiosity is the horseshoe-shaped cross section of the shaft and, as a consequence, the not exactly point symmetric geometry of the whole water tank. Due to this asymmetry in the supporting cantilevers of the water tank, the compression zone of these cantilevers could not develop along a horizontal circle. During a supposed lifting of the water tank, considerable cracking in the tension zone of these cantilevers, which is represented by the spherical shell above the assembly chamber, would have arisen that would have adversely affected its serviceability (water tightness) and durability.

For these reasons, the water tank has been cast right in its final position (i.e. at the top of the shaft) – as usual for many similar towers - instead of being lifted up from the ground after completion. In this case, the connection between the tank and the shaft could be concreted monolithically along the full length and the compression zones of cantilevers could be supported against each other right from the beginning of operation. Another consequence of this construction method is that the outer part of the water tank has been built by the use of precast formwork panels in order to reduce the required quantity of formwork.

The earthquake resistance of the tower has been verified by a complete dynamic analysis using a pendulum model and concentrating the weight of the water into the metacentre. The horizontal peak ground acceleration has been assumed to 0.08g corresponding to the Zone 3 according to the Hungarian National Application Document for MSZ ENV 1998-1-1 (Hungarian 2001).

4. MATERIALS, EXECUTION

4.1 Materials

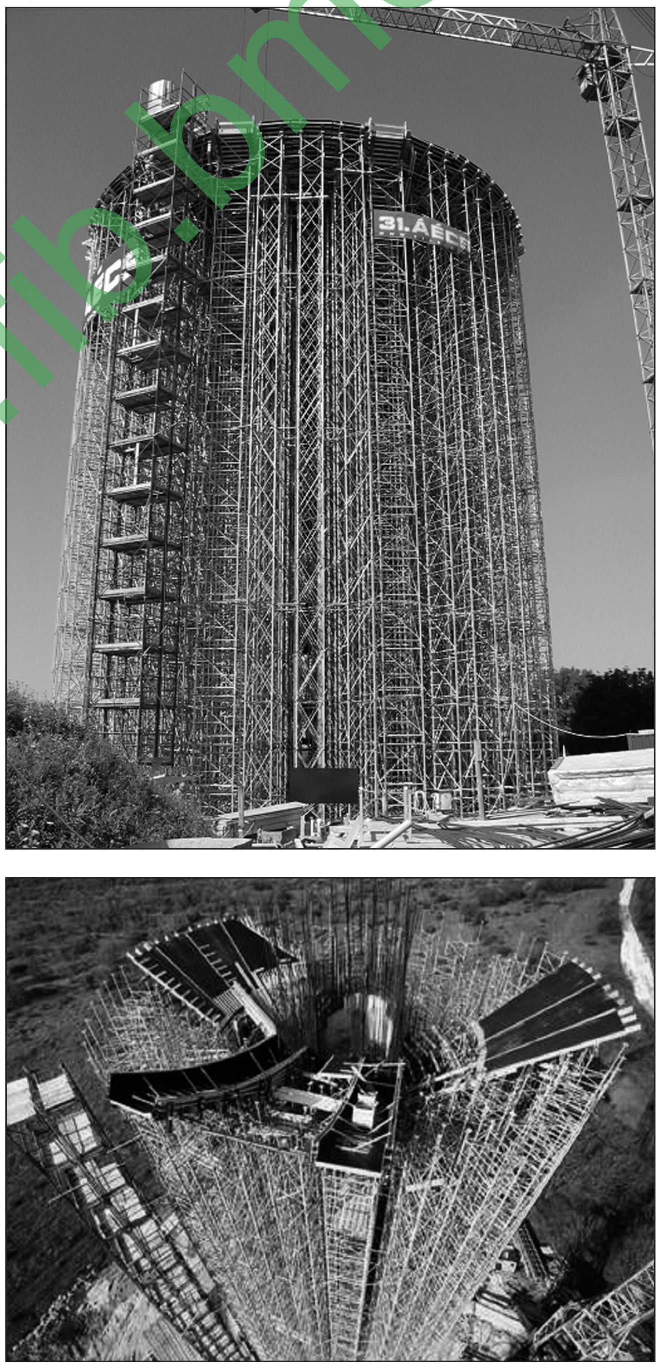
The monolithic base has been made of C25/30 reinforced concrete using conventional formwork. The other parts of the tower have been made of C30/37 concrete. S500B type reinforcing steel has been used for all precast and cast-in-place structural elements. The post-tensioning of the water tank is designed by the use of individually led VT-M 01-150 monostrands having a tensile strength of 1770 N/mm^2 and the corresponding standard VT-VMM 04-150 anchorages (one anchorage device per four strands).

4.2 Execution

The significant difference in the construction method between this tower and the usual concrete water towers is that this water tank is not lifted after completion on the ground but cast on formwork in its final position at the top of the shaft. As a consequence of this, the connection between the shaft and the water tank could be designed and executed by a fully monolithic joint.

The formwork has been installed partly on the base and partly on an individual concrete slab concreted around the base (*Fig. 8*).

Fig. 8: The formwork of the inner core of the water tank



Due to the considerable amount of formwork, the outer part of the water tank is cast in 60 pieces of precast panels (*Fig. 7*). These very slender panels had to be designed closely for all possible transient design situations such as under lifting, after fixing and subjected to wind actions, during the assembly of the reinforcement of the shell and during concreting the shell in more steps.

5. CONCLUSIONS

Besides fulfilling many functional requirements in the water supply network, this concrete tower will represent a usual engineering structure with many unusual structural and architectural solutions. The structural curiosities of this tower are the eccentricities between the centres of gravity lines of the base, the shaft and the water tank. On approaching to the end of the execution of the tower, it became obvious that significant amount of design effort and extra material is needed if the structural form differs from the rotational-symmetric shape compared to usual towers having this optimal shape (Márkus, 1966).

Hopefully, thanks to the interesting design solutions and the variety of the applied construction techniques, after completion this tower will not be only a part of the water supply system but it will hopefully symbolize the fruitful cooperation between the investor, the designer and the building contractor as well as it will become the symbol of the region.

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REINFORCED CONCRETE SLUDGE DIGESTERS IN HUNGARY IN THE LIGHT OF THE EXPERIENCES RELATED TO THE REALIZED STRUCTURES

Gábor Zoltán Péter

This article outlines the recently built digesters and the experiences of the construction of those. From the point of view of structural engineering the conclusions of the designed and implemented reinforced concrete sludge digester structures and the utilization of those at further designing tasks are of essential importance. Keeping the advantageous approved solutions adjusted to the different conditions, reviewing and improving the unhandy complex joints in some cases it is difficult to construct or implement economically good and at the same time „execution-friend" structure.

Keywords: digester, structure, structural joint, building technology, coordination

1. INTRODUCTION

Lately in Hungary a significant development has been in progress regarding environment protection in accordance with the European Union engagement. Sewerage and wastewater treatment can be particularly mentioned. The communal projects contribute evidently to the improvement of living conditions, potable water basis protection and the rehabilitation of environment.

The treatment and final disposal of the wastewater sludge accrued from wastewater treatment as a side product is an important part of the abovementioned developments, as well. Especially at the common wastewater treatment plants of big cities or regions the wastewater sludge originates in a big quantity, one feasible treatment of which is digestion. The biogas accrues in this kind of sludge treatment technology is a second energy source, the utilization of which has also come to the force. That plays significant role regarding the other possible alternative energy solutions, because that is continuously available due to wastewater treatment and the energy production can be carried out continuously. One of the well-accomplished solutions is the combustion in biogas engine producing electric energy.

One member of the abovementioned developments is the Mélyépterv Komplex Engineering Co., where experts of all touched professional lines take part. The starting point of the complex and complicated processes is the digester structure which needs great thoughtfulness regarding structural designing aspects. Upon the final designs made by the experts of our company the 10^{th} reinforced concrete sludge digester has been constructed in the time of the present publication. The continuous evaluation of the designing and construction experiences carried out within the company made possible the retrospective evaluation and to draw the conclusions concerning the big engineering structures executed under different undertaking conditions.

2. ABOUT DIGESTERS IN GENERAL

The technical and technologic conditions of the biogas production mentioned cannot be evaluated in the aspect of structural designer regarding its complexity, however some basic relations can be defined. The useful volume of digesters depends on the quantity of the wastewater sludge originated in wastewater treatment process. The total volume needed for digestion is suitable to be divided into at least two parts (pre- and secondary digester) due to water technologic respects. That has operational advantages, as well. However, one digester is frequently applied because of less sludge quantity or other technologic conditions. The digestion process is fairly constant due to the long retention time (15-20 days), so the technologic disturbances of shorter periods (1-2 days) can be bypassed. In some cases the significant sludge quantities of bigger wastewater treatment plants can be treated in more than two digesters, in which cases the number of digesters shall be defined upon the structural optimum determined by the structural designer.

The digesters are engineering structures of complex shell structures, the load cycles of which are affected by more than one factor. Regarding the topic of that an overall description can be found in the referred article of Péter–Tóth (1999).

In general it can be stated that the geometric configuration of the spatial structure determined in respect of the stress optimisation is of importance. The generally high pressure-column and the high temperature of digestion (35 °C or sometimes 55 °C) determine definitely the scale of the tension and bending stresses. In order to diminish the significant stresses and to keep the temperature needed for operation it is inevitable that the structures shall be equipped by thermo insulation.

The digester tanks are high buildings emerging from landscape and determining the view of the wastewater treatment plant. Due to their sizes they transfer significant load to subsoil. Consequently, the setting-in to landscape or environment and the founding of those shall be thoroughly taken into consideration. To meet the requirements of the suitable evaluation of the related subsoil and soil conditions, analyses of the settling and the interaction of the structure and subsoil is of great importance.

Some of the above highlighted issues need to be analysed in details.

3. FOUNDATION

Regarding the foundation of the digester structures the designer has to accomplish his task very thoroughly, because the site

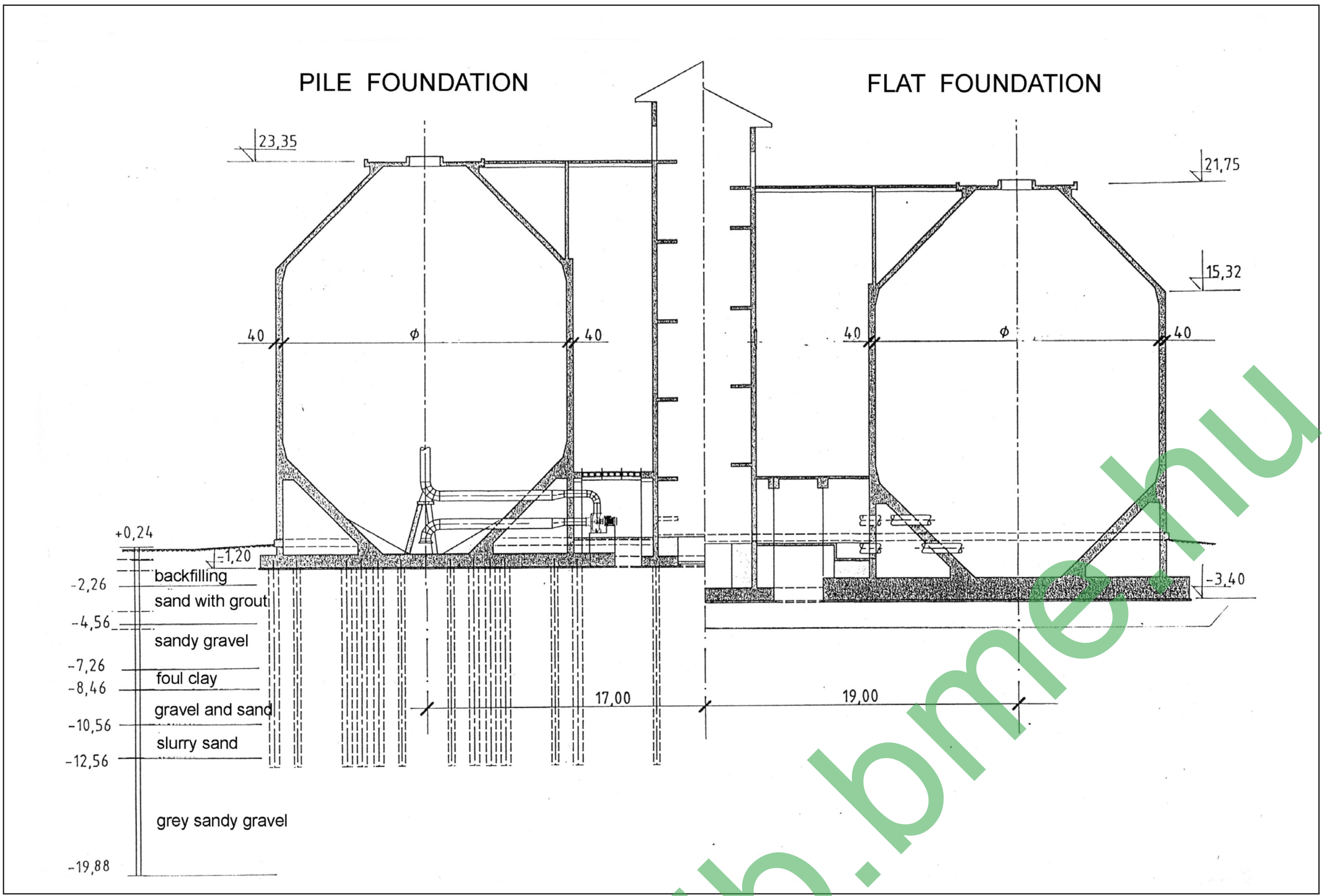


Fig. 1: Options of digester foundations

is generally deep area with high design water stage due to the nearly located treated wastewater recipient, where suitable soil rarely can be found near the surface, which meets the load bearing requirement. In accordance with the experiences in these areas water-soaked fine abrasive soft soils or non-compacted backfilling with mixed structure can be anticipated.

On the area of the site the exploration of subsoil with more than one drillings is inevitable. The spacing and depth of the drillings shall be chosen so that the soil layer structure, its horizontal alignment and thickness variations can be determined until the anticipated critical settling depth.

An incidentally eccentric included soft layer or significant layer thickness variation under the load bearing soil close to the surface perhaps more than one meter deep can conduce to unacceptable deformation of the digesters or the staircase.

Considering a determined superstructure the choice of the foundation level and method is determined basically the expected building groundwater level and the depth of the load bearing soil.

Regarding the foundation of digesters especially the twin arrangement the accumulation of the soil stresses originated from the load shall be taken into consideration. That affects the horizontal alignment of structures and the determination of the foundation method (*Fig. 1*).

As far as the foundations of the digester tanks have been built so far are considered our intention was the designing of flat foundation in some cases combined with soil change wherever it was possible. In some cases this aim could not be accomplished due to the different demands of the contractor. At these cases pile foundation was applied and the building was set up at a higher level. At the start of the construction due to the elevated groundwater level the pile foundation is more suitable to be started from higher level compared to the soil change combined with drainage, which may cause significant problems. Naturally the adversary of the above mentioned method also occurred when flat foundation was implemented with soil change instead of deep foundation. To vary the foundation method caused the modifying of the designs because the alteration reacted to the horizontal alignments and structural solutions. In order to avoid re-elaborations the clarification meetings with the building contractor are of great importance in the early period of construction. Unfortunately the contractor is not always known in the designing period consequently the re-designing can hardly be escaped. In case of known building contractor the foundation of digesters is suitable to be chosen from the professionally fair, soundly based technical solutions so that would meet the demands of the contractor, as well.

4. GEOMETRICAL CONFIGURATION OF DIGESTERS

The configuration, volume and geometric dimensions of digesters are determined by the hydraulic process requirements. A basic requirement of the digestion process is to avoid the formation of slack areas and move the sludge continuously during the digestion period. This requirement can be met by choosing the ideal form of the tank or the mixing technology adjusted to the given geometric form.

Considering the digestion process the egg shape can be deemed really ideal, which form would be favourable regarding as well as the aspect of statics. In a structure of that shape the sludge moving is ideal not depending on the mixing technology.

Digesters of egg shape definitely reasonable over 4000 m^3 useful capacity have not been built in Hungary so far. One reason of that is the fact that structures of that scale have not been constructed yet except the 2·4500 m^3 digester of Debre-

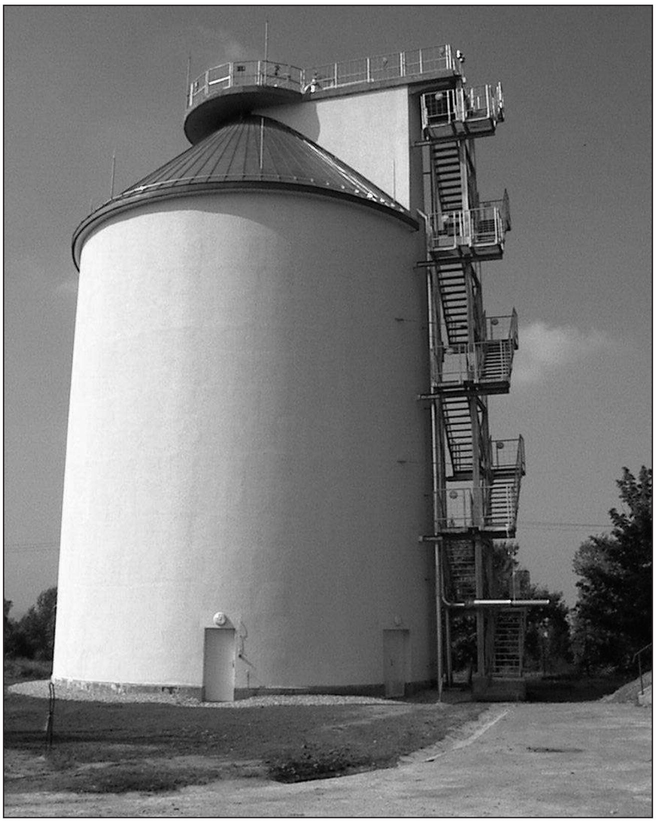


Fig. 2: 2000 m³ sludge digester in Dunakeszi

cen (Péter – Tóth, 1997). On the other hand the contractors' quotations of the doubly curved shell structures ideal from the aspect of statics could not be competitive due to the high prices caused by the more difficult formwork requirements.

Upon these experiences the digesters have been designed applied the configurations similar to the ideal forms (cylindrical shape closed by cones in the upper and lower parts). The digesters with the useful volumes of 1000-4500 m³ have been

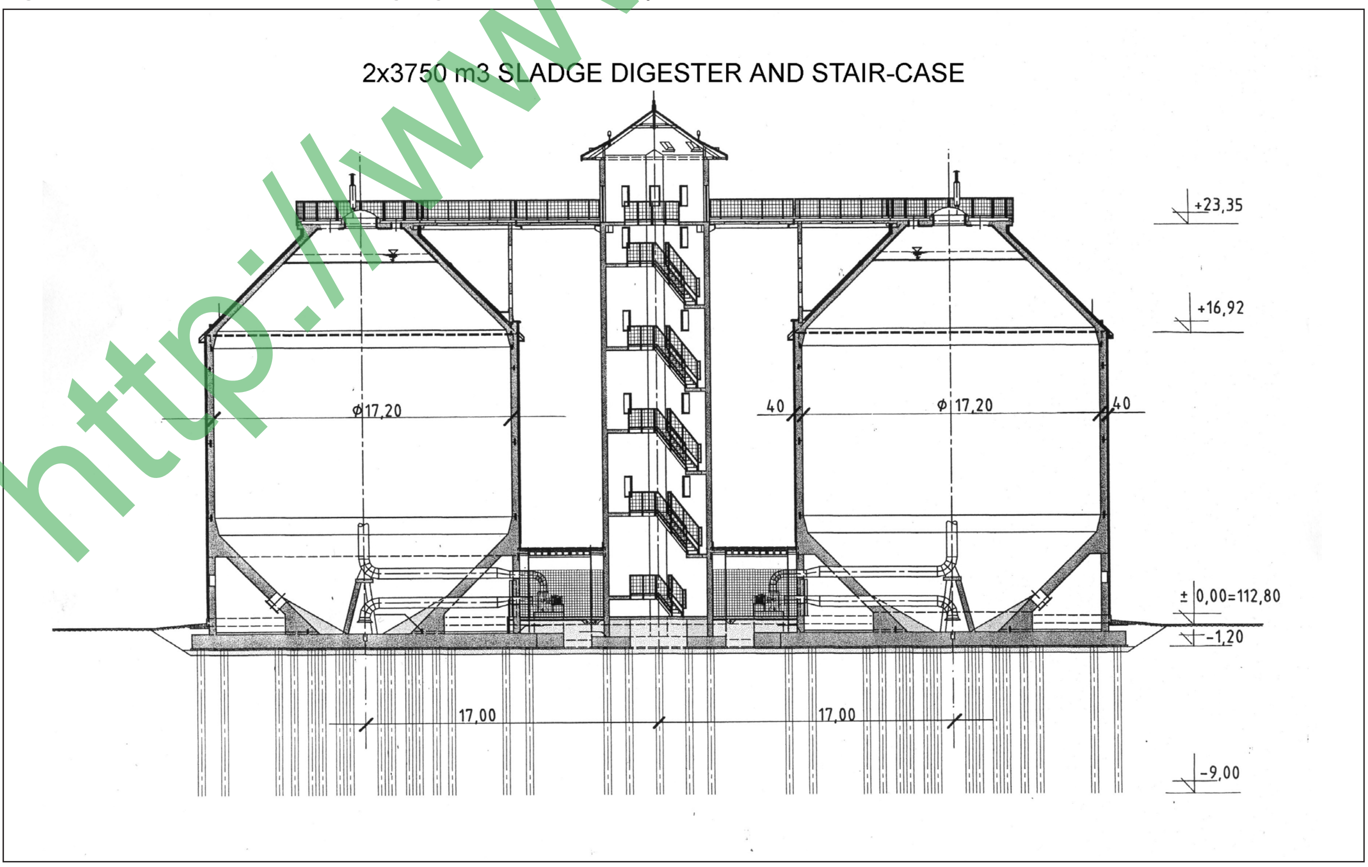
Fig. 3: 2000 m³ sludge digesters with common staircase in Nyíregyháza

Fig. 4: View of the implemented sludge digesters in Győr

implemented applied these forms in isolated (*Fig. 2*) and twin arrangement (*Figs. 3 and 4*).

The angles of the lower and upper cones were chosen the equal in order the multi-utilization of the formwork elements.

Fig. 5: Vertical section of the 2·3750 m³ sludge digester and stair case in Győr



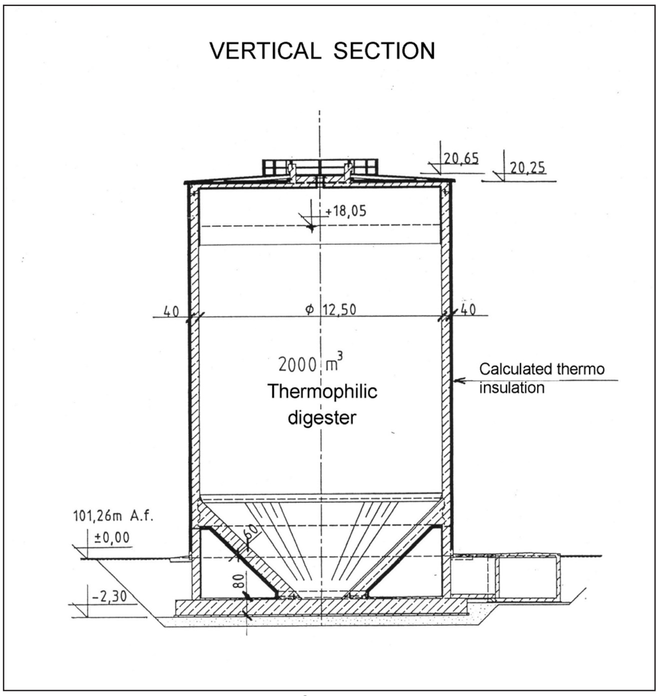


Fig. 6: Section A-A of the 2000 m^3 sludge digester in South-Pest

Regarding the implemented digesters the angle of the lower and upper cone was 45°. The applied geometric configuration can be seen on Figure *5*.

The structural design of the 2000 m^3 digester in the South-Pest Wastewater Treatment Plant was accomplished with a horizontal upper closing floor with top slabs in accordance with the requirements of the foreign process designer (Figs. *6; 7 and 8*).

The mixed round symmetric shell structure digesters designed by our company with the form of cone, cylinder, cone operating by the mixing technology of pump and propeller blade operate correctly in accordance with the consentaneous opinion of operators. The digesters answered one's expectations.

Regarding the digesters designed with different geometric features compared to the above-described structures mechanical engineering modifications were needed for the correct operation.

The geometric form similar to the ideal shape improves significantly the safety of the digestion technology and operation consequently it is reasonable to go on applying the experienced and approved method.

5. STRUCTURE, STRUCTURAL JOINTS

The digestion technology and the structural form of building are in interaction. After heating the sludge is fed into the digester, where the digestion of the sludge is going on with gas evolution under constant temperature and continuous mixing.

In the digestion period two important loads affect the structure namely the hydraulic and heat load. For the technology requires constant temperature the thermal protection of the structure shall be provided and the structure shall meet the requirement of this constant temperature under operating condition.

Due to the hydraulic load tensile and bending stresses originate in the complex shell structure in circumferential and generatrix direction.

The cylindrical walls and lower cones of the digester are

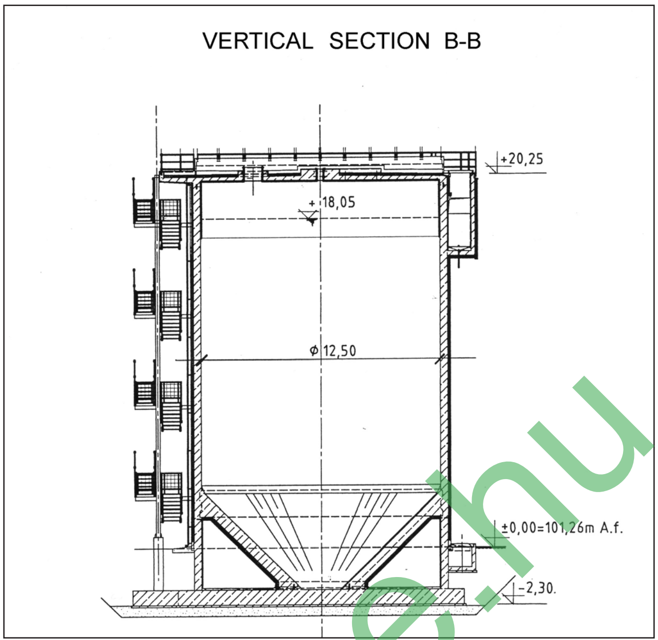


Fig. 7: Section B-B of the 2000 m^3 sludge digester in South -Pest

Fig. 8: View of the digester in operation in South -Pest

loaded by significant hoop stresses, while at the curvature variations (at the cone–cylindrical wall joint) bending moments originate in generatrix direction.

Above the useful volume of 2000-2500 m^3 the hoop stresses are reasonable to be accepted by stressing for the extraordinary dense reinforcement assuring the max. 0.1 mm crack-width.

In case of complete prestressing the scale of the prestressing force assures the compressed status in the structure in spite of the tensile stresses occurred from the hydraulic pressure.

In case of the digesters with the useful volume of 2000-2500 m^3 the stresses can be accepted by reinforcement, as well. The problem of the application of prestressing is determined upon economic and building technologic aspects.

However it has to be stated that the compressive force led to the structure by prestressing meets the waterproofing requirement at a higher level than the calculation for the crack-width of 0.1 mm.

In case of the digesters with the useful volume of 2500-4500 m^3 the posttensioning is enough to accomplish in annular direction on the cylindrical wall and lower cone because the scale of the tensile stresses occurred in the upper cone is significantly lower and the load accepting here can be solved by reinforcement without any difficulties.

At the digesters of these scales the stresses in generatrix direction can be accepted by reinforcement due to the beneficial haunches applied at the vertex points of curvature variations.

For the posttensioning of the structure we designed the tensioning staples with the so called “slip inserts”, which has been considered up-to-date recently.

The staples with the slip inserts can be applied before concreting within the structure and after construction out of the structure. Both solutions are applied in the Hungarian design practice.

In our opinion, in case of a new structure the tensioning staples applied for the posttensioning of the digesters are advantageous to build in to the structural concrete and their ends anchor after leading through the supports so called “aprons” (*Fig. 9*).

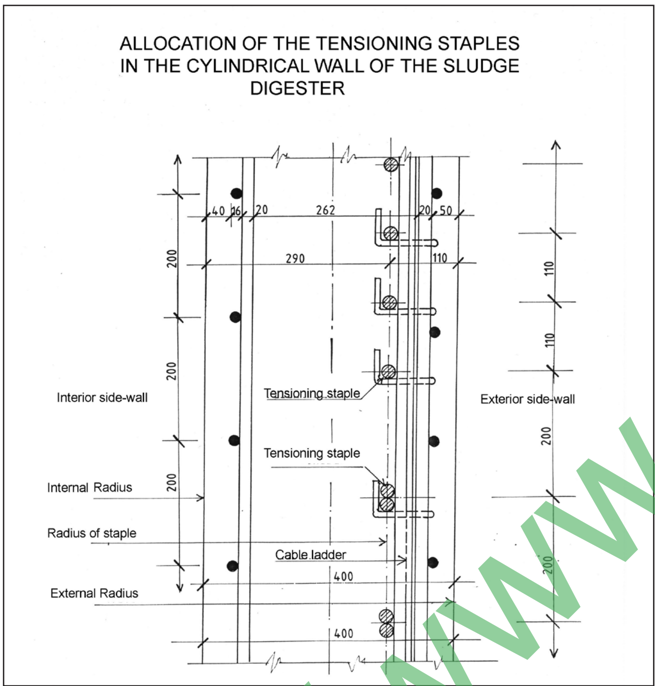


Fig. 9: Arrangement of the tensioning staples located in concrete cross-section

The above mentioned solution warranties the long-term protection of the staples and continuous surface force transfer with the bedding into concrete. The advantage of the configuration of the external staples is that the staples can be inbuilt after the casting of the structures. The construction cost is lower for aprons not needed. However the risk of damage and corrosion is higher and there is no continuous force transfer. The correction and rehabilitation costs coming from those can exceed significantly the sum saved at construction (*Fig. 10*).

The other significant impact loaded the structure is the heat effect derived from the digestion technology.

From the point of view of the digestion technology there are two distinctive methods the mesophilic and thermophilic digestions. At the mesophilic digestion the operational temperature is of 33-35 °C, but at the thermophilic that is of 55-57 °C.

Tensile forces and bending moments derive from the equable and non-equable thermal fluctuations due to the operational temperature of digestion. In order to mitigate the related stresses the structure shall be equipped by calculated thermo insulation on the external side of the structure. With


Fig. 10: Damage of the tensioning staple located out of concrete cross-section

adequate calculated thermo insulation the stresses occurred from the non-equable thermal fluctuation can be significantly diminished.

Due to the equable thermal fluctuation coming from the average temperature difference of the building and the operational conditions a significant stress increase occurs at the curvature variation locations of the structure, which is proportionable to the increase of the temperature difference.

Recently the mesophilic digestion method has been applied in Hungary. Corresponding to that the digesters are designed to accept the operational temperature of 33-35 °C.

However, upon the favourable foreign experiences the immediate appearance and spread of the thermophilic digestion method can be prognosticated in Hungary.

Consequently as far as the structures currently designed as mesophilic digesters concerned the stresses coming from the higher temperature effect would be reasonable to take into consideration at calculation and forming. That would result an increase of the building cost at about 5-7 %, but make possible the technology transformation without any new projects or new structures.

6. COHERENCES OF BUILDING TECHNOLOGY AND STRUCTURE

The structural joints of the digesters having been constructed so far had to be adjusted to the building technology applied by the contractor. This is the reason, why every designing task was different even at the same geometric configuration.

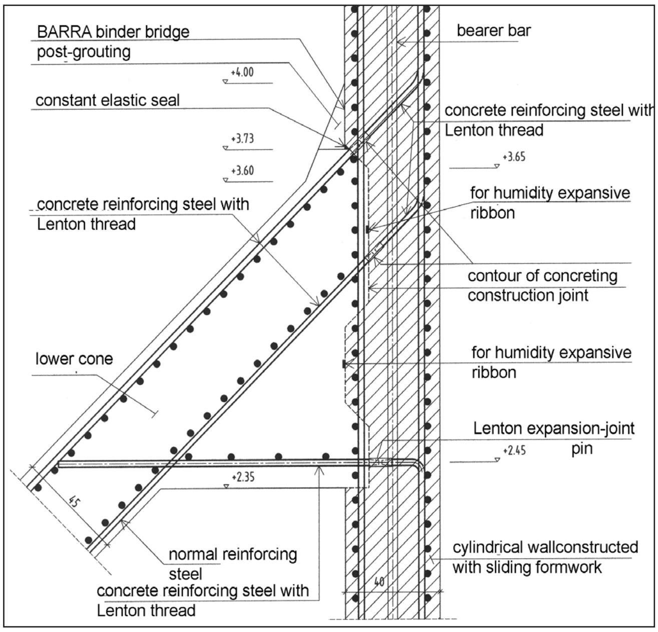


Fig. 11: Joint of cylindrical wall and lower cone in case of sliding shuttering building technology

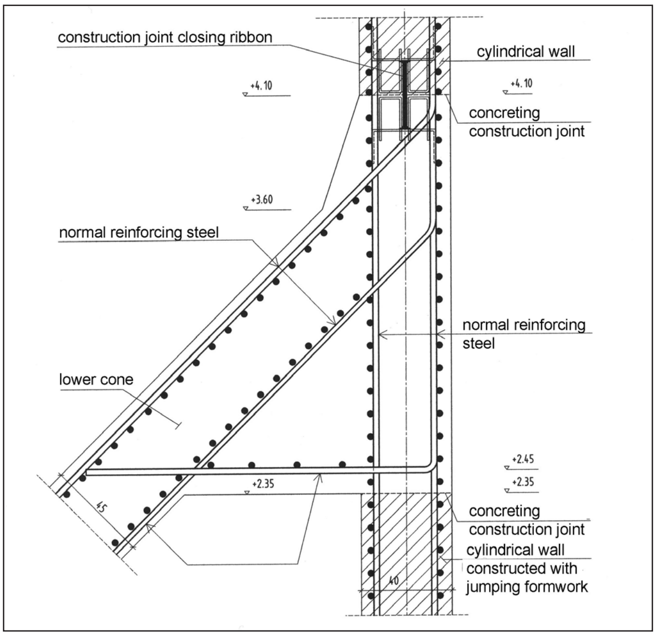


Fig. 12: Joint of cylindrical wall and lower cone in case of climbing formwork building technology

As far as the building technologic aspect is considered the key problem is the joint of the cylindrical wall and lower or upper cones.

Regarding the construction of the built digesters the sliding or climbing formwork were applied. The joints of the cylindrical wall and lower cone were different at the two solutions.

Fig. 11 indicates the joint of the cylindrical wall and lower cone in case of the climbing formwork building technology, while the same joint is shown in *Fig. 12* in case of the climbing formwork technology.

The different building technologies draw along the alternation of the alignment, arrangement and diameter of the reinforcement applied and the different configurations of the dilatations, construction joints and structural joints.

In *Fig. 13* the reinforcement of the joint of the cylindrical wall and lower cone at the 2250 m^3 digesters in Sopron can be seen, which are under construction. The structure has been designed for the application of climbing formwork technology.

The contractor is generally known in the designing period, but the actual building contractor has not been chosen. That is why the building technology applied by the designer is frequently not in correspondence with the building technology formed in accordance with the appliances of the winner building-contractor.

In these cases the redesigning of the structures would be needed but generally there is no time for that. Consequently at the start of the construction the building contractor realises the solution not in harmony with the technology of his company. That can be followed by the compromises, the series of design modifications, which are disadvantageous regarding the aspect of both the designer and the contractor.

It is reasonable to complete the designing after the structure related clarification meetings held after the building-contractor having been chosen. That assists the smoothness of the construction and the expected and required quality can be assured via the considered, clarified technical solution.

Fig. 13: Reinforcement of the digester under construction in Sopron

7. CONCLUSIONS

The implemented digesters met the load bearing and deformation requirements of their structures as well as the water and gas proofing at the suitable level in all of the cases. The structures confirmed that waterproof structures could be constructed with even sliding formwork building technology if the concrete mixing formula and the corresponding pour and post treatment technology is correct.

The displacements and settlings of the structures remained within the acceptable values due to the careful soil explorations and water filling instruction system in correspondence to the foundation methods.

The foregoing operational experiences of the digesters have been built so far are favourable. Constructed with taking into consideration the technologic demands the digesters meet the requirements regarding the structural aspects and operate correctly.

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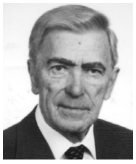
Gábor Zoltán Péter (1953): structural engineer with university degree. Started his designing career in 1975 at MÉLYÉPTERV. Main scope of his designs are that of steel structures, structural design, reinforced concrete structures of water supply and wastewater treatment and other special buildings and also development of form works and building technologies connected. Leading designer of Mélyépterv Komplex Co. Member of the Hungarian Group of ***fib***.

IMPROVING ACCURACY OF THE CONCRETE STRENGTH VERSUS WATER-CEMENT RATIO RELATIONSHIP




Assoc. Prof. János E. Ujhelyi – Prof. Sándor Popovics

Formulas used to estimate compressive strength of concrete are based on supposition, that the concrete strength is determined by the strength of the cement paste, while quantity of paste has no influence. Therefore the knowledge of water/cement ratio is enough to estimate concrete strength. However the analysis of test results demonstrates, that this supposition is not quite true. For instance, if two similar concretes have the same water/cement ratio, the strength of concrete with more cement content or with more fluidity is less. The difference is not large, but observable, in particular at great cement content. New functions having new complementary variables are presented for quantitative approach of this phenomenon. The new complementary variables may be water content, cement content, paste content, consistency, their powers, their combinations etc. The amplification is illustrated by Abrams-formula, but the method is suitable for any other formulas predicting concrete strength based on water/cement ratio. It improves not only the fitting of strengths, but also widens the validity limits. Correlation coefficients verify quantitatively, that the new formulas have more accuracy than the existing ones.

Keywords: concrete mixture, composition, water-cement ratio, compressive strength, density

1. INTRODUCTION

More than 100 years is known, that strength of similar concretes does not depend primarily on cement content, but on ratio of water content and cement content. (Expression "similar" means concretes having the same density, treatment, made by materials of identical quality and investigated with the same method). This rule is presented in *Fig.1* (Kaplan, 1960).

According to always reliable Graf (1960), Zielinski was probably the first, who investigated at the beginning of 1900 systematically the effect of the water-cement ratio on the compressive strength of mortars and published relationship on it (Zielinski-Szuk 1901. Zielinski 1909). In spite of this, the technical literature knows it as Abrams-law on the basis of study issued at 1918 (Abrams 1918). It seems that at the beginning of the past century the importance of water-cement ratio, the revolutionary discovery of Zielinski, was forgotten. For lack of better expression, this study denominates the relationship between concrete strength and water-cement ratio as *rule of water-cement ratio* instead of Abrams-law.

As the water-cement ratio is very important from the point of view of designing both compressive strength and composition of concrete, many tests were made and many formulas were expanded for long years to verify accuracy of this rule (*Figs. 2* and *3*) (Popovics 1998. Ujhelyi 1991). From these the most popular rule are: in USA the Abrams-formula, in France the Feret-formula, in some European countries the Bolomey-formula.

One advantage of formulas using water-cement ratio as one variable, that it is enough to know two reference-strengths (f_{c1} and f_{c2}) being attached to two water-cement ratios having significant difference (e.g. $x_1 = 0.35$ and $x_2 = 1.0$), because

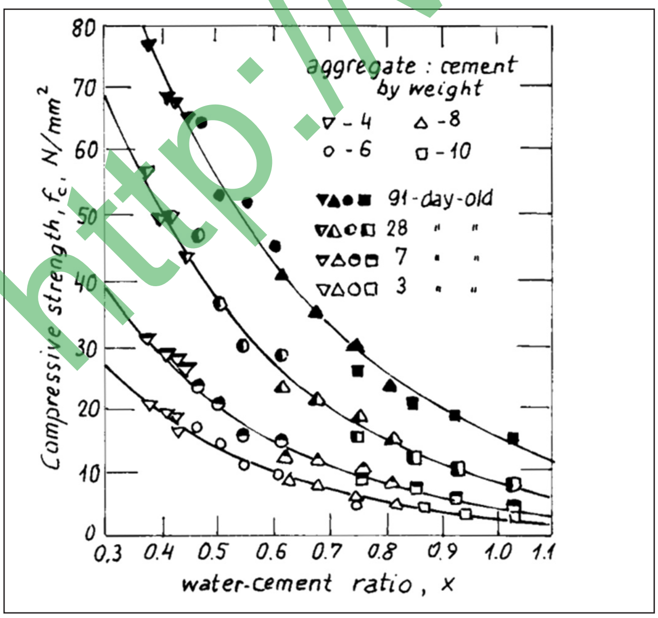


Fig. 1: Compressive strength of different aged concretes vs. water-cement ratio (1 ksi = 6.895 N/mm2) (Kaplan, 1960)

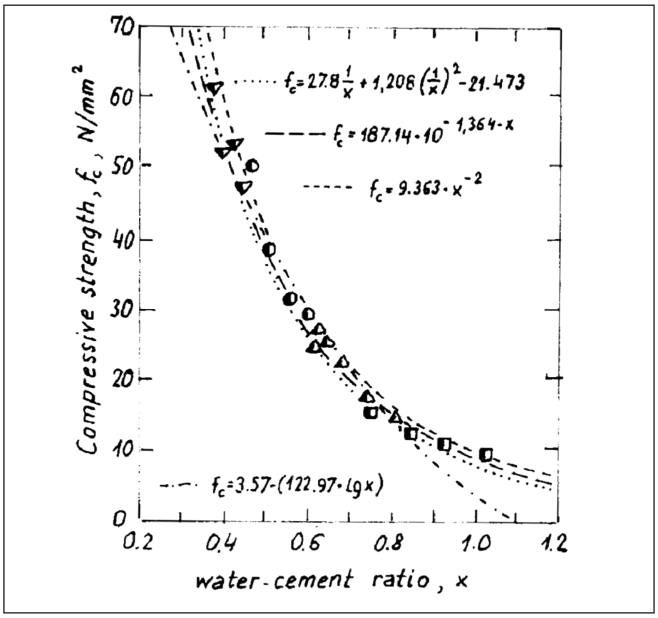


Fig. 2: Comparison of compressive strengths in 28 days estimated by equations based on water-cement ratio (data from Fig.1.)

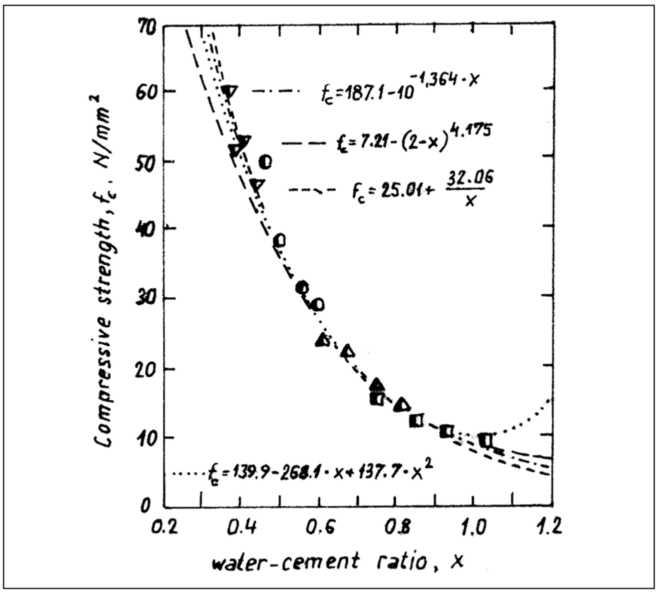


Fig. 3: Comparison of compressive strengths in 28 days estimated by equations based on water-cement ratio (data from Fig.1.)

the constants of the formulas can be calculated from these data. E.g. the constants *A* and *B* of the Abrams-function with following forms:

$$f_c = A \cdot e^{\ln B . x} \tag{1}$$

can be determined as follows, if the connected values: of $x_1 - f_{c1}$ and $x_2 - f_{c2}$ are known (Ujhelyi 1988):

$$A = \exp\left[\frac{(x_2 \cdot \ln f_{c1}) - (x_1 \cdot \ln f_{c2})}{x_2 - x_1}\right] \quad \text{és} \tag{2a}$$

$$\ln B = \frac{\ln\left(\frac{f_{c2}}{A}\right)}{x_2} = \frac{\ln\left(\frac{f_{c1}}{A}\right)}{x_1} \tag{2b}$$

Many experiments verified the suitability of rule of water-cement ratio, but some deviations are found. One of these is an observation, that the calculated compressive strengths fit to the test results only in the neighbourhood of small (0.3-0.4) and large (0.8-1.0) water-cement ratio, but at middle water-cement ratios the calculated strengths are smaller or larger than true ones. This deviation can be eliminated when the water-cement ratio is not used by its original value but by some exponent that is the Abrams-function I

$$f_c = A \cdot exp\ (lnB \cdot x^n) \tag{3}$$

where $0.2 \leq n \leq 5$ (Ujhelyi, 1989/1). Moreover noteworthy is that the relationships calculated by the Abrams-function according to (1), (3) and that by the Bolomey-function according to (4)

$$f_c = C \cdot \left(\frac{1}{x} - D\right) \tag{4}$$

are different as it can be seen on *Fig. 4*, where pairs of tested values are: $x_1 = 0.35$ and $f_{c1} = 70\ N/mm^2$, as well as $x_2 = 1.0$ and $f_{c2} = 15\ N/mm^2$. According to Hungarian tests strengths of the concretes made with cements of 1930-1950 fitted in

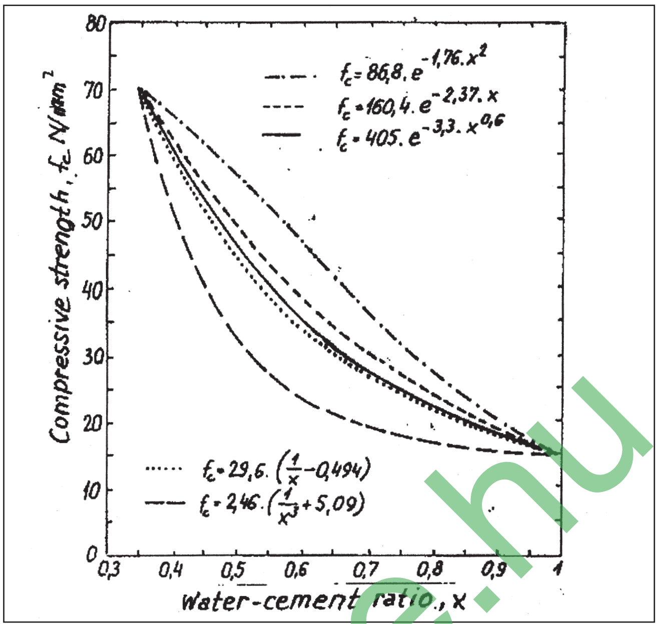


Fig. 4.: Compressive strengths calculated by Abrams and Bolomey equations on the basis of reference strengths at water-cement ratios x = 0.35 and x = 1.0 (Ujhelyi, 1989/2)

Bolomey-function (Palotás, 1952), while of late years fitted in Abrams-function (Ujhelyi, 1989/2).

Other experience is that the cement content and water content have secondary effect on concrete strength outside of water-cement ratio. These effects are apparent, thought relatively small, therefore they are known only qualitatively and not quantitatively. Object of this study is to analyse quantitatively some such effects.

This study does not discuss the effects of aggregate quality and of pore content on concrete strength.

2. THE ABRAMS-FUNCTION

Abrams recommended probably the first strength formula (Abrams, 1918):

$$f = \frac{A}{B^x} \tag{5}$$

or in other form:
$log\ f_c = log\ A - x \cdot log\ B$ (6)
or see (1) and (3) functions, where
f_c = concrete strength,
x = water-cement ratio,
A and *B* (or *log A* and *log B*, or *lnB*) = experimental constants obtained by fitting.

The experimental constants depend on quality of basic materials (first of all on cement type), on type of strength (e.g. compressive strength), on treatment, on method of test (e.g. on saturation of compressed surface, on loading rate etc.), on maturity of concrete, on validity of limits of rule of water-cement ratio, but independent of concrete composition. For instance the following equations can be fitted with value of $R^2 = 0.84$ to experimental results shown in *Fig. 5*:

$$f_c = \frac{99.5}{7.226^x} \quad \text{or} \quad log\ f_c = 2.16 - 0.8588 \cdot x \tag{7}$$

or with value $R^2 = 0.89$: $\quad f_c = 44.3 \cdot exp\ (-5.3 \cdot x^5)$ (8)

where f_c is given in N/mm² (1/6.895 ksi), x is given by weight and *R* correlation coefficient. The good fitting to 24 experimental results of these equations is illustrated in *Fig.7* not only by broken line determined by the equation (7), but the statistically significant minor value in level 0.01 is $r = 0.51$.

This experimental equation with one variable were followed by many other similar experimental equations (logarithmic, polynomial etc, see Popovics, 1998) with more or less similar approach for relation of $f_c - x$ (*Figs. 2-3*). As these equations were developed experimentally, neither has theoretical advantage. In some case, it is possible, that one of these equations give better approach, as the others. Therefore the suitable equation is select on practical basis, e.g. linear function for linear programming.

All these functions of one variable embody the fundamental idea in different forms, that the strength of a hardened concrete is controlled by the strength of its hardened cement paste. This

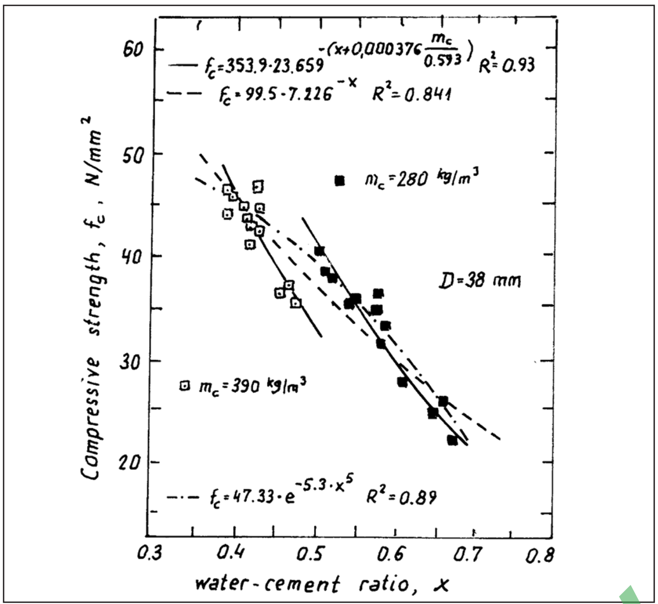


Fig. 5: Effect of cement content on compressive strength depending on water-cement ratio. The Abrams-equations are demonstrated by broken line (Popovics, 1990) and by dotted line (Ujhelyi, 1991). The equation extended by cement content is demonstrated by continuous line (1 ksi = 6.895

is reliable for the most structural concretes. The paste strength depends considerably on the porosity in fresh state, which is the *initial porosity* when the compaction of concrete is perfect so air content of the paste is negligible. Its volume is equal to the mixing water content i.e. the more the mixing water is compared to cement content (water-cement ratio !), the more the initial porosity and the less the paste strength. The exact relation between the initial porosity and water-cement ratio is as follows

$$p = \frac{x}{x + \frac{1}{\gamma_c}} \qquad (9)$$

where p = initial porosity, % 100,
x = water-cement ratio by weight,
γ_c = specific gravity of cement.

This equation is independent of water and cement content separately. If the water-cement ratios are identical in similar cement pastes, the initial porosities of pastes and their strengths are also identical independent of the paste quantities. If this train of thoughts is transferred to the concrete, the rule of water-cement ratio can be illustrated in elementary manner as follows:

a) Prepare cement paste sufficient to make more specimens. Take from it a portion and mix to it aggregate with 1:1 cement:aggregate ratio. Prepare specimens with specified air content (negligible small) from this mixture of given consistency by compaction fitted to the consistency and treat in specified way.
b) Take out new paste portion and mix to it aggregate with 1:3 cement:aggregate ratio. Then the mixture will be drier as in the case a). Prepare specimens by compaction fitted to this consistency with negligible small air content and treat in specified way.
c) Repeat this action with 1:4, 1:5 etc. ratios, by gradually more vigorous compaction until the paste content is enough to fulfil the hollows between aggregate grains and to remain the air content negligible small.

If these specimens are broken in the same age and method, there is logical to wait identical strengths despite of different cement content. The trials verify the rule of water-cement ratio in the rate at which this expectation is realised.

3. AMPLIFICATION OF ABRAMS FUNCTION

Accurate analysis of testing results shows, that the above train of thoughts however logical it is, true only in first approach. Fitting of single variable functions as (4), (5) to test data can be improved by a second independent variable. This can be the cement content (Popovics, 1990) or cement paste content (Ujhelyi, 1989). Experimental data support the addition of equation (5)

$$f_c = \frac{A}{B^{(x + C \cdot m_c)}} \qquad (10)$$

where
m_c = cement content, the others are identical with Eqs. (4)-(6).

The improved fitting of (10) to the test data can be seen in *Fig. 5* from experiments of Walker and Bloem (1961) with value of $R^2 = 0.93$. Augmentation of correlation coefficient from 0.84 to 0.93 shows the improving effect of cement content as parameter on the fitting.

It follows, that the strength of similar concretes with more cement content is less, even if the water-cement ratios are the same. The improvement of fitting is small, but definite. This phenomenon is surprising, because it is inconsistent with independence conception of the rule of water-cement ratio, as mentioned before and what is more: it can be expected with more cement content to increase the concrete strength. Scilicet with higher cement content there is less aggregate in the concrete, therefore less aggregate-paste interface, which is the weakest point in concrete.

The secondary effect of cement content on concrete strength is also explained, that porosity is only in cement paste, while that in aggregate is negligible. Strength of normal aggregate is greater, than that of cement paste. The latter made with customary process (compaction, treatment) has compressive strength maximum about 150 N/mm² (water-cement ratio: 0.21), the former has about 300 N/mm². So the more is the paste content, the more the porosity, the less the volume ratio of aggregate:cement paste, the less the concrete strength.

Porosity of hardened cement paste can be calculated. After full hydration the cement binds water about 23 % by weight, the rest evaporates. Specific volume of cement ≈ 0.32 cm³/g, that of hydrate product ≈ 0.398 cm³/g. It follows, that (Neville, 1981):

1.23 g hydrate product originates from (1 g cement + 0.23 g water)

equivalent volume of hydrate product: *1.23 · 0.398 = 0.49 cm³*

The total volume of cement and water participated in the reaction: *0.32+0.23 = 0.55 cm³*, so the volume of hydrate product is less with *0.55-0.49 = 0.06 cm³* (about with 11 %), as the original volume of cement and water participated in the reaction. This decrease of solid phases increases the porosity of cement paste, moreover produces inside chemical shrinkage, it is not visible in the total volume contraction, but possibly the structure of gel is cracking (peaks in stress develop). *Increasing paste content ⇒ increasing gel volume ⇒ increasing porosity ⇒ increasing cracking*; perhaps this can be the reason of cement content effect.

This explanation is disputable because the increasing capillary porosity decreases the strength of the paste and so that of the concrete, but only then, if the growth of porosity is the consequence of increasing water-cement ratio and not that of increasing paste content. It can be illustrated by elementary kind.

If the strength of a cement paste is tested with two specimens, a ∅100·200 and a ∅150·300 mm³ cylinders, the larger cylinder would provide smaller strength, the difference, however, is small, although the total volume of the capillary porosity in the larger cylinder more, than three times larger. So, the disproportionally higher volume of capillarity porosity in the larger specimen cannot explain the minor strength reducing effect of the higher cement content. This explanation is disputable also, because the specific porosity of the identical cement paste unchanged, therefore their strengths are also the same.

More ideas can be formulated to explain the effect of cement content, but it has to establish, that the exact mechanism of effect for strength reduction is not known. One probable mechanism is, that the more the cement paste is in the concrete, the greater the shrinkage and/or the bleeding, which weakens the sticking of the paste to the aggregate. One other probable mechanism is, that the more the paste, the higher is the hydration temperature, which diminishes the sticking and causes early cracking. It is necessary to carry out more experiments to explain the mechanism.

It has to be noted, that the above statements are not limited on Abrams-function. They are valid practically all functions of single variable for relation between the strength and the water-cement ratio. Moreover in the most cases instead of cement content can be used also the water content, the paste content, the aggregate content, the consistency, their powers or their combination with equivalent fitting, but more than two variables do not appear usable (Popovics, 1998). When the Abrams-function is expanded e.g. with the water content (see *Fig. 7*), the following relationship is valid:

$$\log f_c = 1.4 \cdot (2.126 - 0.792 \cdot x - 0.00185 \cdot v) \quad (ksi) \qquad (11)$$

where v = water content lb/yd³ (0,593 kg/m³), *1 ksi* = 6,895 N/mm² (the other symbols are as above).

In the *Fig. 6* are compared the above mentioned test results of Walker-Bloem (1961) with the values calculated from Abrams-functions (7), while in the *Fig. 7* the same results are elaborated by function (11). Comparing these figures it can be seen, that the function (11) gives better approach, than (7), but does not give better, than function (8).

Fig. 5 shows also, that the curves illustrating the completed equations are near to a line. It is resulted, that these relationships also can be approached among practical limits by linear

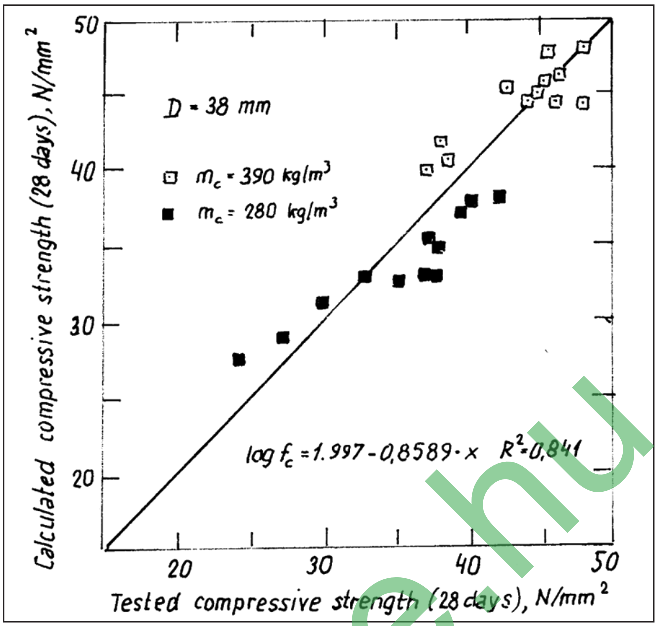


Fig. 6: Comparison of compressive strengths obtained by tests with strengths obtained by Abrams-equation [test data are from Walker and Bloem (1961)]. Symbols see in Fig.5.

equations. It is exemplified by equation (12), which compares again experimental results of Walker and Bloem (1961) with values calculated by the following linear equation completed with paste content:

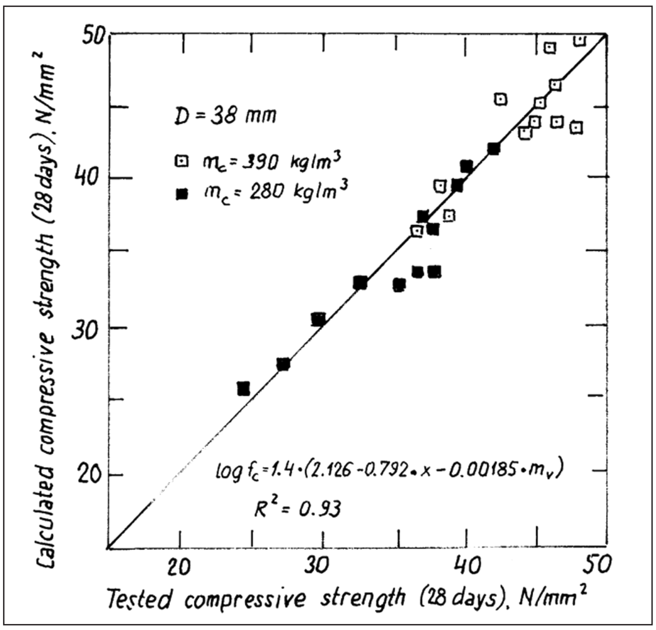
$$f_c = 13.851 - 12.51 \cdot x - 2.26 \cdot (v + c) \qquad (12)$$

where f_c is given in *ksi*. The correlation coefficient is $R^2 = 0.95$.

4. OTHER EFFECT OF CEMENT PASTE CONTENT

The change in strength calculated by equations of single variable does not depend, that the water-cement ratio is modified by cement content or by water content. According to such a completed equation, as (10), increasing of water-

Fig. 7: Comparison of compressive strengths obtained by tests with corresponding values calculated by equation including water content [test data are from Walker and Bloem (1961)]



cement ratio diminishes the concrete strength more rapidly by increasing of water content as by diminishing of cement content. E.g. in *Fig. 5* the unbroken curves are steeper, as the broken ones, because in these are increased the water-cement ratio only by increasing water content, so the paste content had been also greater. On the other hand, if the greater water-cement ratio is resulted by diminishing cement content, then paste content also diminishes; therefore degree of strength loss is less. It follows again that the concrete strength can be more effectively increased, if the water-cement ratio is decreased by decreasing water content, than by increasing cement content (c.p. with *Fig. 11*). We must not by all means forget, that compacting method should be fit to the consistency, so less water content requires more effective compaction.

The amplification of Abrams-function with other variables results in similar correction, as amplification with cement content or water content. The amplification with slump gave the following result:

$$f_c = \frac{22.86}{24.49^{(x+0.00618.s)}} \tag{13}$$

where f_c = compressive strength in *ksi*, s = standard slump in *inch* (25,4 mm). *Fig. 8* shows the fitting of equation (13) to test results of Gruenwald (1985).

The effect of concrete consistency on the rule of water-cement is shown in *Fig. 9* for two test series. In one series Vebe-time was 7 s. in the other was 170 s. The results are taken out from the dissertation of Kamenski (1985). The difference of the results of two series is definite, but it does not nullify the suitability of rule of water-cement ratio.

The change of compressive strength depending not only on water-cement ratio, but also on concrete composition is made recognizable by the method of investigations on the one hand and by the method of drawing of figures on the other. By investigation method is understood, that which two parameters are selected for constants from the variables of cement content, water content, consistency and aggregate quality and which two other parameters are changed to modify the water-cement ratio. The changes should be between such limits, which permit to make concrete of specified density. From the possibilities are as follow:

- cement content (e.g. 300 kg/m^3) and aggregate quality (e.g. limited sand content) are constant, water content and consistency change. The change in water-cement ratio comes about to 0.4-0.8;
- cement content (e.g. 300 kg/m^3) and consistency (e.g. slump is 20 mm) are constant, aggregate quality and water content (fitting to the aggregate) change. The change in water-cement ratio comes about to 0.3-0.9;
- water content (e.g. 150 kg/m^3) and consistency (e.g. slump is 50 mm) are constant, aggregate quality and cement content change. The change in water-cement ratio comes about to 0.5-1.5;
- water content (e.g. 150 kg/m^3) and aggregate quality are constant, consistency and cement content change. The change in water-cement ratio comes about to 0,4-1,5;
- aggregate quality and consistency are constant, water content and cement content change. The change in water-cement ratio comes about to 0.25-1.8.

The results of investigations showed, that the correlation between water-cement ratio and compressive strength is not independent from systematic alteration method of water-

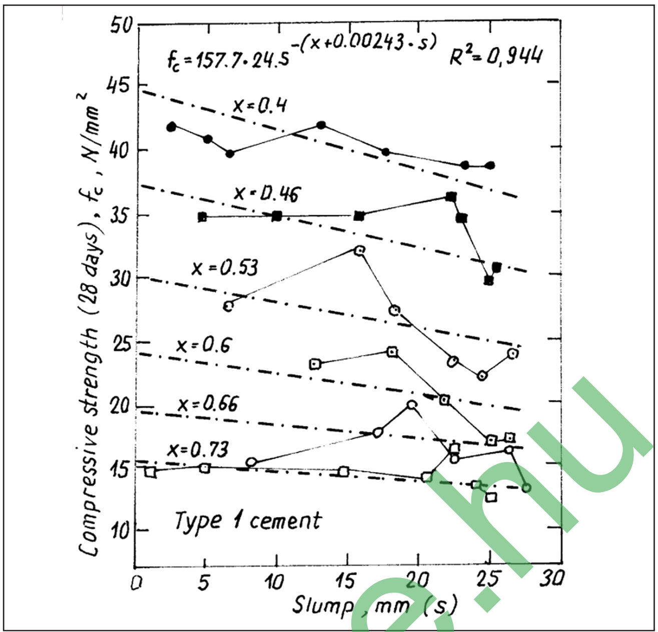


Fig. 8: Effect of slump on relationship between compressive strength and water-cement ratio. Test results of Kamenski (1985) are demonstrated by continuous lines. The broken lines present the values calculated by equation extended with slump.

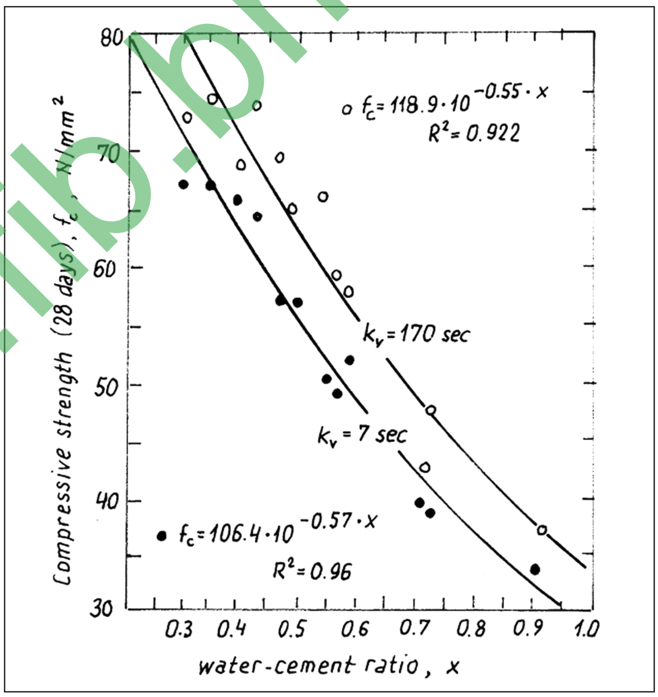


Fig. 9: Compressive strength of moister (softer) concrete is smaller at the same water-cement ratio, than the drier one. The points present the test results, the lines present the calculated values (1 Mpa = 145 psi) (Kamenski , 1985)

cement ratio, as it can be seen in *Fig 10* (Ujhelyi, 1991) for concretes made with constant aggregate quality (limited sand content) and the other constant is either cement content, or water content, or consistency.

Consequences of alteration in drawing the figures, in representation of relationship, can be seen in *Figs 11* and *12* (Ujhelyi, 1989).

Test results of compressive strengths in 28 days of concretes made with three types of aggregates (modulus of fineness are 6.55; 5.596 and 4.922) and with four consistencies (Vebe-times are 25-42 s, 10-17 s, 2-4 s 1 and 0-1 s) can be seen in *Fig.11* Equation of the curve calculated from all results is as follows:

$$f_c = 2873 \cdot exp\,(-5.766 \cdot x^{0.4}) \tag{14}$$

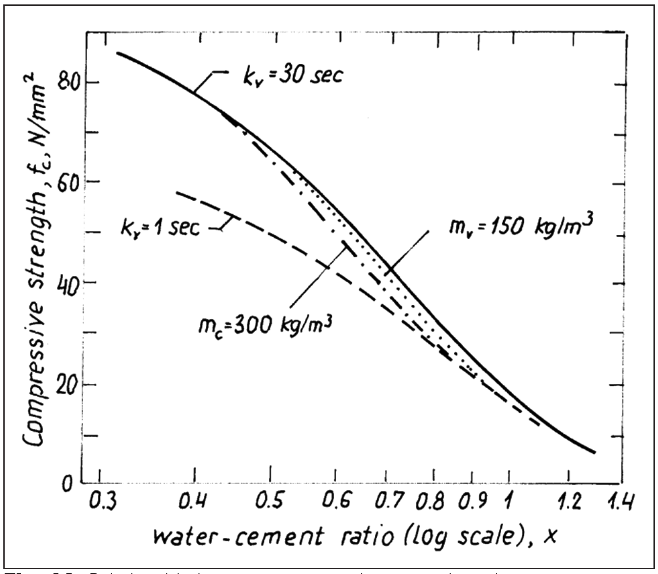


Fig. 10: Relationship between compressive strength and water-cement ratio depending from method of systematic alteration in water-cement ratio (Ujhelyi, 1991)

This equation has a correlation coefficient of R^2 = *0.89*. In *Fig.11* results of concretes of Vebe-time ≈ 35 s are drawn with dotted line, which has an equation according to the Abrams-function as follows:

$$f_c = 508{,}7 \cdot exp\,(-\,3{,}834 \times x^{0{,}4}) \qquad (15)$$

This equation has a correlation coefficient of R^2= *0,945*. In *Fig.11*. results of concretes of Vebe-time ≈ 1 s are drawn with dotted line, which has an equation according to the Bolomey-function as follows:

$$f_c = 13.32 \cdot \left(\frac{1}{x^{1.5}} - 0.399\right) \qquad (16)$$

This equation has a correlation coefficient of R^2 = *0.951*. According to the curves shown in *Fig.11* there is no considerable deviation between calculated and tested values, but there is a definite difference between data of dry and fluid concretes. E.g. the following compressive strengths can be calculated for the concretes of water-cement ratio 0.5:

- ➢ for every test results, by Eq. (14): f_c = *36.4 N/mm²*;
- ➢ for test results of dry concretes, by Eq. (15):*40.5 N/mm²*;
- ➢ for test results of fluid concretes, by Eq. (16):*32.4 N/mm²*.

These differences can not be read off the *Fig. 11*.

Change for given water-cement ratio in compressive strength depending on consistency and on cement content can be better appreciated in *Fig. 12*, where test results of concretes made with aggregate of fineness modulus *m = 6,55* are processed from *Fig. 11*. Probably this figure, together with *Fig. 8*., gives firstly possibility to the following conclusions.

Cement content of a concrete of given water-cement ratio made with aggregate of given quality determines the consistency. E.g. ordinary concrete mixtures of water-cement ratio *x = 0.4* made with aggregate of fineness modulus *m = 6.55* claim cement contents for consistencies dry, semi plastic, plastic and fluid consecutively *330, 400, 480* and *560 kg/m³* (Ujhelyi, 1988). It follows, that the necessary water content consecutively *130, 160, 190* and *220 kg/m³* (40 % of cement content). The dry mixture must be compacted very vigorously to remove air carried into, to reach the full density: the specimens made in laboratory should be vibrated during long time (60-180 s) under load. If the cement and water content diminishes further, the concrete will be such dry, that the full density, freedom from air, can be reached only by pressing. As a result of this, the adhesion between cement paste and aggregate surface increases, dislocation in aggregate-paste interfaces lessens, concrete compressive strength increases.

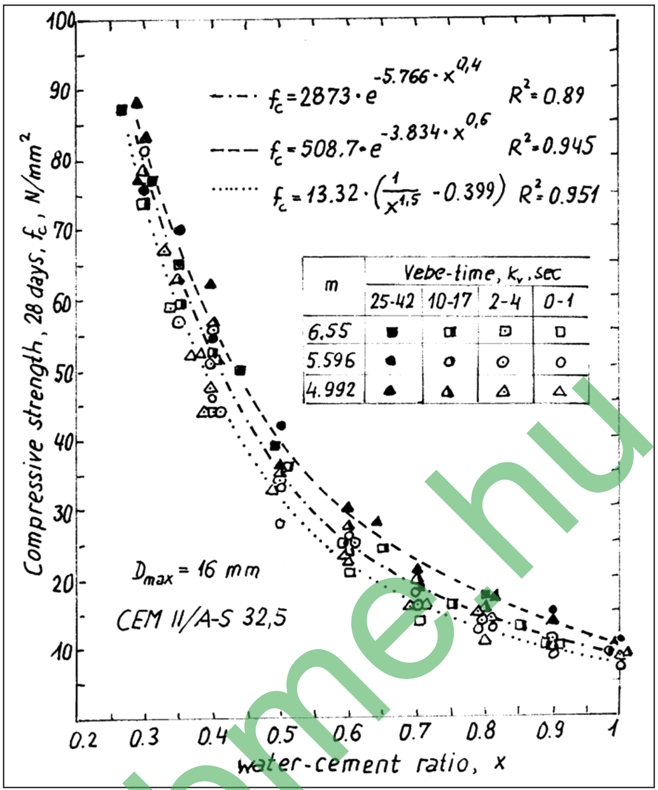


Fig. 11: Realationship between compressive strength and water-cement ratio for concretes made with aggregates of different grain-size distribution and with different consistency (Ujhelyi, 1989/2)

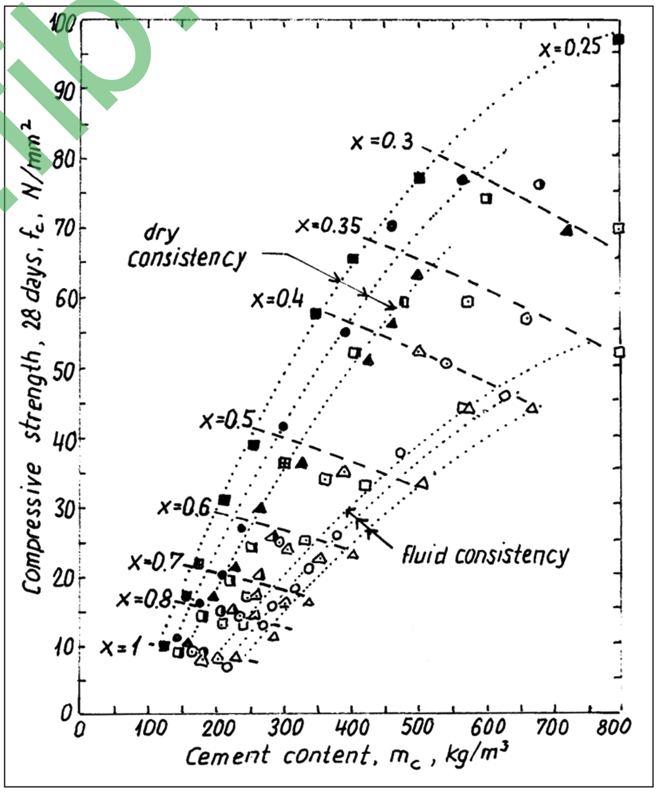


Fig. 12: Relationship among cement content, consistency, water-cement ratio and compressive strength of concretes of 28 days made with aggregate of fineness modulus m = 6.55 [from data of Fig. 11. (Ujhelyi, 1989/2)]

If the consistency is softer than the dry one, degree of compaction must be decreased, otherwise the concrete disintegrates, bleeding happens. In the case of fluid concrete mixture is enough to pour carefully into the forms and to level (self-compacting concrete). It may be supposed rightly, that the quality of aggregate-paste interface is worst in this case, in spite of the fact, that strength of cement paste is unaltered, as the water-cement ratio is also unaltered.

Thought the initial porosity of the cement paste according to (9) for $x = 0.4$ comes to ~0.55, but the initial porosity of well compacted concrete increases together with softening consistency and with increasing cement paste content. In the given example ($x = 0.4$) cement paste contents at specific gravity of $\gamma_c = 3.1\ g/cm^3$ are consecutively *236, 289, 345* and *400 liter/m³*, therefore the compacted concretes have initial porosities equal to water contents.

Test results shown in *Figs 11* and *12* call attention to control the concrete compaction under maintaining validity of rule of water-cement ratio, since by this means concrete compressive strength can be estimated more accurately and concrete composition can be more exactly designed.

5. CONCLUSION

There is known more than 100 years, that the compressive strength f_c of concretes depends considerably on water-cement ratio (*water/cement* = x). Basis of this fact is, that strength of structural concrete is controlled essentially by the strength of its hardened cement paste, which is a valid statement for most of structural concretes. Several experimental equation were developed for relationship between water-cement ratio and compressive strength, among these the Abrams-formula is probably the most common.

In this study is demonstrated, that introduction of a second variable, e.g. the cement content (and the consistency resultant from cement content) improves the fitting of equation to the tests results and increases the limits of its validity. Therefore new equations can be developed for relationship between cement composition and compressive strength. It is also demonstrated quantitatively, that in the case of equal water-cement ratio concrete with more cement content has less compressive strength. Moreover change of concrete strength depends not only on change of water-cement ratio in greatness, but does also on that water-cement ratio is altered by variation of cement content or of water content. If the water content is constant and the water-cement ratio is diminished by augmentation of cement content, the increasing of compressive strength is less, as if at constant cement content the water-cement ratio is diminished by decreasing of water content. This latter is the more economical method.

Together with decreasing water it is necessary to increase the effectiveness of compaction, because the mixture becomes drier. The conclusion can be drawn from this, that the basic relationship of water-cement ratio and concrete compressive strength characterizes firstly the chemical properties, primarily bond of cement paste, while the secondary features (e.g. cement content) are influenced by physical conditions of making concrete, namely effectiveness of compaction.

6. NOTATIONS

x	water-cement ratio
m_c	cement content
γ_c	specific gravity of cement
m_v	water content
m	fineness modulus of aggregate
D_{max}	maximum grain size of aggregate
S	slump
k_v	Vebe-time
p	initial porosity, % 100
f_c	concrete compressive strength
A, B, C, D	experimental constans
R	correlation coefficient

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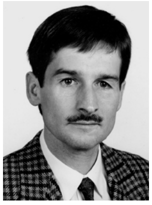
ANALYSIS OF STATIC EQUILIBRIUM OF CIVIL ENGINEERING STRUCTURES

Prof. György Farkas – Dr. Zsolt Huszár – Assoc. Prof. Antal Lovas – Prof. Kálmán Szalai

The loss of the static equilibrium or with other words the loss of the overall stability is one of the possible cases of the collapse of the civil engineering works. The paper summarises the methods for analysis against buoying, gliding and overturning according to Hungarian Standard and solutions were proposed in accordance with EC prescriptions.

Keywords: buoying stability, gliding, overturning, risk analysis, partial safety factors

1. INTRODUCTION

Due to the loss of the static equilibrium engineering structures may become unserviceable. This may happen by a change of the position, in which the strength of the material will not be exhausted (*Fig. 1*).

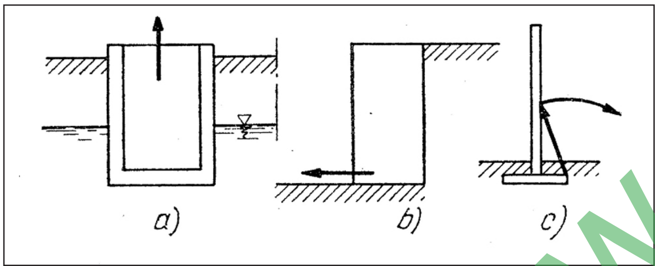


Fig. 1: Cases of loss of overall stability
a) buoying b) gliding c) overturning

The following three main types of such changes are usually distinguished in static equilibrium such as: buoying (*Fig. 1.a*), gliding (*Fig. 1.b*) or overturning (*Fig. 1.c*).

2. ANALYSIS OF THE STATIC EQUILIBRIUM ACCORDING TO THE HUNGARIAN STANDARD

2.1 Analysis according to the Hungarian Standard MSZ 15021 and MSZ 15226-54 R

According to the Hungarian Standard (MSZ 15021 "Load carrying structures of buildings. General Prescriptions" and the Standard MSZ 15226-54 R "Structural design of hydraulic engineering works") the engineering structure should be stable, i.e. safe against overturning, gliding or sinking, resp. buoying. For this purpose it must be proved that:

$$1.1\ \Sigma Y_{fa} + \Sigma k_e Y_{fe} + \Sigma Y_{fj} \leq 0.9 \Sigma Y_{sa} \qquad (1)$$

and

$$1.25\ \Sigma Y_{fa} \leq 0.9 \Sigma Y_{sa}$$

where

Y_{fa}, Y_{fe} and Y_{fj} – overturning moments, resp. slipping forces originating from constant and accidental loads, resp. effects as well as accessory effects originating from possible least advantageous combination of the constant and variable loads from the point of view of overturning, fall, gliding, resp. buoying.

Y_{sa} – permanent moments, resp. forces (without accidental loads and accessory effects) blocking fall, overturning, resp. buoying.

k_e - safety factor of the accidental load, the value of which is usually 1.1, but if the effect (e.g. water pressure) reduces the ultimate effect of the action, than this value is 0.9. If the load is the only possibly type of load than k_e=1.4.

For example in the case according to *Fig.1.a* stability should be checked against the buoyant force. The ultimate value of the buoyant force is on the basis of Eq. (1): $1.4\Sigma Y_{fa}$ and the force assuring resistance is than $0.9\Sigma Y_{sa}$. The bearing force is appropriate, if the

$$0.9\sum Y_{sa} \geq 1.4\sum Y_{fa}$$

condition is fulfilled, the formula can be expressed with the expected values in the following way $R_m = \Sigma Y_{sa}$ and $E_m = \Sigma Y_{fa}$, the formula is as follows:

$$\gamma_{RE} = \frac{R_m}{E_m} = \frac{\sum Y_{sa}^{(-)}}{\sum Y_{fa}^{(+)}} \geq \frac{1{,}4}{0{,}9} = 1.56$$

2.2 Analysis according to Hungarian Standard MSZ 15021/1-71

The condition of static equilibrium according to the Hungarian Standard MSZ 15021/1-71 is:

$$\frac{Q^{(-)}}{Q^{(+)}} \geq k_s$$

where

$Q^{(-)}$ and $Q^{(+)}$ are extreme combination of favourable, resp. unfavourable, effects of action to be calculated from these, resp. from the point of view of the static equilibrium, which should be defined taking into account second order deformations and k_s=1.0.

If loads, resp. resistances influencing static equilibrium arise mainly from loads, resp. resistances of the subsoil and their changes, resp. extreme values are not reliably known, the checking can be carried out with a combination calculated from characteristic values of loads, resp. characteristic values of the soil parameters. In this case second order deformations can be neglected and the values of k_s are shown in Table 1.

Note: k_s essentially has the same meaning as γ_{RE} used in other cases.

Table 1: The k_s safety factors (according to Hungarian Standard MSZ 15021/1-1971)

Type of loss of static equilibrium	usually	in a subordinated case*
buoying	1.4	1.2
gliding	1.6	1.3
overturning	2.2	1.6
Note: * the loss of static equilibrium does not cause any life danger		

By the analysis of the overturning of the construction works, the height of the gravity centre of which lies above the foundation level higher than the triple of its foundation width, more precise calculation should be preferably applied.

2.3 Procedure according to Hungarian Standard MSZ 15021/1-86

According to Hungarian Standard MSZ 15021/1-86 the safety factor of the concrete, reinforced concrete, metal and timber structures is

- usually: 1.1 (which was changed to 1.2 when modification was carried out in the year 2000),
- by the analysis of static equilibrium, if it is more unfavourable from the point of view of stability than 0.8.

The Eq. (1) according to 2.1 is by the application of the modification of 2000 (1.2 instead of 1.1):

$$1.2\,\Sigma Y_{fa} + \Sigma k_e Y_{fe} + \Sigma Y_{fj} \leq 0.8\Sigma Y_{sa}$$

and it can be written in form of:

$$1.25\,\Sigma Y_{fa} \leq 0.8\Sigma Y_{sa}$$

With respect to static equilibrium the value of Y_{fa} should be regarded authoritative (note: the value of k_s can not be namely evidently defined) so the quotient of values of resistance R_m, resp. effect E_m is as follows:

$$\gamma_{RE} = \frac{R_m}{E_m} = \frac{\sum Y_{sa}^{(-)}}{\sum Y_{fa}^{(+)}} \geq \frac{1{,}4}{0{,}9} = 1.56$$

3. PROCEDURE ACCORDING TO THE MANUAL OF HYDRAULIC ESTABLISHMENTS

The procedure presented in this chapter is based upon the handbook "Manual of hydraulic establishments" by György (1974).

3.1 The notion of foundation

The foundation is the structural element of the construction that transmits the loads to the load carrying subsoil in such a manner that

- the use of the construction work according to its function is not limited during the expected lifetime by the bearing power of the subsoil and its deformation caused by loading,
- static equilibrium of the construction works is assured and
- establishment outside the constructions would not more seriously damaged during its building and use than allowed.

The foundations can be divided into two big groups according to their build-up and the manner of their construction:

Flat foundations, transmit loads to the subsoil through flat surfaces contacting directly the basic face. In this case bearing layers positioned in a higher level of the subsoil are used.

In case of d*eep foundations*, the lateral surfaces of the foundation also take part effectively by transmitting the loads. This solution is mainly used by a foundation where the load bearing layers are in lower level.

For completion of both flat and deep foundation, if soil excavation was necessary, either a dried building pit should be built or special underwater procedures should be applied. From the point of view of the reconsideration if the foundation can be considered either as a flat or a deep foundation, the height of water-level is not determinant.

The design of the foundation takes place after the analysis of the lamination of the foundation soil, the water-level (opened or ground water), the manner, depth of the foundation and other conditions in the following steps:

A) definition of ultimate loads,

B) definition of the action effect of the subsoil,

C) analysis of static equilibrium of the foundation as well as the contacting structure (*Fig. 1*),

D) analysis of the rotation or gliding of the whole construction along a cylindrical or a flat sliding base (analysis of the gliding of the subsoil)

E) definition of sinking and deformations of the foundation structure.

3.2 Analysis of the static equilibrium of the foundation

The analyses relating to the "static equilibrium" in point C) (Chapter 3.1) – i.e. buoying, overturning, gliding – in accordance with the paper should be carried out in the following manner.

According to the Hungarian Standard should be proved with calculations that relevantly to points B) and C) (of Chapter 3.1) the conditions are fulfilled:

$$\sum n_a Y_a + \sum n_e Y_e + \sum Y_j \leq \sum \alpha_s Y_s \qquad (2)$$

and

$$1.25\,\Sigma Y_a \leq \alpha_s\,\Sigma Y_s \qquad (3)$$

are also fulfilled in deeper layers from the point of view of the bearing capacity and gliding on the level of the foundation, where

Y_a, Y_e and Y_j are stresses, forces, resp. moments calculated from permanent and variable loads, as well as accessory effects in the most unfavourable combination from the point of view of the base failure, gliding, resp. overturning or buoying,

Y_s is the tension, force, resp. moment calculated from permanent and variable loads as well as accessory effects on the basis of the most unfavourable combination, blocking base failure, gliding, swing, resp. overturning or buoying and effecting simultaneously with previous effects,

n_a and n_e – are dispersion and safety factors prescribed by relevant standards,

α_s – special safety factor, which should be taken into account either from the point of view of action effect of the soil depending on foundation methods, or taking in view *Table 2*.

While from the point of view of the effect of action on the subsoil according to B) the stability against base failure can be defined by means of Eq. (2), the use of this formula often leads to a false result in the analysis of the static equilibrium. For this it is enough to check visually the cases indicated in *Figs. 4 and 6.b*, in which the values of safety against buoying and overturning are much lower than values calculated on the basis of the above mentioned standard. The value of γ_{RE} safety factor, like the quotient of expected values of R_m resistance, resp. E_m effect can be calculated on the basis of Eq. (3) taking into account the data of *Table 2*.

The γ_{RE} safety factor is in the case of gliding or overturning:

$$\gamma_{RE} = \frac{R_m}{E_m} = \frac{\Sigma Y_s}{\Sigma Y_a} = \frac{1{,}25}{0{,}7} = 1.786$$

and in the case of buoying:

$$\gamma_{RE} = \frac{R_m}{E_m} = \frac{\Sigma Y_s}{\Sigma Y_a} = \frac{1{,}25}{0{,}9} = 1.39$$

in the above manner.

3.3 Remarks concerning effects of actions during the foundation

A) Case of buoying

On the left side of Eq. (2) the water buoyancy marked with 6 in *Fig. 2* is indicated, while on its right side the minimum values of the loads marked with 1, 2, 3, 4 and 5 should be taken into account as stabilising weight combined with appropriate α factors.

The base surface F taken into account in the Y_e water buoyancy in Eq. (2) is usually

$$F = \varepsilon\, F_0$$

where F_0 is the area of the foundation.

If the subsoil is from granular material without cohesion or having a virtual cohesion then $\varepsilon = 1.0$. If the soil has a very strong cohesion and that cohesion also remains later, then $\varepsilon \leq 1.0$, e.g. the normally compact and not cracked rock. Eg. in the Austrian central power station over the Danube $\varepsilon = 0.85$.

Aside from rocks the calculation is to be carried out with the condition $F = F_0$ according to Eq. (2) acknowledged in the practice. This is shown in *Fig. 4*. The explanation of $\varepsilon < 1.0$ is shown in *Fig.5.b*.

In addition to the calculations according to the standard the increase of water-level according to *Fig. 4.b*:

$$\Delta h = h_2 - h_1$$

should also be checked the cases 5.b, where buoying takes place (extreme position) and must be analysed depending on the local conditions. In this analysis forces should be taken into account without safety factor and with their expected values.

B) Case of overturning

By the analysis of overturning on one hand in the calculation by means of formula (2) the values of forces increased with factors may lead to false results besides the uncertainties mentioned

Table 2: Safety factors (prescriptions of hydraulic engineering MSZ EN 1990:2004 Eurocode)

		α_s rounded-off decimal value	
in $\alpha_s\,\Sigma Y$ / if Y_s		If the shear strength of the soil is analysed in laboratory	If the shear strength of the soil is taken from the table
Frictional resistance or active earth pressure		0.7	0.5
Strut pressure		1.0	1.0
Earth pressure at rest		0.7	0.6
Passive earth pressure		0.5	0.3
Water-pressure from any direction		0.8	0.8
In buoying, if it is also blocked by a significant wall friction taken into account also in calculation		1.0	1.0
In the case of forces originating from dead-load and other permanent weights	gliding, overturning	0.7	0.7
	buoying	0.9	0.9
α_s factor should be defined on the basis of the analysis of all conditions in the case of variable loads and special accessory effects, but its value should be at least		0.7	0.5

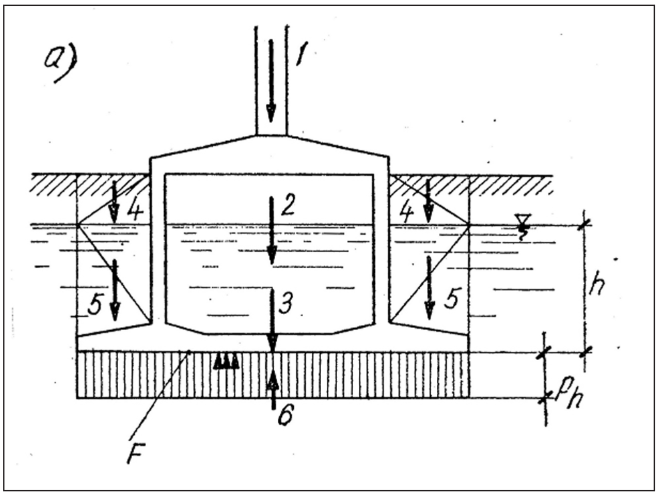


Fig. 2: Interpretation of water buoyancy (so-called normal case)

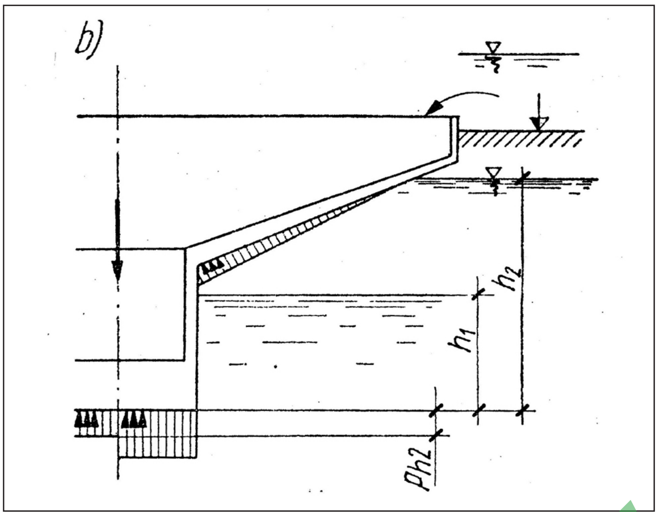


Fig. 3: The shape of the foundation modifies the water buoyancy (the safety is lower than in the case shown in Fig. 2)

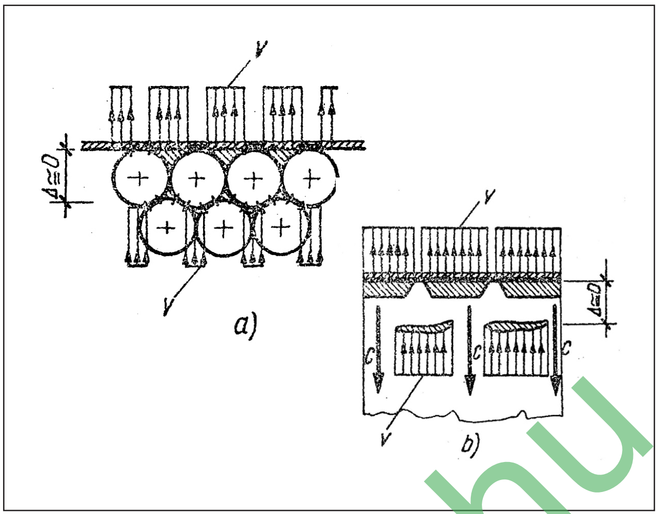


Fig. 4: Buoying with the increase of water level

Here it is mentioned that the base level of an engineering work built on a granular soil without cohesion there is never gliding, because the shear resistance of the subsoil exhausts sooner to the effect of the biased and usually eccentric resultant before. On the other hand the location of the overturning point (*Fig. 5*) may be further indent from the toe point of the base (O') in the point O. The suggested verifying calculation is as follows:

Calculating with the "expected" values of forces according to *Fig. 5.b* the location of the expected O overturning point can be defined by means of the formula shown in this Fig. by defining the R resultant effecting on the ground level and with the knowledge of soil-physical characteristics under the ground-level after the calculation of the soil bearing capacity (σ_t and σ_t'). The safety against this overturning can be calculated in the following manner:

$$n = \frac{a}{e + e_0} \geq 1.5 \tag{4}$$

The $e_0 = 0.10$ value in Eq. (4) assures that initial accidental eccentricity can not be less than 300 mm in the case of centric loading either.

C) Case of gliding

In the calculation according to Eq. (2) it should be taken into account if the immobility of the foundation in glide direction is required or not. If yes, the resistance blocking gliding can only be defined on the base of the earth pressure at rest (*Fig. 6*).

If immobility is not required, the horizontal soil reaction should be calculated, the extreme value of which is equal to the passive soil pressure. In this case friction should only be taken into account with 10 % reduction because of the motion.

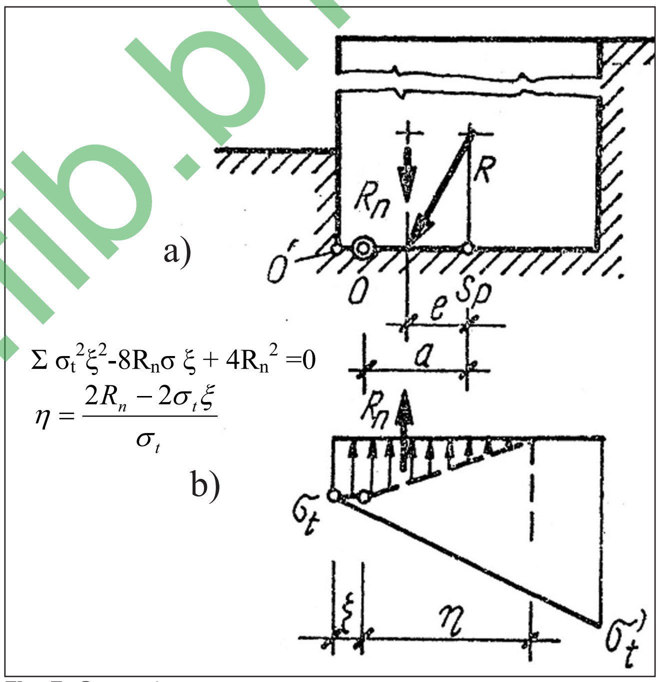


Fig. 5: Overturning
a) forces and possible overturning point
b) calculation of the location of the possible overturning point

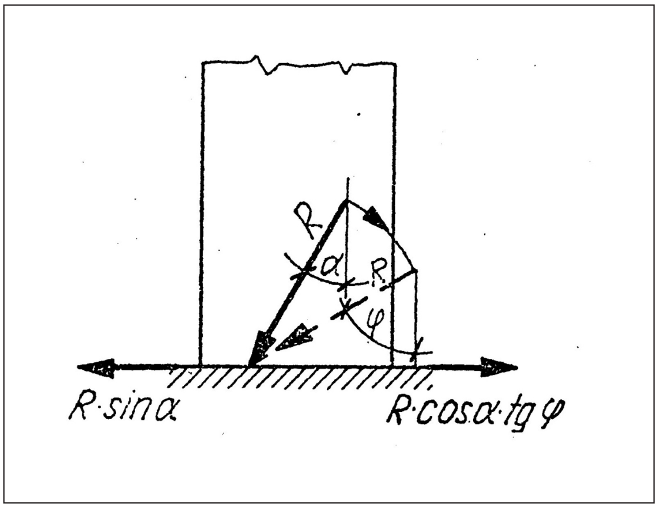


Fig. 6: Gliding

force along a surface of sliding lies deeper. Therefore in such a case the safety against base failure mentioned in case B. should also be checked besides the conventional gliding analysis. In this case eccentricity and angularity should be taken into account together, because the calculation carried out separately, does not lead to a result reliable enough.

4. ANALYSIS WITH THE RELIABILITY METHOD

4.1 Dimensioning according to the Eurocode

The probability of the loss of static equilibrium should always be chosen according to Eurocode (EC) so that it is equal to the probability of any other baring power exhaustion p_{RE} (Farkas, Huszár, Kovács, Szalai, 2006). Accordingly, to the equation of the conditions of the safeness of static equilibrium is – at the end of the designed service life – as follows:

$$\mathrm{Prob}[R(t) - E(t) = \Delta(t) \geq 0] \geq (1\text{-}p_{RE})$$

where

p_{RE} – risk, which is assumed for the loss of the static equilibrium (regarded authoritative in common case according to EN0 p_{RE}=10^{-4})

R(t) – passive effects playing role in static equilibrium

E(t) – active effects playing role in static equilibrium.

The characteristic particularity of the analysis of the static equilibrium of the abutment (see the above) is that in the analysis with partial factor of safety the weight of the abutment is taken into account on the side of the resistance. At the same time the strength of the ground results the resistance from the point of view of the effect of the abutment taken to the subsoil. Besides the analysis of the bearing power of the abutment shown in the previous case can be regarded as the analysis of the static equilibrium.

In the partial safety factor system according to EN0 (in common cases) the risk of the effect side is p_E = 0.01 (1 %), while the risk of resistance is p_R = 0.001 (1 ‰) (*Fig. 7*). It is conspicuous that EN0 does not mention any distinguished (buoying, gliding and overturning) cases.

See below it will be carried out the analysis of the home practice according to EC prescriptions

In addition to, from the point of view of structural analysis the design value (liminal value) of permanent load depends on that in what manner influences the bearing power or the resistance.

The structural situation according to static equilibrium (like any other structural situation) can be analysed: with one-factor procedure or procedure with partial factors; resp. by surveying directly the risk on the basis of reliability procedure.

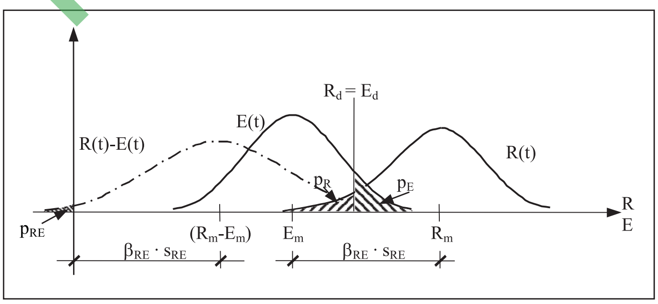


Fig. 7: *Interpretation of the risk at the ultimate limit state*

4.2 Method with partial factors according to EC prescriptions

4.2.1 Principle of the analysis according to EN0

According to EN0 for permanent and temporary design position the combination of the effect (MSZ EN 1990:2004 Eurocode) can be written as:

$$E_d = \left[\sum_{j\geq 1} (\gamma_{G,j,\sup} G_{k,j,\sup} \text{"+"} \gamma_{G,j,\inf} G_{j,\inf}) \text{"+"} \gamma_P P_k \text{"+"} \gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,1} Q_{k,i} \right] \gamma_{Sd}$$

where

$G_{k,inf}$, $G_{k,sup}$ – are lower and upper characteristic values of permanent effects. (In general with a lower liminal value of 5 % and an upper liminal value of 95 %, can be calculated with the formula $G_{k,j,inf} = G_m (1 \pm 1.645 v_G)$, where the diffusion of bulk density of the permanent load is v_G). In lack of appropriate date information $G_{k,inf}$ = 0.95 G_k and $G_{k,sup}$ = 1.05 G_k can also commonly be applied.

γ_{Sd} –safety modifying factor, the value of which is chosen by the designer is depending on the difference of damage proportion in comparison to average value.

Notes:

Value of γ_i partial factor in the case of the *geotechnical* effects:

– *for the analysis of the static equilibrium of a structure or structural member regarded as rigid body*: $\gamma_{Gj,sup}$ =1.0; $\gamma_{Gj,inf}$ = 0.9 and $\gamma_{Q,1} = \gamma_{Q,i}$=1.50, if the effect of Q is unfavourable, but if the effect (from the point of view of the static equilibrium) is favourable than $\gamma_{Q,1} = \gamma_{Q,i}$=0

– *when proving the static equilibrium, if the resistance of structural members should also be taken into account,* $\gamma_{Gj,sup}$ =1.35; $\gamma_{Gj,inf}$ = 1.15 and $\gamma_{Q,1} = \gamma_{Q,i}$=1.50, if the effect of Q is unfavourable, but if the effect is favourable, $\gamma_{Q,1} = \gamma_{Q,i}$=0

– *for defining design values of geotechnical effects:* $\gamma_{Gj,sup} = \gamma_{Gj,inf}$ = 1.0 and $\gamma_{Q,1} = \gamma_{Q,i}$=1.30, if the effect of Q is unfavourable, but if the effect is favourable, $\gamma_{Q,1} = \gamma_{Q,i}$=0.

4.2.2 Dimensioning with partial factors according to EC

In the procedure according to EC0 the condition of static equilibrium can be written by means of the formula

$$R_d \geq E_d$$

where

R_d – design value of the resistance (e.g. design value of the stabilizing effects of the bridge-head)

$$R_d = \sum_{j\geq 1} (\gamma_{G,j,\inf} G_{k,j,\inf})$$

$G_{k,inf}$ = 0.95 G_k – lower characteristic value of the weight (resistance) of the structure, (see above), as well as $\gamma_{Gj,inf}$ = 0.9.

The design value of bearing power is:

$$R_d = 0.95 \cdot 0.9 \cdot G_m = 0.855 G_m \quad (5)$$

E_d – the effect, which can put an end to the static equilibrium (e.g. buoyant force, effect promoting overturning or gliding). Its value is:

$$E_d = \left[\gamma_{Q,1} Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \psi_{0,1} Q_{k,i} \right] \gamma_{Sd} \quad (6)$$

γ_{Sd} –commonly for gliding and overturning with a value of 1.0 and in the case of buoying with a value of 0.9 can be taken into account. (by applying of EN0 recommendation according to its meaning)
$\gamma_Q = 1.50$,
$\psi_{0,i}$=0.7 (in the case of store house: 1.0).
$Q_{k,i}$ – characteristic (usually expected) values of the effect(s) promoting the loss of the overall stability (buoying, gliding or overturning).

In the case of supposing the value $\psi_{0,i}$=1.0 the design value of the effect is:

$$E_d = \sum_{i \geq 1} \gamma_{Q,i} Q_{k,i} \gamma_{Sd}$$

On the basis of the equality of Eqs. (5) and (6) as well as with denoting G_m=R_m and $Q_{k,j}$=Q_m=E_m:

$$0.855\, R_m = E_m\, \gamma_Q\, \gamma_{Sd} \quad (7)$$

nevertheless the condition of static equilibrium taken into account γ_Q = 1.50 and the above values of γ_{Sd} (1.0; 0.9):

$\gamma_Q \cdot \gamma_{Sd}$ = 1.5·1.0=1.50 – in the case of overturning and buoying
= 1.5·0.9=1.39 – in the case of buoying

In the case of the simultaneous manipulation of the two sides (R_m,E_m) the overall stability is appropriate Eq. (7), if the below mentioned conditions are fulfilled:

$$\gamma_{RE} = \frac{R_m}{E_m} \geq (1.50/0.855) = 1.75 \qquad \text{– overturning-gliding}$$

$$\gamma_{RE} = \frac{R_m}{E_m} \geq (1.35/0.855) = 1.58 \qquad \text{– buoying}$$

4.3 Dimensioning for the risk according to EC

According to Hungarian Standard-EN-1990 and on the basis of *Fig. 7* the design value of the bearing power, resp. action effect can be written in the following manner (Farkas, Huszár, Kovács, Szalai, 2006):

$$R_d = R_m \exp(-\beta\, \alpha_R^{(+)}\, \nu_R) \quad (8)$$

$$E_d = E_m (1 - \beta\, \alpha_E^{(-)}\, \nu_E) \quad (9)$$

where
β - reliability index
ν_E and ν_R – relative deviation of the effect, resp. resistance (uncertainty of measuring, geometric and calculation models)
α_i - sensitivity factors

$$\alpha_E = \frac{\kappa_E}{\sqrt{\Sigma \kappa_i^2}} = \frac{E_m \nu_E}{\sqrt{\Sigma \kappa_i^2}}$$

$$\alpha_R = \frac{\kappa_R}{\sqrt{\Sigma \kappa_i^2}} = \frac{R_d \nu_R}{\sqrt{\Sigma \kappa_i^2}}$$

- on the side of the effect (load)

$$\kappa_E = \left[\frac{\delta f}{\delta E_d} \right] s_E \Rightarrow -s_E = -E_m \nu_E$$

- in the case of the resistance

$$\kappa_R = \left[\frac{\delta f}{\delta R_d} \right] R_d \nu_R \Rightarrow R_d \nu_R$$

On the basis of the connections contained by α_i sensitivity factors

$$\sqrt{\Sigma \kappa_i^2} = \sqrt{\kappa_E^2 + \kappa_R^2} = \sqrt{(E_m \nu_E)^2 + (R_d \nu_R)^2}$$

$$\Sigma \alpha_i^2 = \alpha_E^2 + \alpha_R^2 = 1.0$$

Going out from the Eqs. (8) and (9)

$$R_m = \exp\left[\beta \alpha_R^{(+)} \nu_R\right] \cdot E_m (1 - \beta \alpha_E^{(-)} \nu_E)$$

connection arises and within this connection the global factor of safety can usually be indicated in the following way:

$$\gamma_{RE} = \frac{R_m}{E_m} = \exp(\beta \alpha_R^{(+)} \nu_R) \cdot (1 - \beta \alpha_E^{(-)} \nu_E) \quad (10)$$

In accordance with home interpretations so far as the cases of the overall stability – buoying, gliding and overturning – get on being taken as a basis. The cases of the loss of static equilibrium are differentiated according to EN0 prescription due to the basis of the *Table 3*. Further on the consequence of the damage originating from overturning is regarded CC3 classification (β_1 = 3.8), the case of buoying is regarded CC1 as classification (β_3 = 3.3) and the case of gliding is regarded the case having the intermediate consequence between the two other cases. Taking into account the three cases of supposed rate of the damage and on the basis of the relation (10) the γ_{RE} values calculated by means of the block diagram contained by the annexe, are indicated in the *Table 4* as F1 – F3.

$\gamma_{RE}^{(1)}$, $\gamma_{RE}^{(2)}$ and $\gamma_{RE}^{(3)}$ values belonging to relative deviation of (ν_E=0.125 and ν_R=0.10, resp. ν_E=0.25 and ν_R=0.20, as well as ν_E=0.3 and ν_R=0.35) value couples chosen from tables F1-F3 are indicated in *Table 4*.

On the basis of the values contained in *Table 4*. it can be established that the difference between $\gamma_{RE}^{(1)}$ values belonging to buoying and overturning cases is about 7 %. This difference is 12 %, resp. 18 % in the case of $\gamma_{RE}^{(2)}$ and $\gamma_{RE}^{(3)}$. The same difference is simultaneously increasing with the increase of relative deviation. On the basis of this it can be established that in the case of a low relative diffusion of parameters indicated in the two sides of the equation the difference between the three cases of the loss of the static equilibrium is negligible (this is otherwise done by EN0). Then it can be taken into consideration that the distinction is reasonable in the case of a requirement for a more precise calculation.

Table 3: Classification and β values of buildings on the basis of the rate of damage according to EC0

Classification according to rate of damage	Description	Reliability classification	β minimum values
CC3	The probability of loss of the human lives is very high and so economic and social consequences have an extraordinary importance	RC3	4.3
CC2	The probability of loss of the human lives is average and so economic and social consequences are considerable	RC2	3.8
CC1	The probability of loss of the human lives is low and so economic and social consequences are negligible	RC1	3.3

Table 4: Values of safety factors of static equilibrium according to Hungarian Standard-EN 1990

β	$\gamma_{RE}^{(1)}$	$\gamma_{RE}^{(2)}$	$\gamma_{RE}^{(3)}$	The displacement
$\beta_1 = 3.3$	1.63	2.54	4.06	buoying
$\beta_2 = 3.55$	1.69	2.71	4.48	gliding
$\beta_3 = 3.8$	1.75	2.88	4.95	overturning

Note: (1) $\upsilon_E = 0.125$ $\upsilon_R = 0.10$
(2) $\upsilon_E = 0.250$ $\upsilon_R = 0.20$
(3) $\upsilon_E = 0.300$ $\upsilon_R = 0.35$
calculated with the above relative diffusion values.

Table 5: $\gamma_{RE} = R_m / E_m$ values according to different procedures

	Hungarian Standard 54	Hungarian Standard 71	Hungarian Standard 86	Water conservancy prescriptions	Procedure according to Hungarian Standard-EN 1990			
					Partial factors	P_{RE} risk		
						$\upsilon_E=0.125$ $\upsilon_R=0.1$	$\upsilon_E=0.25$ $\upsilon_R=0.2$	$\upsilon_E=0.3$ $\upsilon_R=0.35$
1		1.4		1.39	1.58	1.63	2.54	4.06
2	1.56	1.6	1.56	1.79	1.75	1.69	2.71	4.48
3		2.2				1.75	2.88	4.95

Note: values relating to the case of line 1: buoying; line 2; gliding line 3: overturning.

5. CONCLUSIONS

In above the procedures used before according to Hungarian standard were summarised and solutions were proposed in accordance with EC prescriptions.

The values of the quotient of expected values of the bearing capacity and effect of action are shown in *Table 5*.

From the table it can be ascertain that the difference between the values proposed for the case of buoying and overturning is about 10 % according to the solution proposed on the basis of the procedure based on the system of partial factors according to Hungarian Standard-EN 1990. This difference can also be regarded negligible according to EC prescriptions, but in a given case can also be advantageous, moreover important. The calculation based on risk enables that the design also takes into account the accidental higher uncertainty of the parameters.

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ACCEPTANCE OF CONCRETE COMPRESSIVE STRENGTH


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The new concrete standards give directives regarding the checking if the hardened concrete conforms to the compressive strength requirements of the designed compressive strength class. The acceptance or rejection of conformity is the function of the compressive strength testing methods and the evaluation of the test results. In the paper through examples we show the role of the acceptance probability and the acceptance constant during the evaluation of test results and their significance during the conformity verification procedure.

Keywords: concrete, concrete grade, conformity, continuous manufacturing, continuous testing, identification testing, acceptance probability, acceptance constant, design value

1. INTRODUCTION

According to Table 3.1. of MSZ EN 1992-1-1:2005 (Eurocode 2) standard, during the design of concrete structures the mean compressive strength value $f_{cm,cyl}$ is derived from the characteristic compressive strength value $f_{ck,cyl}$ by using the following relationship:

$$f_{cm,cyl} = f_{ck,cyl} + 8 \quad [\text{N/mm}^2] \tag{1}$$

This relationship is valid for cylindrical samples of 150 mm diameter, 300 mm high, 28 days of age and cured under water throughout the time (wet cured). Generally the concrete technology tests the conformity of compressive strength on cubes with the sizes of 150 mm, at the age of 28 days which were mix cured (first 7 days under water, 21 days on air).

If we accept that according to the new concrete standard, namely MSZ EN 206-1:2002 European standard and it's roles of application in Hungary MSZ 4798-1:2004 document, until the compressive strength class C50/60:

$f_{c,cube}/f_{c,cyl}$ = 0.97/0.76 is the ratio between the wet cured, cylindrical samples of 150 mm diameter, 300 mm high and cubic samples with the sizes of 150 mm, and

$f_{c,cube}/f_{c,cube,H}$ = 0.92 is the ratio of the compressive strengths of the wet cured and mix cured normal concrete cubic samples with the sizes of 150 mm,

then the connection between the compressive strengths of the mix cured cubic samples with the sizes of 150 mm ($f_{c,cube,H}$) and the wet cured cylindrical samples of 150 mm diameter, 300 mm high ($f_{c,cyl}$) can be expressed as

$$f_{c,cube,H} = 0.97/(0.76\times0.92)\times f_{c,cyl} \sim 1.387\times f_{c,cyl}, \tag{2}$$

which after substituting into the right and left side of relationship (1) will derive to

$f_{cm,cube,H}/1.387 = f_{ck,cube,H}/1.387 + 8$ [N/mm²], and from here we can arrive to the

$$f_{cm,cube,H} = f_{ck,cube,H} + 11.1 \ [\text{N/mm}^2]$$

expression. This gives the relation of the mean compressive strength and characteristic compressive strength of the 28 days old sample cubes with edges of 150 mm, mix cured, according to MSZ EN 1992-1-1:2005 until the compressive strength class of C50/60.

So by the approach of MSZ EN 1992-1-1:2005 for example the concrete of C25/30, compressive strength class at the age of 28 days, mix cured, should have a minimum mean compressive strength determined using cubes with 150 mm edges of

$$f_{cm,cube,test,H} \geq f_{cm,cube,H} = 33 + 11.1 = 44.1 \quad [\text{N/mm}^2].$$

Based on the MSZ EN 206-1:2002, and the MSZ 4798-1:2004 Hungarian standards the concrete having the compressive strength class of C25/30 at the age of 28 days, mix cured and the compressive strength is determined on cubes with the sizes of 150 mm will have a mean compressive strength instead of the above calculated $f_{cm,cube,H}$ = 44.1 N/mm² only $f_{cm,cube,H} = f_{ck,cube,H} + 1.48\cdot1.387\cdot\sigma_{min} = 33 + 1.48\cdot1.387\cdot3 =$ 39.2 N/mm². A certain safety margin is given by that between the design compressive strength value of concrete (f_{cd}) and the characteristic value of it defined on standard cylindrical samples with 150 mm diameter, 300 mm height and wet cured ($f_{ck,cyl}$) exists the relation of $f_{cd} = f_{ck,cyl}\cdot\alpha_{cc}/\gamma_c$, where γ_c = 1.5 is the safety factor of concrete compressive strength in the ultimate limit state and α_{cc} = 0.85 is the decrease factor taking into consideration the long term load bearing capacity. Accordingly for example the design compressive strength of a concrete (f_{cd}) defined on standard cylinders which were wet cured is f_{cd} =14.2 N/mm².

If it is found during the statical calculation – not taking into consideration the environmental conditions – that the practice would require a concrete having f_{cd} = 14.2 N/mm² design value, then the designer, based on MSZ EN 1992-1-1:2005 will prescribe concrete of C25/30 compressive strength class. To this class, according to the above $f_{cm,cube,H}$ = 44.1 N/mm² mean compressive strength belongs, considering mix cured standard cubes. At the same time the mixing plant will satisfy the prescription – based on MSZ 4798-1:2004 – with a concrete having a mean compressive strength of $f_{cm,cube,H}$ = 39.2 N/mm² considering also mix cured standard cubes.

The deviation arises from the different relationship calculation of the characteristic and mean values, which is being further complicated by the unusualness of the acceptance constant (λ_n) given in the new concrete standards.

Generally the hardened concrete is to be characterized by compressive strength and body density, in special cases by frost resistance, corrosion resistance, water permeability, resistance against wear etc., and according to these properties, based on MSZ EN 206-1:2002, and the MSZ 4798-1:2004 Hungarian standards should be classified. The 4.3. paragraph of MSZ 4798-1:2004 Hungarian standard deals with the classification of hardened concrete, the testing and requirements are given in paragraph 5.5., the conditions of conformity and the controlling procedures are dealt with in the 8. paragraph.

The concrete satisfies the compressive strength requirements if it complies to paragraph 8.2.1. and appendix A and B of MSZ 4798-1:2004 Hungarian standard regarding the compressive strength and the body density.

During compressive strength testing and evaluation of the test result differentiation must be made between the discrete concrete constitutions sampling and testing procedure, while in the conformity conditions between initial production and testing, the continuous production and testing and the identifying testing procedure.

The evaluation of the continuous production and identifying testing is to be done by the mean value (f_{cm}) and standard deviation (s_n) of the test values.

The characteristic value (f_{ck}) and so the derived compressive strength class is significantly influenced by the acceptance constant (λ_n):

$$f_{ck} = f_{cm} - \lambda_n \times s_n \qquad (3)$$

2. THE CONTINUOUS PRODUCTION AND TESTING

The continuous production of concrete starts at the time when at least 35 consequently under the same conditions produced concrete test results are available within the period of longer than three month but not more than twelve month, and this is called by the new standards the initial production. From the test results of the initial production the standard deviation must be calculated (σ), which gives a good approximation of the theoretical standard deviation and which under certain circumstances should be taken into consideration during the evaluation of the test results of the continuous production.

$$\sigma = \sqrt{\frac{\sum_{i=1}^{n}\left(f_{ci} - f_{cm,test}\right)^2}{n-1}} = \sqrt{\frac{\sum_{i=1}^{n} f_{ci}^{\,2} - n \cdot f_{cm,test}^{\,2}}{n-1}}, \quad \text{where } n \geq 35.$$

From here on the smallest value of the σ standard deviation, in the case of wet cured standard cylinders:

- in case of normal concrete (if the compressive strength class is $\leq$ C50/60): 3 N/mm²;
- in case of high strength concrete (if the compressive strength class is $\geq$ C55/67): 5 N/mm².

The result of continuous production can be evaluated by at least 15 consequent sampling and testing within a maximum period of 12 month. The samples are to be taken continuously during the production but not more frequently than one sample out of every 25 m³.

In case of continuous production one sample may only be one test cylinder or cube. At the beginning of continuous production, until 15 samples are not yet available the number of samples should be complemented with the samples taken at the end period of the initial production.

For the evaluation of the results of continuous production must be given at least 15 test results, the average of the at least 15 test results, and using the following expression must be calculated the standard deviation of the at least 15 test results:

$$s_n = \sqrt{\frac{\sum_{i=1}^{n}\left(f_{ci} - f_{cm,test}\right)^2}{n-1}} = \sqrt{\frac{\sum_{i=1}^{n} f_{ci}^{\,2} - n \cdot f_{cm,test}^{\,2}}{n-1}}, \quad \text{where } n \geq 15.$$

During the continuous production the concrete satisfies the requirements of the compressive strength class (conformity) if the following conditions are fulfilled at the same time:

1. condition in case of all compressive strength class according to table 14. of MSZ 4798-1:2004 Hungarian standard in case of wet cured standard cylinders:

$$f_{cm,test} \geq f_{cm} = f_{ck} + 1.48 \cdot \sigma$$

where σ is the standard deviation calculated from the initial production, based on the test results of at least 35 sample and 1.48 is the value of the acceptance constant ($\lambda_{n=15}$) (Table 6). The smallest value of standard deviation to be taken into consideration in case of wet cured standard cylinders:

- in case of normal concrete (if the compressive strength class is $\leq$ C50/60): 3 N/mm²;
- in case of high strength concrete (if the compressive strength class is $\geq$ C55/67): 5 N/mm²;
- further on for all types of concrete: $0.63 \cdot \sigma \leq s_n \leq 1.37 \cdot \sigma$; that is in case of samples from continuous production the s_n experienced standard deviation determined on a minimum of 15 samples may not be less than 0.63 times the σ theoretical standard deviation determined from the minimum of 35 samples of the initial production and may not be more then 1.37 times of the same σ value.

If the above requirement of the standard regarding the standard deviation is satisfied then the standard deviation σ determined during the initial production may be used in the period of continuous production for the compliance checking of conformity.

If the above requirement of the standard regarding the standard deviation is not satisfied, then based on the test results of the latest at least 35 samples (since continuous production is assumed, at least 35 test samples) a new value for standard deviation σ must be calculated.

If the manufacturer is unable to prove the value of standard deviation for the initial production period, then for wet cured standard cylinders we must calculate with the value of $\sigma \geq 6$ N/mm² (8.2.1.3. paragraph of MSZ 4798-1:2004 Hungarian standard).

2. condition according to Table 14 of MSZ 4798-1:2004 Hungarian standard in case of wet cured standard test cylinders:

- in case of normal concrete (if the compressive strength class is $\leq$ C50/60): $f_{ci} \geq f_{ck} - 4$ [N/mm²];
- in case of high strength concrete (if the compressive strength class is $\geq$ C55/67): $f_{ci} \geq 0.9\, f_{ck}$.

The MSZ EN 1992-1-1:2005 and the MSZ EN 206-1:2002 standards set up conformity for the compressive strength of concrete based on compressive strength test results of cylinders having a diameter of 150 mm and a height of 300 mm which were kept under water through the time of 28 days. Due to this reason in case of evaluating compressive strength results of cubes – having sizes of 150 mm and wet or mix cured – we do not make mistakes if we calculate the individual result values to the compressive strength of the wet cured standard cylinders and evaluate these values by taking into consideration the conditions of conformity.

According to paragraph 5.5.1.2. and N2. topic of MSZ 4798-1:2004 Hungarian standard until the compressive strength class of C50/60 the ratio of the compressive strengths of the standard cylinder and cube if wet cured according to expression (2) is $f_{c,cube,H}/f_{c,cyl} = 0.97/(0.76 \cdot 0.92) \sim 1.387$.

Implicitly using the exchange rate given in the national application document's NAD 3.2. note in MSZ 4798-1:2004 standard and dividing by this factor the measured compressive strength of a cube with the sizes of 150 mm mix cured, we can get to the compressive strength of a standard cylinder with 150 mm in diameter and 300 mm height which was wet cured as can be seen in the numerical example.

A further condition of conformity is that the fresh concrete test samples which are prepared for compressive strength testing may not alter in their body density more then ± 2 % from the designed density of the concrete.

For the evaluation of compressive strength test results in case of continuous production a numerical example is given in *Table 1*.

Table 1: Numerical example for the evaluation of compressive strength test results in case of continuous production

Sign of sample (1 sample = 1 specimen)	Sample cube $f_{ci,cube,test,H}$	Sample cylinder $f_{ci,cyl,test}$	*2. condition* $f_{ci,cyl,test} \geq f_{ck,cyl} - 4$
1.	47.1	34.0	34.0 > 21.0
2.	45.4	32.7	32.7 > 21.0
3.	44.3	31.9	31.9 > 21.0
4.	47.9	34.5	34.5 > 21.0
5.	49.3	35.5	35.5 > 21.0
6.	44.8	32.3	32.3 > 21.0
7.	45.0	32.4	32.4 > 21.0
8.	46.9	33.8	33.8 > 21.0
9.	48.8	35.2	35.2 > 21.0
10.	44.9	32.4	32.4 > 21.0
11.	46.7	33.7	33.7 > 21.0
12.	44.5	32.1	32.1 > 21.0
13.	44.0	31.7	31.7 > 21.0
14.	46.2	33.3	33.3 > 21.0
15.	44.8	32.3	32.3 > 21.0
	$f_{cm,cyl,test} =$	**33.2**	mean
	$s_{15} =$	1.21	standard deviation
	$s_{min} =$	3.0	standard deviation at least
	$\sigma_{35} =$	1.77 → **3.0**	$= \sigma_{min}$ from initial production
$0.63 \cdot \sigma_{min} = 1.89 < s_{min} = 3.0 < 4.11 = 1.37 \cdot \sigma_{min}$			
$f_{ck,cyl,test} = f_{cm,cyl,test} - 1.48 \cdot \sigma_{min} = 33.2 - 4.4 =$ **28.8**			
1. condition $f_{ck,cyl,test} =$		$28.8 > 25 = f_{ck,cyl}$	
		$f_{cm,cyl,test} = 33.2 > 29.4 = f_{cm,cyl} = f_{ck,cyl} + 1.48 \cdot \sigma_{min}$	
Compressive strength class: **C25/30**		Unit: N/mm²	

3. THE ACCEPTANCE CONSTANT

In Table 14. of the new concrete standards MSZ EN 206-1:2002 and MSZ 4798-1:2004 for the 1. criteria of compressive strength conformity of continuous production the acceptance constant is the λ_n value, by which multiplying the standard deviation s_n, or σ and so obtaining $\lambda_n \cdot s_n$, or $\lambda_n \cdot \sigma$ respectively (lead value). By subtracting these from the mean compressive strength value f_{cm} we arrive to the characteristic strength f_{ck} of concrete. The sign of λ_n in case of *t* distribution: t_n.

If values of f_{cm}; s_n; and λ_n are given than the characteristic compressive strength can be calculated using formula (3):

$$f_{ck} = f_{cm} - \lambda_n \cdot s_n.$$

In the new concrete standards (MSZ EN 206-1:2002 and MSZ 4798-1:2004) in the 1. criteria for the conformity of continuous production for $n = 15$ samples the given lead value $\lambda_{n=15} = 1.48$. This is practically taking the role of the *Student's* factor used by the previously valid Hungarian standard (MSZ 4720-2:1980). The value of this *Student's* factor depend on the number of samples and it was in all cases at least 1.645 but in case of small sample number this was significantly higher. The change of the standard – at least in this point – is by no question in the favour of the producer since the smaller the multiplicator of the 1. criteria, the easier to satisfy the condition. In order to understand the reasons which explain the value $\lambda_{n=15} = 1.48$ of the acceptance constant (and the values given in the previous standard) it is useful to think over the principles of concrete classification by compressive strength. We must indicate that for the acceptance constant values given in both the previous standards and the 1.48 are mathematical statistically absolutely correct, but – and this is the real reason for the difference – under totally different circumstances. In our explanation we mainly lean on the papers of Taerwe (1986) and Zäschke (1994).

The requirement to be able to classify the concrete into compressive strength classes is that if we would be able to examine the total amount of the used concrete (such being able to determine the distribution of the compressive strength), 95% of the so obtained results (typical value) should reach the predetermined f_{ck} characteristic strength. We may also say that the 5% quantile of the distribution of the compressive strength ($f_{ck,test}$) is bigger or equal to f_{ck} ($f_{ck} \leq f_{ck,test}$).

To the typical value of the compressive strength values of concrete belong that portion, which do not reach the f_{ck} threshold value. The portion of the strength values below the f_{ck} is usually denoted by *p*, the value of which is between 0 and 1 (many case expressed in percentages). The $f_{ck} \leq f_{ck,test}$ requirement we can express with the help of this portion in the form of $p \leq 5\%$.

Would we know the value of *p*, our task would be simple, since if $p \leq 5\%$ we would accept the sample and reject in other cases. Naturally we never know the value of *p* (since for that we would have to examine the total concrete lot as a sample), therefore we need different statistical methods. In all applied method is common the assumption of normal (Gaussian) distribution of the obtained compressive strength values. Further we assume that the test results follow a normal distribution with an unknown expected value μ and σ standard deviation. In this case the 5% quantile of the distribution can be determined by the $f_{ck,test} = \mu - 1.645 \cdot \sigma$ formula.

The *Student's* factors in the earlier Hungarian standard MSZ 4720-2:1980 are explained by elementary mathematical facts. If we know the standard deviation σ of the compressive strength, then the average of the test results $f_{cm,test}$ will give the undistorted estimation of the expected value μ and such $f_{cm,test}$ - $1.645{\cdot}\sigma$ is a natural estimation of the 5% quantile. In the standard the condition $f_{ck} \le f_{cm,test}$ - $1.645{\cdot}\sigma$ expresses exactly that the estimated value of the 5% quantile ($f_{ck,test}$) must remain above the prescribed strength limit which is (f_{ck}).

In case we do not know the standard deviation, then the situation is slightly more complicated since it has to be also estimated. For such a case the

$$\frac{f_{cm}-\mu}{\sigma_n}\sqrt{\frac{n}{n-1}}$$

quantity will follow the so called *Student*-type *t*-distribution with n - 1 freedom and the value of the 5% quantile can be estimated by taking the value of the *t*-distribution from table.

Based on the available data the methods given in MSZ 4720-2:1980 in case of both known and unknown standard deviation estimated the value of $f_{ck,test}$ empirical parameter which was to be matched to the critical prescribed characteristic value f_{ck}. Due to the symmetry of the used probability distributions the property of the so obtained method is that if the manufacturer produced concrete of just „critically good" (that is p = 5%) then the concrete was accepted with a probability of around 50%. Shall we introduce the *A(p)* acceptance probability of the concrete having p characteristic strength, - which would show that with what probability will we accept a concrete having a p portion of the strength values below f_{ck} – then it's meaning is $A(0.05) \approx 0.5$.

The new Hungarian standards MSZ EN 206-1:2002 and MSZ 4798-1:2004 wish to ensure the compressive strength acceptance of the concrete in such a way, which is part of a more widely understood quality control system.

In case of any acceptance, criteria can be interpreted for a concrete with a given p characteristic value and a corresponding $A(p)$ acceptance probability. If we plot $A(p)$ as a function of p then we obtain the acceptance curve (*Fig. 1*).

The base of the acceptance decision of the new concrete standards (MSZ EN 206-1:2002 and MSZ 4798-1:2004) is the next train of thoughts (Taerwe, 1986, Zäschke, 1994): we would like a quality control system which satisfies for all p characteristic values the criteria of

$p{\cdot}A(p) \le 5\%$

the uppermost curve (*Fig. 1*). Further we can state that the criteria system in the new concrete standard satisfies this criteria in the upper curve of Fig. 1, since for example:

if	p = 0.05 then	$A(p) \le 1.0$
if	p = 0.07 then	$A(p) \le 0.7$
if	p = 0.10 then	$A(p) \le 0.5$
if	p = 0.25 then	$A(p) \le 0.2$

Any acceptance criteria can be taken as a kind of filter: retains the unacceptable samples and lets pass the acceptable ones. Let us assume that we are testing such a material – e.g. rebar – the conformity of which can be checked before application and we are testing it continuously. The confirming lot is being used while the nonconforming lots are substituted by perfect quality ones. In this case obviously the quality of the total used lot will be better due to the filtering of the acceptance criteria. The $p{\cdot}A(p) \le 5\%$ criteria ensures that the p value of the filtered lot would remain below 5%. It is important

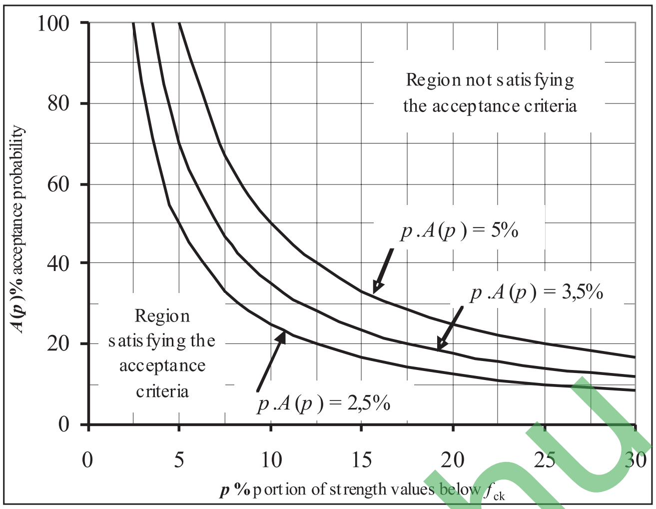

Fig. 1: Acceptance curves

to emphasize on the point that even in this case a *continuous* (and not random) *testing* is to be assumed.

In case of concrete obviously the nonconforming deliveries cannot be substituted with perfect quality ones since by the time the nonconformity is realized the material was long before used. The $p{\cdot}A(p) \le 5\%$ criteria will only have sense if the continuous tracking of the usage location of the material is assumed and the part of the structure where the nonconforming material is applied would be post reinforced or by any other means we achieve that it would be practically perfect. (That is the non conforming transports will be transformed to confirming ones afterwards.)

If the poured concrete is *continuously* checked and the non conform parts are afterwards transformed to „perfect", then the $p{\cdot}A(p) \le 5\%$ criteria really ensures that in the completed structure the amount of concrete below the f_{ck} compressive strength value (prescribed characteristic value) will be below 5%.

In Table 14. of the new concrete standards (MSZ EN 206-1:2002 and MSZ 4798-1:2004) in the 1st criteria of compressive strength acceptance in case of continuous production the $\lambda_{n=15}$ = 1.48 value for the acceptance constant belonging to n = 15 samples was determined by *assuming such a continuous control and post reinforcing* quality control system. The quality control system obtained this way will surely satisfy the $p{\cdot}A(p) \le 5\%$ criteria by a significant safety margin (Zäschke, 1994). It will allow for example that if a mixing plant is producing „critically good" concrete, that is concrete with a portion of the strength values below the f_{ck} p = 5% then the probability of the acceptance $A(0.05)$ is to be 1.0 (the top curve of *Fig. 1.* where $A(0.05) = 1.0$). The quality control system provided by the $\lambda_{n=15}$ = 1.48 constant $A(0.05) \approx 0.7$, meaning that if the produced concrete is „critically good", then the criteria system will accept it with a probability of about 0.7 (the middle curve of *Fig. 1.* where $A(0.05) = 0.7$). This probability value is significantly smaller then 1.0 which is required by the $p\,A(p) \le 5\%$ basic criteria, but remarkably more then 0.5 which is ensured by MSZ 4720-2:1980 (the bottom curve of *Fig. 1.* where $A(0.05) = 0.5$). The reason for the safety margin is partly that the $\lambda_{n=15}$ = 1.48 acceptance constant is calculated by such a model, which allows a weak correlation between the test results (If we measure a lot, then between the results obtained close in time will be some correlation.). Would we assume that the test results are absolutely independent, then for the $\lambda_{n=15}$ = 1.48 value we would obtain 1.318. The values of λ_n acceptance constants are belonging to an offered *OC* curve, and were determined by numerical simulation based on random numbers (Taerwe, 1986). The values of λ_n acceptance constants are in *Table 6*.

By comparing the old MSZ 4720-2:1980 and the new MSZ EN 206-1:2002 and MSZ 4798-1:2004 Hungarian standards, we can see that the old one wants by a randomly sampling acceptance criteria while the new ones want by continuous testing and post repair possibility ensuring acceptance criteria to become a part of the quality control system. The problem with MSZ EN 206-1:2002 and MSZ 4798-1:2004 standards is that, they contain only the acceptance criteria but the requirement of continuous testing and post repair is missing.

Until now we have dealt with only the 1st, compressive strength criteria of the new standards (MSZ EN 206-1:2002 and MSZ 4798-1:2004) which we could do because according to the practice and simulation tests the 2nd criteria has almost no influence on the criteria system (Zäschke, 1994).

4. IDENTIFICATION TESTING OF COMPRESSIVE STRENGTH

The identification testing of the compressive strength of concrete is to be carried out — according to Annex B of MSZ 4798-1:2004 Hungarian standard — if we want to know that:

- the fresh concrete belongs to the same lot for which the producer certified the conformance of the characteristic strength;
- the fresh concrete conforms with the strength class or other properties which are warranted by the producer, if the producer did not carry out tests for the confirmation certification;
- the hardened concrete in the structure is conforming to the compressive strength class which is warranteed by the producer.

According to our understanding *identification testing* is done by an *independent laboratory*, if its task is not the conformance testing of initial or continuous production (it was carried out by the producer or by an other laboratory), but – by the order of either the producer or the client – the task is only the determination of that the compressive strength class of the concrete is according to the one which is certified by the producer. Same, identification type testing can be done by the client or the contractor in his own laboratory. It is advisable to agree with the producer in the circumstances of the tests and to carry out the tests in his presence. Initial or continuous testing may only be carried out by the producer or his trustee, and based on the result the producer – if necessary by using a qualifying body – issues the conformance statement. The reliability of the conformance statement is checked through identification testing by the customer or his trustee.

In the number of samples „*n*" and the location where the sampling is done the interested parties (prescriber, client, producer) must agree previously in writing.

The result of the handing over procedure of concrete, the acceptance or rejection of the lot depend on the result of the *identification test*. From the point of the safety of our structures it can be appreciated if in this procedure differently from the basic idea of the new standards the risk of the two handing over parties is the same, in other words if the acceptance probability $A = 50\%$ of the concrete having $p = 5\ \%$ portion of the strength values below the f_{ck}, and the results of the compressive strength tests are evaluated to this acceptance criteria ($p\ A(p) = 2.5\ \%$). Our offer is not opposing the new standards, it is stricter and leads to the increase of the safety of concrete and reinforced concrete structures and the application does not require the finding of the non conforming concrete or post reinforcement. The procedure can be applied by separate agreement of the interested parties.

The mathematical statistical base of the offered conformity criteria of the identification test is not strange neither to the new (MSZ EN 206-1:2002 and MSZ 4798-1:2004) nor to the old (MSZ 4719:1982 and MSZ 4720-2:1980) Hungarian standards and may be summarized as follows:

- during identification testing of concrete we make no difference if the concrete is produced by production quality control or not;
- based on the mean value, the standard deviation of the compressive strength and the number of samples is the conformity of the concrete determined;
- we assume that the test results follow the normal distribution;
- the characteristic value is ordered to the 5% underfalling portion level based on normal distribution in such a way that in the handing over procedure the acceptance probability of the critically conforming concrete is about 50-50%, the acceptance criteria is to be $p{\cdot}A(p) = 2.5\ \%$ against the order of MSZ EN 206-1:2002 and MSZ 4798-1:2004 Hungarian standards, according to which in case of continuous production the acceptance-rejection probability for critically conforming concrete is around 70-30% and the acceptance criteria $p\ A(p) = 3.5\ \%$ (Taerwe, 1986);
- the characteristic value in case of more then 40 samples is determined by using the $f_{ck} = f_{cm} - 1.645{\cdot}\sigma$ formula and in case of less samples (n) the $f_{ck} = f_{cm} - t_n{\cdot}s_n$ formula is to be used, where σ is the theoretical standard deviation, s_n is the empirical standard deviation, t_n is the *Student's* factor (Stange et al., 1966) as a function of the number n of the samples;
- assume that until the compressive strength class C50/60 the mixed cured sample cubes with 150 mm edge length and the 150 mm in diameter and 300 mm in height under water cured cylindrical samples compressive strength have a relation of the following formula $f_{ci,cube,H} = 1.387{\cdot}f_{ci,cyl}$, which is also valid for the prescribed standard deviation values, that is $\sigma_{cube,H} = 1.387{\cdot}\sigma_{cyl}$ and $s_{cube,H} = 1.387{\cdot}s_{cyl}$;
- the sample may be of only one piece;
- the procedure may also be used in case of wet cured (under water) standard cubes and cylinders.

The concrete is conforming the designed compressive strength class if the next criteria are simultaneously satisfied:

1. criteria:

$f_{cm,cyl,test} \geq f_{cm,cyl} = f_{ck,cyl} + t_n{\cdot}s_n$

where the value of s_n may not be smaller then the value in *Table 2,* the smallest allowed standard deviation (s_{min});

t_n is the *Student's* factor belonging to 5% underfalling portion level and n sample number with a freedom of $n - 1$, at 50% acceptance probability, the values of which are in *Table 6*.

2. criteria:

in case of ≤ C50/60 compressive strength class normal concrete:

$f_{ci,cyl} \geq f_{ck,cyl} - 4$ [N/mm²];

in case of ≥ C55/67 compressive strength class concrete:

$f_{ci,cyl} \geq 0.9{\cdot}f_{ck,cyl}$.

The further requirement of conformity statement is that the individual body density of the *fresh concrete* samples prepared

Table 2: The sample number, the Student's factor, and the smallest permitted values of the standard deviations for compressive strength identification test using the Student's factor (offer)

Concrete properties	Without conformity statement	With conformity statement			
		In case of serial production			
Compressive strength class	Unique (not serial) production, in all cases	C8/10 – C16/20		C20/25 – C50/60	C55/67 – C100/115
Concrete designed components		Designed concrete, prescribed concrete and prescribed industrial concrete		Designed concrete and prescribed concrete	
Exposure class		XN(H), X0b(H), X0v(H)	Other classes	All exposure classes	
Number of samples, at least, *n*	-	200 m³	150 m³	100 m³	50 m³
	-	at least 1 of each concrete volume, but at least 1 of each lot			
	3	3	6	9	9
The *Student's* factor t_n belonging to the 5% underfalling portion level, by 50 % acceptance probability, in the function of *n* required sample number (Stange et al., 1966)					
t_n, if the freedom *n*-1	2.920	2.920	2.015	1.860	1.860
Smallest allowed value of standard deviation, in case of wet (under water) cured, standard cylinders, s_{min}, N/mm²	6	2	3	3	5

for compressive strength tests may not alter more then ± 1.5 % from the designed value. (This requirement is 0.5% less then the loose value given in MSZ 4798-1:2004 Hungarian standard and it is according to 15 liter/m³ air content.)

The number of samples, the *Student's* factors, and the smallest permitted values of the standard deviations for the offered compressive strength identification test of sample cubes are given in *Table 2*.

An example is given for qualification of concrete based on 9 samples (9 standard cubes) identification test results, according to the offered acceptance criteria (*Table 3*.)

A numerical example is given in *Table 4* in which for comparison, the evaluation of the compressive strength test results given in *Table 3* is done in case of having conformity statement, by the standard identifying testing method. In *Table 5* we have prepared such a numerical example, in which the evaluation of the compressive strength test results given in *Table 3.* is done – also for comparison – according to the „old" (MSZ 4719:1982 and MSZ 4720-2:1980) Hungarian standards.

In *Table 6* we give *Student's* factors belonging to the 5% underfalling portion level by 5% acceptance probability (Stange et al., 1966). The *Student's* factors in *Table 6.* are the distribution of $N(0.1)$ *t*-distribution – belonging to the one side 5% underfalling portion level – $t_{95\%,f}$ is the statistical variable (quantile of $p = 0.05$ value, threshold value, if the number of samples is *n*, and *n*-1 is the freedom of the *t*-distribution). These values differ in a certain amount from the *Student's* factors given in MSZ 4720-2:1980 Hungarian standard, because those were determined by approximation (Owen, 1962; Palotás, 1979, 9.93.4. point; Szalai, 1982, 2.8.5. point). If $n \rightarrow \infty$,

Table 3: Numerical example for the evaluation of compressive strength test results in case of identification testing procedure using Student's factor (offer)

Sign of sample (1 sample = 1 test specimen)	Sample cube $f_{ci,cube,test,H}$	Sample cylinder $f_{ci,cyl,test}$	2. criteria $f_{ci,cyl,test} \geq f_{ck,cyl}$ - 4
1.	48,7	35.1	35.1 > 21.0
2.	47.7	34.4	34.4 > 21.0
3.	44.5	32.1	32.1 > 21.0
4.	46.6	33.6	33.6 > 21.0
5.	45.8	33.0	33.0 > 21.0
6.	47.6	34.3	34.3 > 21.0
7.	43.1	31.1	31.1 > 21.0
8.	43.8	31.6	31.6 > 21.0
9.	46.2	33.3	33.3 > 21.0
	$f_{cm,cyl,test}$ =	**33.2**	Mean value
	s_9 =	1.37	Standard deviation
	s_{min} =	**3.0**	Minimum Standard deviation
	t_9 =	1.86	*Student's* factor
$f_{ck,cyl,test} = f_{cm,cyl,test} - t_9 \cdot s_{min} = 33.2 - 5.6 = \mathbf{27.6}$			
1. criteria $f_{ck,cyl,test} = 27.6 > 25.0 = f_{ck,cyl}$ $f_{cm,cyl,test} = 33.2 > 30.6 = f_{cm,cyl} = f_{ck,cyl} + t_9 \cdot s_{min}$			
Compressive strength class: **C25/30**		Unit: N/mm²	

then the *Student's t*-distribution tends to the *Gaussian* normal distribution (*Fig. 2*).

Table 4: Numerical example for the evaluation of the compressive strength test results given in Table 3. is done in case of having conformity statement, by the standard identifying testing method.

Sign of sample (1 sample = 1 test specimen)	Sample cube $f_{ci,cube,test,H}$	Sample cylinder $f_{ci,cyl,test}$	*2. criteria* $f_{ci,cyl,test} \geq f_{ck,cyl} - 4$
1.	48.7	35.1	35.1 > 21.0
2.	47.7	34.4	34.4 > 21.0
3.	44.5	32.1	32.1 > 21.0
4.	46.6	33.6	33.6 > 21.0
5.	45.8	33.0	33.0 > 21.0
6.	47.6	34.3	34.3 > 21.0
7.	43.1	31.1	31.1 > 21.0
8.	43.8	31.6	31.6 > 21.0
9.	46.2	33.3	33.3 > 21.0
	$f_{cm,cyl,test}$ =	**33.2**	mean value
	$f_{ck,cyl,test} = f_{cm,cyl,test} - 4 =$	**29.2**	
1. criteria $f_{ck,cyl,test} = 29.2 > 25.0 = f_{ck,cyl}$ $f_{cm,cyl,test} = 33.2 > 29.0 = f_{cm,cyl} = f_{ck,cyl} + 4$			
Compressive strength class: **C25/30**		Unit: N/mm²	

Table 5: Numerical example, in which the evaluation of the compressive strength test results given in Table 3. is done according to the „old" (MSZ 4719:1982 and MSZ 4720-2:1980) Hungarian standards.

Sign of sample (1 sample = 1 test specimen)	Sample cube $f_{ci,cube,test,H}$	Evaluation according to the MSZ 4719:1982, the MSZ 4720-2:1980, and the MÉASZ ME-04.19:1995 Hungarian regulations
1.	48.7	
2.	47.7	
3.	44.5	
4.	46.6	
5.	45.8	
6.	47.6	
7.	43.1	
8.	43.8	
9.	46.2	
$R_{m,cube,test}$ =	**46.0**	mean value
s_9 =	1.89	Standard deviation
$s_{min,cube}$ =	**2.0**	Minimum standard deviation, MSZ 4720-2:1980
t_9 =	1.82	MÉASZ ME-04.19:1995 → Table 4.18.
k_R =	1.24	MÉASZ ME-04.19:1995 → Formula 4.61.
$R_{k,cube,test}$ =	**41.5**	$= 46.0 - 4.5 = R_{m,cube,test} - k_R \cdot t_9 \cdot s_{min}$
Criteria $R_{k,cube,test} = 41.5 > 40.0 = R_{k,cube}$ $R_{k,cube} = 40.0 \rightarrow 35.0 = R_{k,cyl}$		
Compressive strength class: **C35/40**		Unit: N/mm²

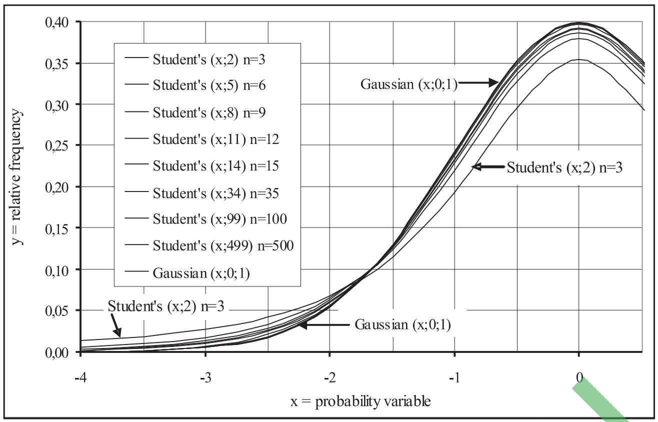

Fig 2: Standardised frequency curves of Gaussian and Student's distributions

Table 6: Acceptance constants

Number of samples n	Freedom in case of *Student's*-distribution n - 1	*Student's* factor t_n	*Taerwe* factor λ_n
	(Stange et al., 1966)		(Taerwe, 1986)
2	1	6.314	
3	2	2.920	2.67
4	3	2.353	2.20
5	4	2.132	1.99
6	5	2.015	1.87
7	6	1.943	1.77
8	7	1.895	1.72
9	8	1.860	1.67
10	9	1.833	1.62
11	10	1.812	1.58
12	11	1.796	1.55
13	12	1.782	1.52
14	13	1.771	1.50
15	14	1.761	*1.48*
20	19	1.729	
30	29	1.699	
	∞	*1.645*	

5. CONCLUSIONS

According to the new concrete standards in the initial and continuous stage of production the concrete is being tested by the producer, and based on the test results of the continuously produced material issues the conformity statement for a characteristic value which is determined by a 70-30 % probability of acceptance-rejection. The reliability of the conformity statement is checked by the client through identification testing procedure. The result of the continuous and identification testing is significantly influenced by the evaluation method of the characteristic strength value, in which the value of the acceptance constant has the major role. From the point of the safety of our structures it could be appreciable if the risk of the producer and the client during the hand over procedure would be 50-50%.

6. ACKNOWLEDGEMENT

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7. THESAURUS

Portion of the strength values below the f_{ck} (underfalling portion) (Anteil der Festigkeitswerte unterhalb von f_{ck}). The portion of the material in the total material volume (lot) which does not satisfy the conformity (acceptance) criteria. It is the characteristic value of *x* statistical variable. Sign: *p*.

Lead value (Vorhaltemass). It is the product of the acceptance constant and the standard deviation (e.g. $\lambda_n \cdot s_n$, or $t_n \cdot s_n$), by subtracting it from the mean compressive strength value f_{cm} the result will be the characteristic value, in other words it is the difference between the characteristic and mean values of the compressive strength.

Acceptance constant (Annahmekonstant). A multiplicator, by which first multiplying the standard deviation of the compressive strength test results and afterwards subtracting this product from the mean value of the compressive strength results, we obtain the characteristic strength. It's sign is generally: λ_n, and in case of the *t*-distribution: t_n (*Student's*-factor), where *n* is the number of samples.

Acceptance characteristic (curve) (Annahmekennlinie). A curve, which shows the acceptance probability $A(p)$ in the function of the underfalling portion *p*. The function is: $p \cdot A(p)$ = constant (*Fig. 1*).

Acceptance probability (Annahmewahrscheinlichkeit). The probability of the acceptance of a concrete volume (lot) having *p* underfalling portion. Sign: $A(p)$.

Continuous production (stetige Herstellung). The production which follows initial production, lasting for at least 15 consequent compressive strength test result of without intermission under the same circumstances produced concrete, where the duration of the production time period is maximum 12 month before the last test is carried out.

Initial production (Ersthertstellung). The initial production is lasting for at least 35 consequent compressive strength test result of without intermission under the same circumstances produced concrete, where the duration of the production time period is longer than three monthes but maximum 12 monthes before the last test is carried out.

Quantile (Quantil). Mathematical statistical variable x_p belonging to the *p* underfalling portion, a threshold value, characteristic value (e.g. $f_{ck,test}$). The 5% quantile of the distribution may be determined by using the formula: $f_{ck,test} = \mu - 1.645 \cdot \sigma$.

Verification of conformity (conformity statement) (Konformitätsbestätigung). A statement – usually by involving a verifying body – about that the concrete conforms to standard requirement (e.g. a compressive strength class).

Declaration of conformity (Konformitätserklärung). A based on the continuous or initial testing a statement by the producer about that the concrete is conform to a standard requirement (e.g. compressive strength class).

Sample (Probe). A certain enough quantity of concrete separated for the production of one or more specimens for compressive (or other) tests representing the general quality of the material.

Normal distribution, Gaussian distribution (Normalverteilung). A distribution of the compressive strength test results which can be expressed by their statistical expected value (μ) and standard deviation (σ), of which the frequency distribution function is expressed as:

$$p(x) = \frac{1}{\sigma \cdot \sqrt{2\pi}} \cdot e^{-\frac{(x-\mu)^2}{2\sigma^2}}$$

Compressive strength (Druckfestigkeit). The highest stress level, expressed in N/mm^2, by which during the compressive strength test of concrete sample cylinder or cube fails.

Mean compressive strength (mittlere Druckfestigkeit). The average of the individual compressive strength test results.

Identity test of compressive strength (Identitätsprüfung für Druckfestigkeit). By involving an independent laboratory or by the laboratory of the client (mostly by involving the producer also) a test carried out during the hand over procedure for the determination if the concrete is conforming to the compressive strength class which was stated by the producer.

Continuous test of compressive strength (stetige Druckfestigkeitsprüfung). Concrete compressive strength test and evaluation of the results during the period of continuous production. During continuous testing the evaluation should be and may be done by the mean value and the standard deviation which was determined by the initial testing period. Continuous testing may be carried out by the producer or its trustee within intermediate quality control checking.

Characteristic value of compressive strength (charakteristischer Druckfestigkeitswert). That compressive strength value below which maximum 5% of the evaluated compressive strength test results of the concrete fall. It can be a prescribed (f_{ck}) or an empirical ($f_{ck,test}$) value.

Initial test of compressive strength (Erstprüfung für Druckfestigkeit). Concrete compressive strength test and evaluation of the results during the period of initial production. During initial testing the evaluation should be done by the mean value and the standard deviation must be given. Initial testing may be carried out by the producer or its trustee within intermediate quality control checking.

Standard deviation of compressive strength (Standardabweichung der Druckfestigkeit). It is the fluctuation of the individual compressive strength values, determined by the square root of the expected value of the square of the difference between the individual and mean value of the compressive strength. From the test results can be determined the empirical standard deviation (s_n) which is close to the value of (σ) the theoretical standard deviation.

Compressive strength class (Druckfestigkeitsklasse). The requirement for the compressive strength of concrete which is to be given by the prescribed characteristic compressive strength value of the 28 days old, cured under standard circumstances, 150 mm in diameter and 300 mm height cylinders ($f_{ck,cyl}$) and 150 mm edge length cubes ($f_{ck,cube}$).

Operating characteristic (OC curve) (Operationscharakteristik). The operation characteristics of the conformance criteria system. It is an operational characteristic curve showing the probability $L(p,n,c)$ of that the together evaluated *n* specimen just *c* or less are nonconform, in the function of *p*, which is the underfalling portion of the concrete. The value of *c* is a so called decisive number in the *n* test results it is the maximum number of results which may not be conforming, i.e. in our case $0.05 \cdot n$. To give the values of the *OC curve,* instead of the binominal distribution usually the more easily hadleable *Poisson* distribution is used (Felix et al., 1964):

$$L(p,n,c)=\sum_{x=0}^{c}\frac{(n\cdot p)^x}{x!}\cdot e^{-n\cdot p}$$

Poisson distribution (Poisson-Verteilung). It's density function is:

$$p(x)=\frac{(n\cdot p)^x}{x!}\cdot e^{-n\cdot p}$$

The *Poisson* distribution is giving a better and better approximation of the binominal distribution as n increases and p decreases.

Student's factor (Student-Koeffizient). The $N(0.1)$ distributed t-distribution's – belonging to one side 5% underlaying portion – $t_{95\%,f}$ statistical variable (the quantil of $p = 0.05$, it's threshold value).

t-distribution (Student's distribution) (t-Verteilung). A distribution, which is similar to the *Gaussian* normal distribution, and which is also a function of n, the sample number. It's density function is:

$$p(x)=\frac{1}{\sqrt{(n-1)\pi}}\cdot\frac{\Gamma\left(\frac{n}{2}\right)}{\Gamma\left(\frac{n-1}{2}\right)}\cdot\frac{1}{\left(\frac{x^2}{n-1}+1\right)^{\frac{n}{2}}}\quad,$$

where Γ is the sign of the gamma function. In our case the freedom of the t-distribution is: $n-1$. If $n\rightarrow\infty$, then the t-distribution tends to the *Gaussian* normal distribution (*Fig. 2*).

8. MOST IMPORTANT NOTATIONS

$A(p)$ acceptance probability of a concrete having p underfalling portion

C sign of normal density concrete

f freedom of *Student's* distribution

f_c compressive strength of concrete

f_{cd} compressive strength of concrete – design value

f_{ci} compressive strength of concrete – individual empirical value

f_{ck} compressive strength of concrete – prescribed characteristic value

f_{cm} compressive strength of concrete – prescribed mean value

$f_{cm,test}$ compressive strength of concrete – mean of the empirical values

$f_{c,cube}$ compressive strength of concrete – prescribed value for cubes of 150 mm edges, cured under water until the age of 28 days

$f_{c,cube,H}$ compressive strength of concrete – prescribed value for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days

$f_{ci,cube,test,H}$ compressive strength of concrete – individual empirical value for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days

$f_{ck,cube,H}$ compressive strength of concrete – prescribed characteristic value for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days

$f_{cm,cube,H}$ compressive strength of concrete – prescribed mean value for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days

$f_{cm,cube,test,H}$ compressive strength of concrete – mean of the empirical values for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days

$f_{c,cyl}$ compressive strength of concrete – prescribed value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

$f_{ci,cyl,test}$ compressive strength of concrete – individual empirical value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

$f_{ck,cyl}$ compressive strength of concrete – prescribed characteristic value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

$f_{ck,cyl,test}$ compressive strength of concrete – empirical characteristic value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

$f_{cm,cyl}$ compressive strength of concrete – prescribed mean value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

$f_{cm,cyl,test}$ compressive strength of concrete – mean of the empirical values for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

$L(p,n,c)$ the distribution function of *Poisson's* distribution

n the number of samples

p portion of the strength values below the f_{ck} (underfalling portion)

$p(x)$ statistical density function

$R_{cube,test}$ compressive strength of concrete – individual empirical value for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days according to MSZ 4720-2:1980 Hungarian standard

$R_{k,cube}$ compressive strength of concrete – prescribed characteristic value for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days according to MSZ 4720-2:1980 Hungarian standard

$R_{k,cube,test}$ compressive strength of concrete – empirical characteristic value for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days according to MSZ 4720-2:1980 Hungarian standard

$R_{m,cube,test}$ compressive strength of concrete – mean of the empirical values for cubes of 150 mm edges, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days according to MSZ 4720-2:1980 Hungarian standard.

$R_{k,cyl}$ compressive strength of concrete – prescribed characteristic value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 7 days and in laboratory circumstances until the age of 28 days according to MSZ 4720-2:1980 Hungarian standard

s_{min} standard deviation of compressive strength of concrete – required minimum value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

s_n standard deviation of compressive strength of concrete – empirical value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

t_n *Student's* factor

x mathematical statistical variable

α_{cc} durable strength factor

σ standard deviation of compressive strength of concrete – theoretical value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

σ_{min} standard deviation of compressive strength of concrete – required minimum value for cylinders of 150 mm in diameter and 300 mm in height, cured under water until the age of 28 days

σ_n standard deviation of compressive strength of concrete – with unknown probability theoretical value

γ_c safety factor for concrete

λ_n underfalling portion factor

μ probability of expected compressive strength of

9. REFERRED STANDARDS AND CODES

MSZ 4719:1982 „ Concrete (Betonok)" Hungarian standard

MSZ 4720-2:1980 „ The quality control of concrete. General properties (A beton minőségének ellenőrzése. Általános tulajdonságok ellenőrzése)" Hungarian standard

MSZ 4798-1:2004 „ Concrete part1. Technical requirements, fulfillment, production and conformity, and the Hungarian NAD for MSZ EN 206-1 (Beton. 1. rész: Műszaki feltételek, teljesítőképesség, készítés és megfelelőség, valamint az MSZ EN 206-1 alkalmazási feltételei Magyarországon)" Hungarian standard

MSZ 15022-1:1986 „Design of loadbearing building structures, Reinforced concrete structures (Építmények teherhordó szerkezeteinek erőtani tervezése. Vasbeton szerkezetek)" Hungarian standard

MSZ EN 206-1:2002 „Concrete part 1. Technical requirements, fulfilment, production and conformity (Beton. 1. rész: Műszaki feltételek, teljesítőképesség, készítés és megfelelőség)" Hungarian standard

MSZ EN 1992-1-1:2005 „Eurocode 2. Design of concrete structures. part 1-1. General and building rooles (Eurocode 2: Betonszerkezetek tervezése. 1-1. rész: Általános és az épületekre vonatkozó szabályok)"

MÉÁSZ ME-04.19:1995 „Production of concrete and reinforced concrete. 6th Topic, testing, quality control, quality certification. Code of practice (Beton és vasbeton készítése. 6. fejezet: Vizsgálat, minőség-ellenőrzés, minőségtanúsítás. Műszaki előírás)" Hungarian code of practice

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RESIDUAL COMPRESSIVE STRENGTH OF FIRE EXPOSED FIBRE REINFORCED CONCRETE


Éva Lublóy – Prof. György L. Balázs

Fires in tunnels indicated, that the spalling of concrete cover should be prevented because it increases accident danger and makes rescue difficult. The spalling probability of concrete cover can be decreased by the application of synthetic fibres. Using synthetic fibres yields a new possibility in the domain of the applicability of expanded clay aggregates. Due to increased danger of spalling of concrete cover, the lightweight aggregates are not recommended by the experts (Faust, 2003), nevertheless, here also, synthetic fibres could provide some possible solution. However, if synthetic fibres are applied change on the residual compressive strength of concrete can be observed. During our experiments, the residual compressive strength of concrete, as well as the deformation of concrete cover has been analysed. In the experiments concrete was exposed to heat and than cooled down to room temperature.

Keywords: fibre-reinforced concretes, lightweight concretes, high temperature, compressive strength

1. INTRODUCTION

In case of tunnel fires, in addition to the reduction of load bearing capacity, spalling of concrete cover makes further difficulties. It makes the process of the rescue and fire-fighting difficult. Fire cases in Mont Blanc (20 March 1999) and Gotthard (24 October 2001) tunnels included that the spalling of the concrete cover has to be avoided. In these cases the temperature exceeded 1000°C within a relatively short time. The concrete spalled in large pieces and that caused significant material damages. Spalling of concrete cover can be decreased by the application of synthetic fibres (Janson and Boström, 2004). However by the application of synthetic fibres the changing of the residual compressive strength of concrete to fire must also be considered. In our experiments we experimentally analysed the residual compressive strength of concrete and the deformation of the concrete surface after high temperatures.

2. APPLICATION FIELDS AND ADVANTAGES OF SYNTHETIC FIBRE REINFORCEMENT

Synthetic fibres have many advantageous properties based on their chemical composition. They are acid- and alkali-resistant, non-corrosive, non-conductive, nonmagnetic. In order to improve fire resistance, usually polypropylene fibres *(Table 1)* are used, because of their relatively low melting points.

Advantages of the application of the synthetic fibres can be best demonstrated in the improvement of the property of fresh concrete. Synthetic fibres can be effectively used in several fields of concrete production. Currently in Hungary many kinds of synthetic fibres are available in the construction industry *(Fig. 1)*.

Fig. 1: Various synthetic fibres (Kaposplast list of products)

Concrete, cement-mortar and lime-mortar can be reinforced with synthetic fibres. In order to reduce sensitivity for early cracking application of synthetic fibres could be effective in the production of slabs surfaces, prefabricated concrete elements, concrete structures of bridges, pools, industrial floors.

There are several possibilities for the mixing of synthetic

Table 1: Properties of some synthetic fibres (Kaposplast list of products, http://www.avers.hu

Fibre type	polypropylene (PP)	aramid	poliakrilnitril (PAN)
Density (g/cm^3)	0.90	1.38	1.15
Young's modulus (N/mm^2)	6000-18000	50000-150000	17000-20000
Melting point (°C)	160-170	250	-
Decomposition temperature (°C)	360	1600	> 1000°C
Alkali-resistance	++	+	+
Oxidations- resistance	+	+	+

fibres. Synthetic fibres can be added to the dry mix, to the half of the mixing water or to the wet concrete mix.

Formation of early cracks can be significantly reduced by using synthetic fibres, in addition to several other properties of concrete also improve. Due to fewer cracks concrete will become more resistant.

Applying appropriate concrete mixing technology, the application of synthetic fibres has the following advantages:

– stability of fresh concrete is increased
– if shotcrete technology is used, rebound is decreased
– degree of bleeding is decreased
– higher mortar thicknesses without sliding
– synthetic fibres reduce segregation of concrete by transport and casting
– less cracks develop during hardening, therefore the water tightness of concretes is increased, the hygroscopicity is decreased, thus the resistance to frost and to de-icing salt is increased. At the same time fatigue strength, sound-insulating and sound-absorption capacity of concrete are also increased.
– *Resistance to fire of concrete improves, because burning of the fibres helps the water content of concrete evaporate that decreases its inner stress*.

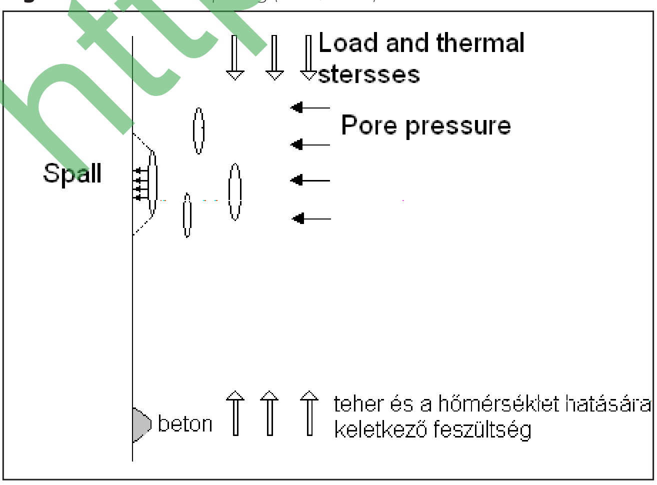
3. BEHAVIOUR OF SYNTHETIC FIBRES REINFORCED CONCRETES IN HIGH TEMPERATURE

Decrease of strength of concrete at high temperatures can many based on the following two factors: (a) transformation of concrete structure and (b) the spalling of concrete cover. Transformation of structure of concrete is caused by the chemical transformation of cement stone (break down of the portlandit around 450°C and the CSH-composite around 700°C), the volumetric increase of quartz gravel aggregate (573°C) (Mészáros, 1999).

Spalling of concrete surfaces may have two reasons: (1) internal vapour pressures (mainly for conventional concretes) and (2) overloading of concrete compressed zones (mainly for high strength concretes). The spalling mechanism of concrete cover can be seen in *Fig. 2*.

To avoid disasters, when tunnels are constructed it is important to prevent the possible spalling of concrete cover. In Austria, in compliance with valid technical regulations, it is mandatory to use synthetic fibres for construction of tunnels.

Fig. 2: Mechanism of spalling (Niels, 2005)



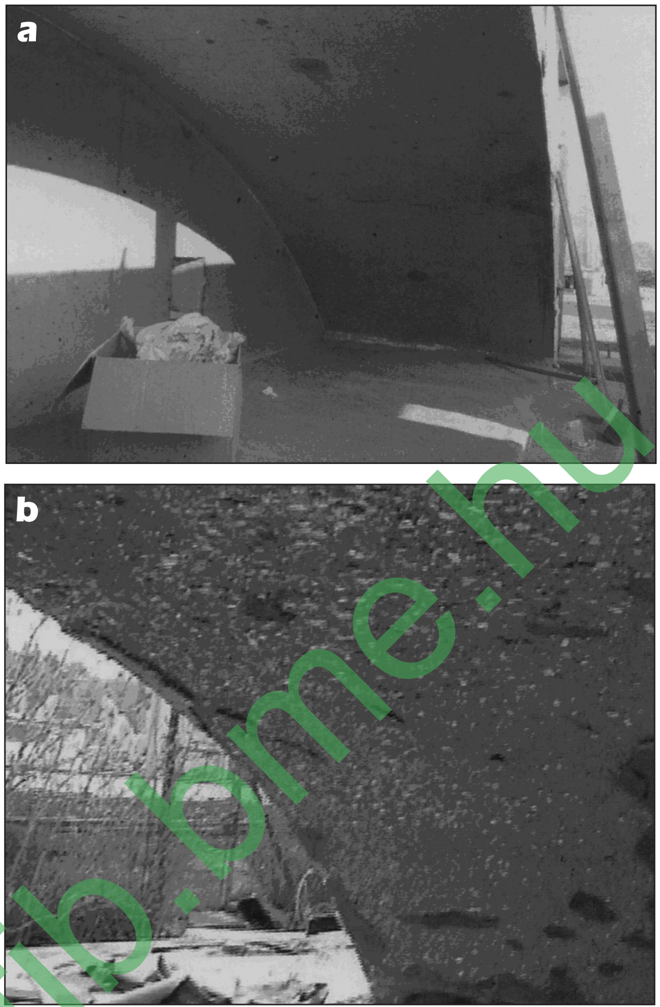


Fig. 3: Tunnel segments after exposure to 1200°C temperature (Mörth, Haberland, Horvath, Mayer, 2005)
a) without fibre reinforcement
b) with 2 kg/m³ polypropylene fibre reinforcement

A great number of experiments supported that the application of synthetic fibres considerably reduced the danger of spalling of concrete cover. Experiments with tunnel segments (length 11 m, height 2 m) carried out by Mörth, Haberland, Horvath and Mayer (2005) indicated, that the cover of the polypropylene fibre reinforced concrete did not spall *Fig. 3a and b)*.

In Switzerland the experiments of the BBT Trading & Consulting GmbH proved that in case of polypropylene fibre reinforcement the possibility of spalling of concrete cover can be decreased. *Fig. 4* shows that by rising the fibre reinforcement the area of the spalled concrete surface decreased.

Janson and Boström (2005) tested slender concrete columns of different concrete composition. If concrete has been reinforced with polypropylene fibres independent of its composition the area of spalled concrete cover essentially decreased.

In Austria a group of researchers had the same findings (Walter, Kari, Kutserle, Lindlbauer, 2005). They tested plates loaded in their planes. In case of conventional concrete the spalling of the concrete cover developed. However, no such phenomenon was experienced when slabs had been reinforced with polypropylene fibres *(Fig. 5 a and b)*.

Utilisation of synthetic fibres do not only reduce the probability of spalling of concrete cover layer, but also reduces the residual compressive strength.

A group of researchers (Horiguchi, 2005) experimentally proved on cylinders that the addition of synthetic or steel fibres has changed the value of the residual compressive strength.

Fig. 4: Behaviour of tunnel segments produced with different fibre reinforcement (http://www.b-b-t.ch/produkte/kunststoff3.htmle)

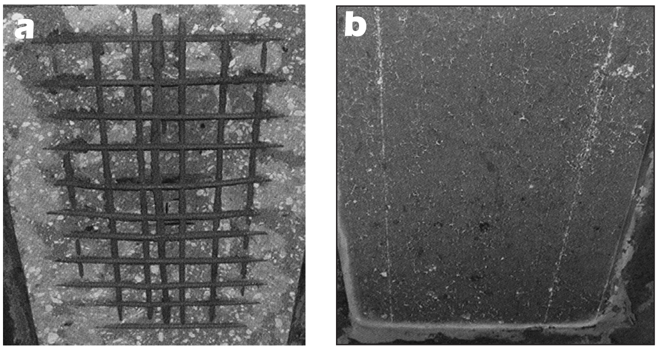


Fig. 5: Surface of the slabs after 2-hour fire exposure (Walter, Kari, Kutserle, Lindlbauer ,2005)
a) without fibre reinforcement
b) with synthetic fibre reinforcement

The experiments were carried out at 20°C, 200°C and 400°C. Cylinders (Φ=100 mm, l=200 mm) were heated by 10°C/minute rate, then kept for 1 hour at high temperature, and finally tested at room temperature (*Fig. 6*). The water-cement ratio during the experiment was 0.3 (with 583 kg/m³ cement).

– Mix A was prepared without fibres,
– Mix B with 0.5 V% polypropylene fibre reinforcement,
– Mix C with 0.5 V% steel fibre reinforcement,
– Mix D with 0.25 V% polypropylene 0.25 V% steel fibre reinforcement.

4. OUR EXPERIMENTS

4.1 Test parameters

The following test parameters were fixed:
1. cement (CEM I 42,5 N, 350 kg/m³)
2. water-cement ratio: 0.43

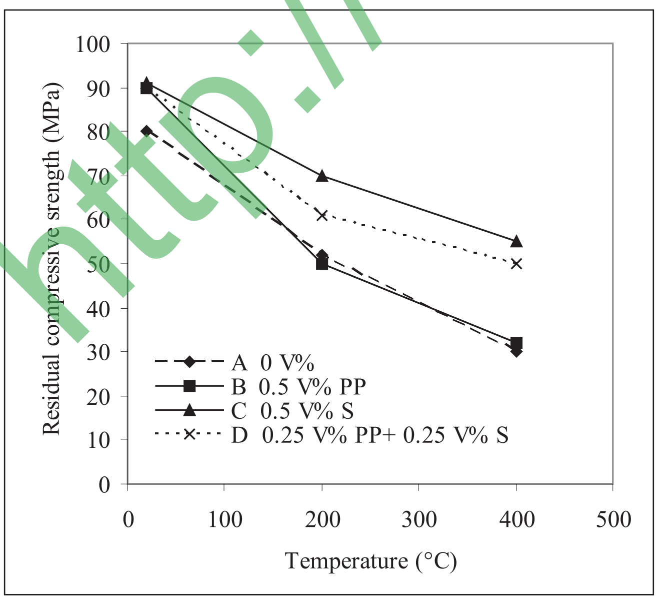


Fig. 6: Residual compressive strength of high strength concrete with or without fibres (Horiguchi, 2005)

3. aggregate to cement ratio: 5.6
4. flow value: 410-430 mm
5. small fraction of aggregate (0-4 mm) was river sand

Test variables were (always to 4-4 specimens):
1. temperature (20°C, 50°C, 150°C, 200°C, 300°C, 400°C, 500°C, 600°C, 800°C)
2. fibre type
   – without fibre 0 V %
   – mono fibre 8 V %
   – macro fibre 1 V %

Characteristics of the applied fibre types are in the *Table 2*.

3. aggregate
– quartz gravel (aggregate diameter 4/16) or
– expanded clay (aggregate diameter 4/8, ρ=830 kg/m³, maximum temperature by production 1200°C, water absorption capacity: 36 m%)

4.2 Test methods

Compressive strength tests have been carried out on cubes of 150 mm sides. Concrete composition and temperature (20°C, 50°C, 150°C, 200°C, 300°C, 400°C, 500°C, 600°C, 800°C) were the changing parameters of the experiment. Specimens were tested at room temperature after the heating process and a 2-hour exposure to temperature.

5. TEST RESULTS

5.1 Visual inspection

5.1.1 Concrete with quartz gravel aggregates

By applying small diameters mono fibres surface deformation was not observed at all up to 800°C (*Fig. 7*). By the concrete

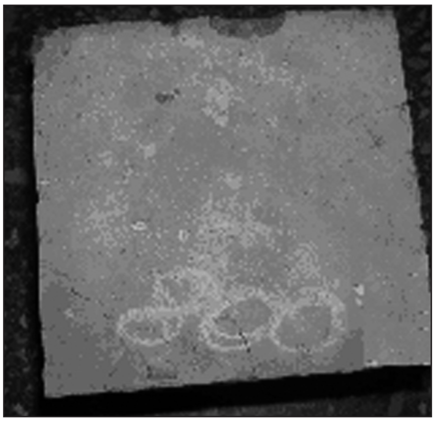


Fig. 7: Mono-fibre reinforced concrete (800°C temperature load)

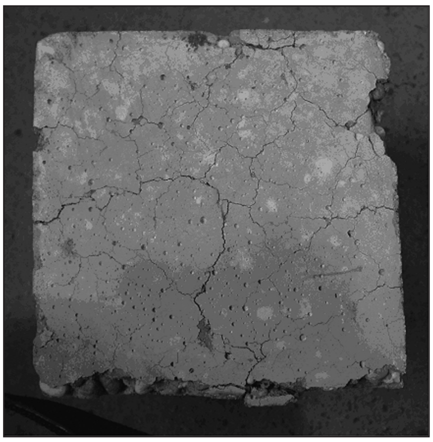


Fig. 8: Concrete without fibre reinforcement (800°C temperature load)

without fibre we have observed surface cracks by heating up to 800°C (*Fig. 8*).

At 200°C and 300°C test parameters macro fibres have bean already melted. Macro fibres close to the surface flowed to the surface then burnt by heaving colour on the surface (*Fig. 9 and Fig. 10*). In some places holes could be observed. They were probably perpendicular to the concrete surface and burnt off in this position. Signs of burning could be seen on the concrete cover. This signs could be avoided in case of other fibres.

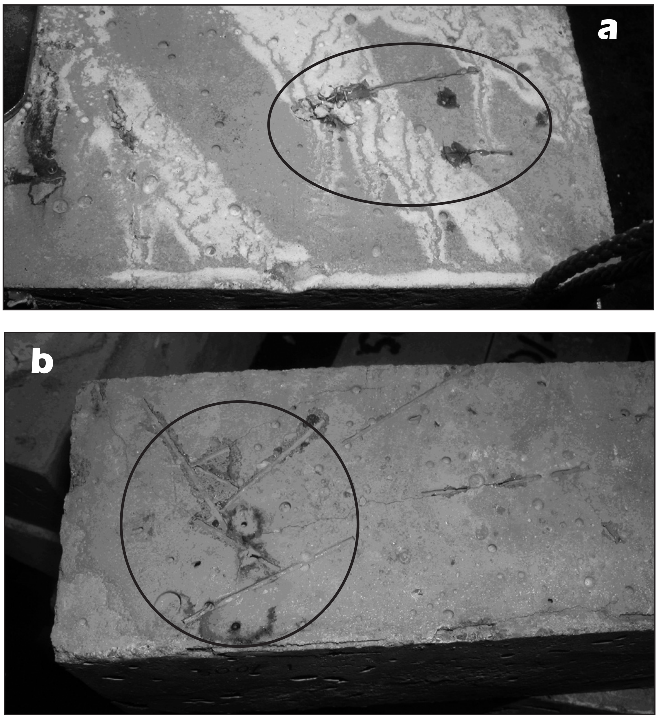


Fig. 9: Macro-fibre reinforced concrete
a) exposed to 200°C b)exposed to 500°C

5.1.2 Concrete with expanded clay aggregates

When expanded clay aggregate has been applied instead of quartz aggregate, the corners of the specimens peeled off as a consequence of 800°C heat exposure. On the peeled off surfaces the cracks were passing also through the aggregate (*Fig. 10*).

When the specimens were produced by using synthetic fibres, damage could not be discovered on the concrete surface with naked eyes.

Fig. 10: Concrete with expanded clay aggregate without fibres after heating to 800°C

5.2. Compressive strength

5.2.1. Concrete with quartz gravel aggregate

Measured values of compressive strength as a function of temperature are presented in *Figs. 11 to 14*. (Compressive strength measurements were carried out on specimens cooled down to room temperature. One point indicates the average of 4 measurements.)

1. Overall tendencies (*Fig. 14)* of strength reduction by increasing temperatures were similar (1) without fibres, (2) with plastic mono fibres and (3) with plastic macro fibres.
2. The residual compressive strengths at our maximal temperature for all of the 3 mixes were between 20% to 30 % of the strength at room temperature.
3. Most considerable reduction of compressive strength took place between 400 and 800°C.
4. Between 20°C to 400°C the values of compressive strength were for the 3 mixes different (compare Figs.11-13).

   The fibres did not burn off from the concrete on 200°C

Table 2: Characteristics of the applied fibre types

Characteristic		
Fibre	macro fibre*	mono fibre**
Material	polypropylene	polypropylene
Length (mm)	40	18
Diameter (mm)	1.1	0.032
Density (g/cm^3)	0,91	0,91
Melting point (°C)	171	160
Decomposition temperature (°C)	360	365
Acid resistance	high	high
Alkali resistance	100 %	100 %

* POLITON V40, Kaposplast Ltd.

** FIBRIN 1832, Kaposplast Ltd.

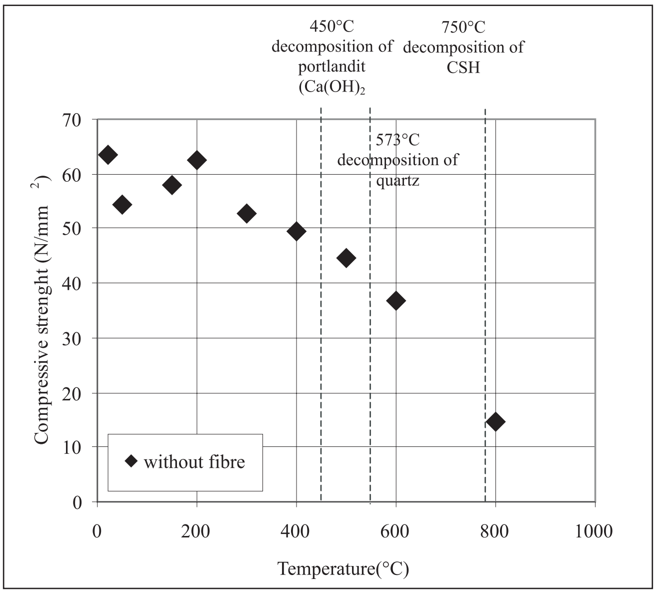


Fig. 11: Development of the compressive strength by concrete without fibre in function of temperature (average of 4 measurements)

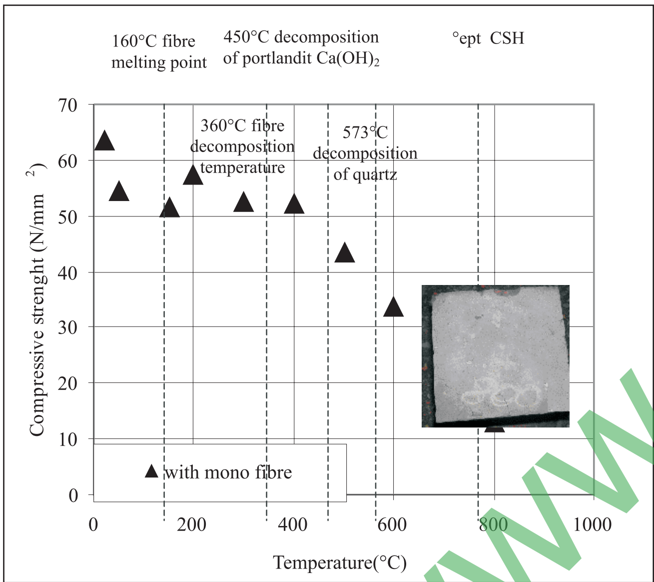


Fig. 12: Development of the compressive strength by concrete with mono fibre in function of temperature (average of 4 measurements)

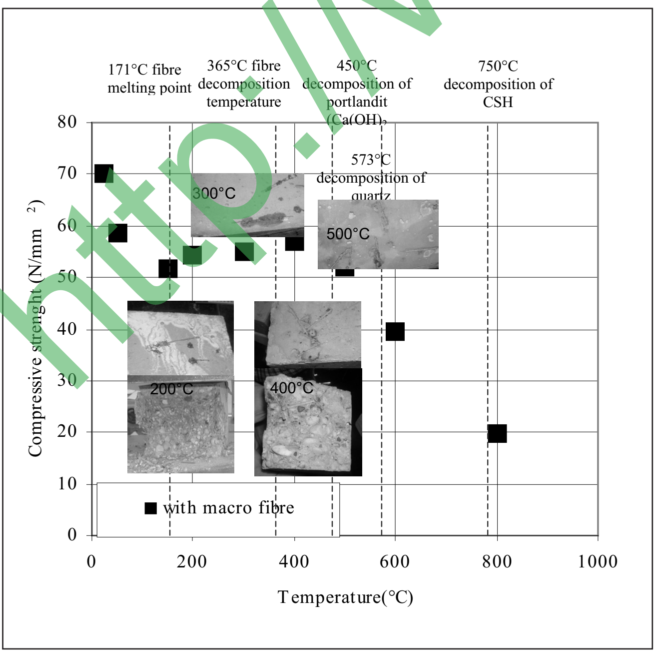


Fig. 13: Development of the compressive strength by concrete with macro fibre in function of temperature (average of 4 measurements)

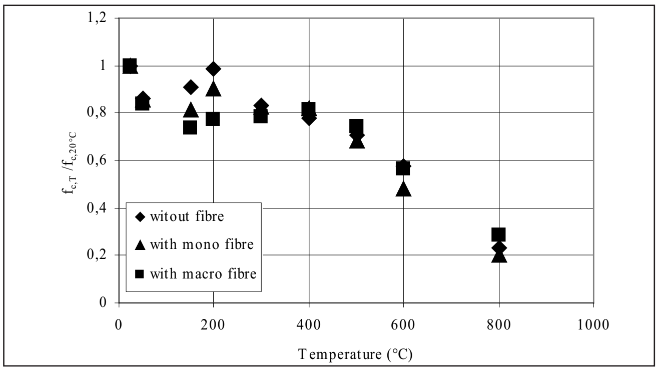


Fig. 14: Percentage values of the compressive strength in function of temperature (average of 4 measurements)

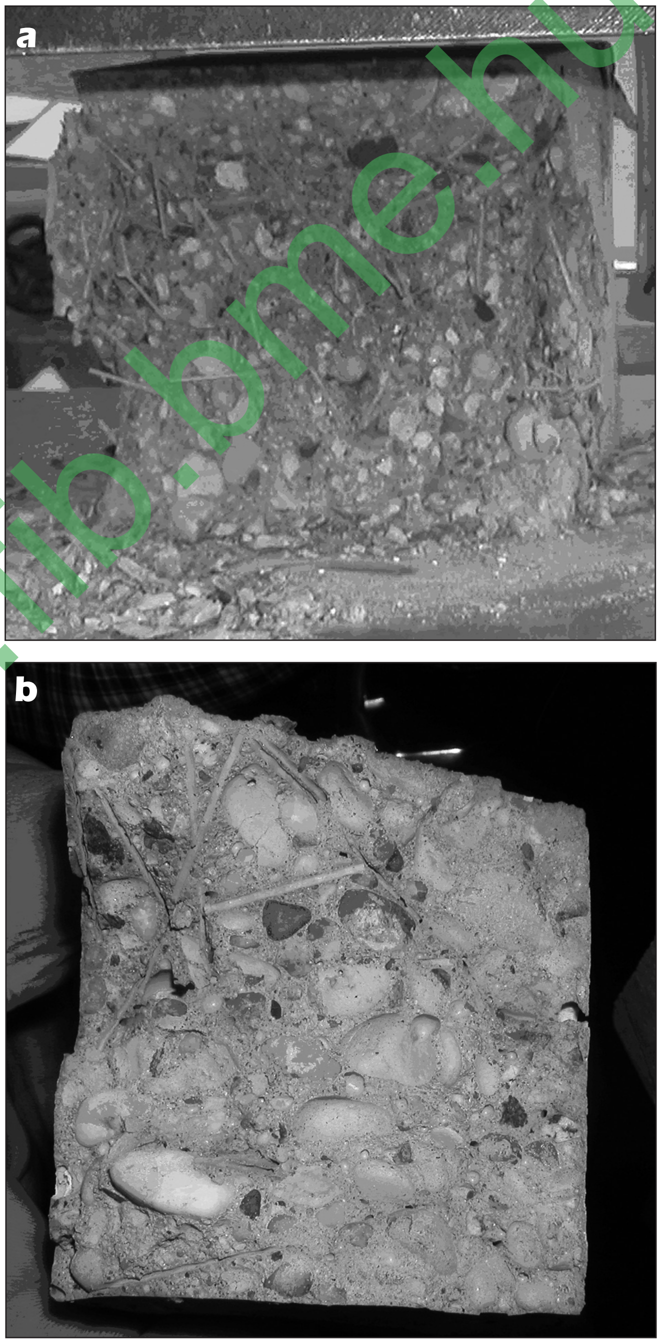


Fig. 15: Failure surface of macro fibres reinforced concrete
a) heated up to 200°C, cooled down to 20°C and finally subjected to compressive test
b) heated up to 400°C cooled down to 20°C and finally subjected to compressive test

after the fracture of macro fibre concrete (*Fig. 15 a)* it can be cloncluded. The fibres can be found inside concrete and on its surface. Above 400°C *(Fig. 15 b)* the fibres burnt off and only their place can be discovered on the concrete cover.

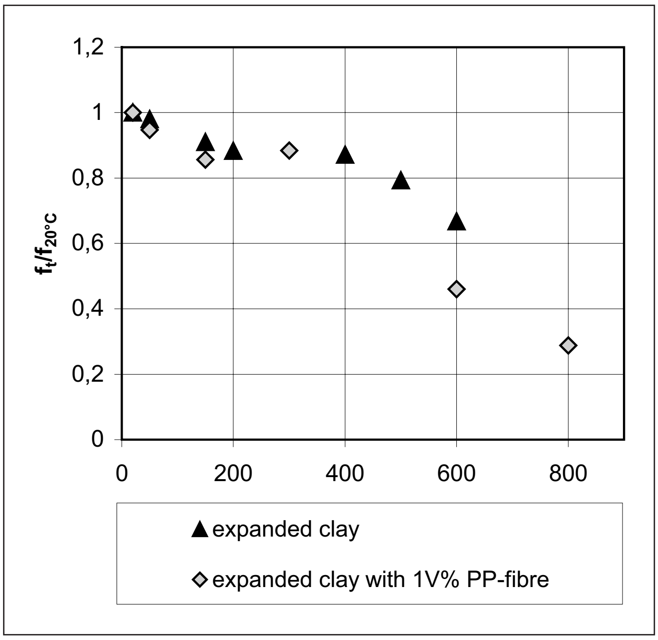


Fig. 16: Change of residual compressive strength after exposing the concrete with expanded clay aggregate to high temperature, measured by room temperature (20°C)

5.2.2. Concrete with expanded clay aggregate

Residual compressive strength of concrete with expanded clay aggregate under temperature increase was higher compared to that of conventional concrete (*Fig. 16*). However, the corner of the specimen spalled between 600°C to 800°C without polypropylene fibres. It did not occur with 1V% polypropylene fibres.

6. CONCLUSION

An experimentaly study was carried out at the Budapest University of Technology and Economics, Department of Construction Materials and Engineering Geology. In our tests concretes with two different aggregates and with two different polypropylene fibre types have been tested in temperature range from 20°C to 800°C. During our experiments, the residual compressive strength of concrete, as well as the deformation of concrete cover has been analysed. Specimens were exposed to high temperature (20°C, 50°C, 150°C, 200°C, 300°C, 400°C, 500°C, 600°C, 800°C) and than cooled down to room temperature, finally compressivesternght was tested.

Spalling of the concrete surface shoud be be prevented. The spalling of concrete cover can be reduced by applying synthetic fibres.

In case of concrete with macro fibre, the burning of the fibre frame can be seen already above 200 °C.

During the test, in case of mono-fibre reinforced concrete, both with quartz gravel or expanded clay aggregate content, hardly any cracks could be observed when the state subsequent to 800°C temperature exposure and then cooling down to room temperature has been applied.

Overall tendencies (*Fig. 14)* of strength reduction by increasing temperatures were similar (1) without fibres, (2) with plastic mono fibres and (3) with plastic macro fibres.

Synthetic fibres, provide good application possibilities in case of lightweight concrete with higher surface spalling inclination. However, when synthetic fibres are applied, change in the residual compressive strength of concrete has to be monitored.

7. ACKNOWLEDGEMENTS

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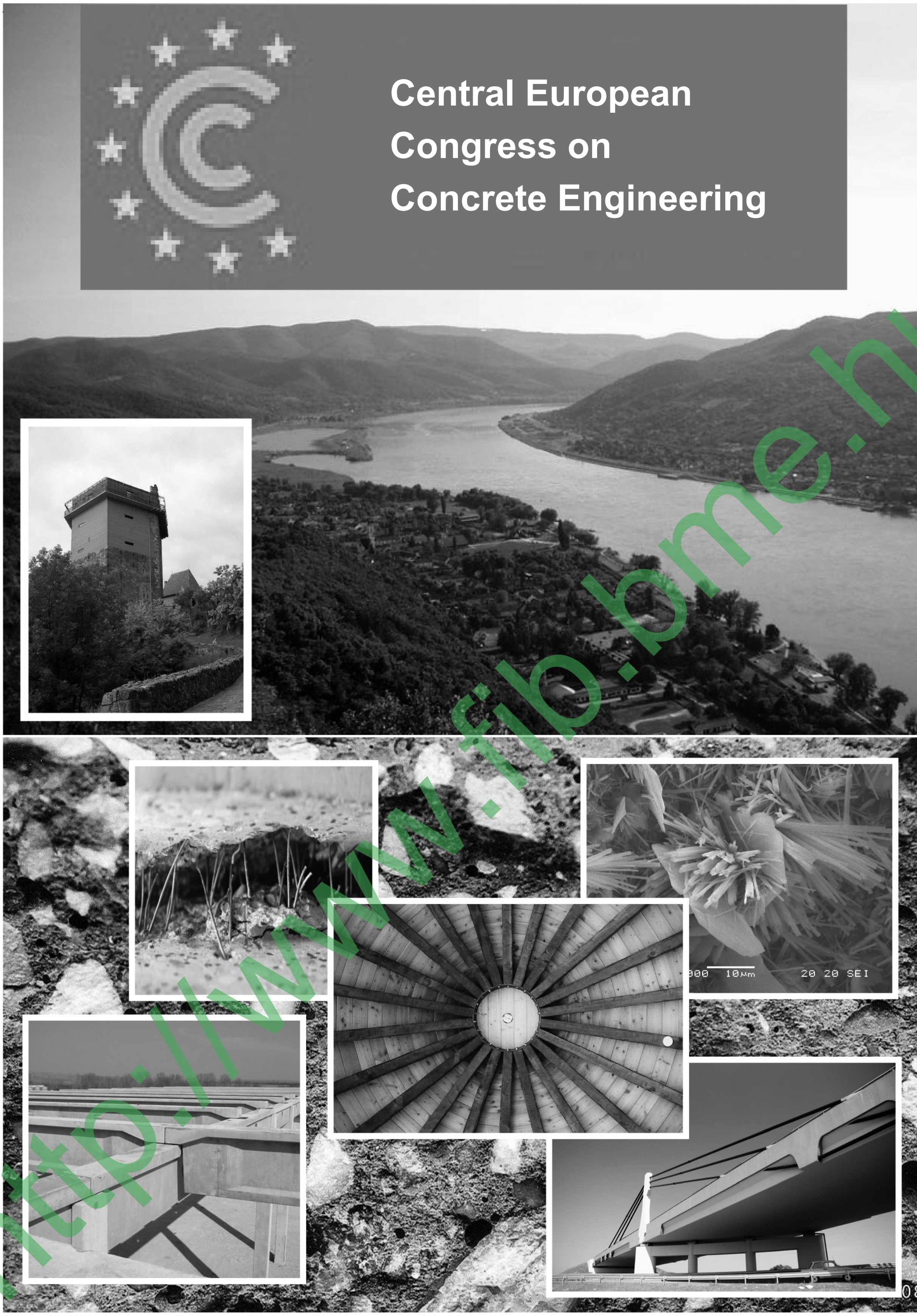

Visegrád, Hungary
17-18 September 2007

INNOVATIVE MATERIALS AND TECHNOLOGIES FOR CONCRETE STRUCTURES

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INVITATION and OBJECTIVES

It is a great honour for me to invite you to the 3rd Central European Congress on Concrete Engineering in Visegrád, Hungary. The Congress focuses on Innovative Materials and Technologies for Concrete Structures. Concrete is an ever developing construction material. There is a continuous development on material properties, constructability, economy as well as aesthetics.

The Congress intends to overview properties of new types of concrete (including all constituents) and reinforcements as well as their possible applications which already exist or can exist in the future.

The Congress will be organized in a beautiful ambient provided by the picturesque Danube Bend.

The host organisation of the Congress is the Hungarian Group of *fib*. Co-organizers of the Congress are the Hungarian Concrete Association and the Hungarian Precast Concrete Association.

We have a pleasure to invite representatives of clients, designers, contractors, academics and students to take part at this regional European event, which will give excellent social and technical conditions for exchange of experience in the field of concrete engineering. We are looking forward to meeting you in Visegrád.

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

ABOUT CCC

The four founding countries – Austria, Croatia, the Czech Republic and Hungary – decided in 2004 to collaborate more closely and to launch as their first joint project the Central European Congresses on Concrete Engineering (CCC Congresses) as a forum for an annual cross-border exchange of experience among principals, authorities, contractors, design engineers and academics in the field of construction materials and technology, concrete structures and civil engineering, as well as applied research and development. The 1st CCC Congress in Graz (Austria) 2005 was devoted to *Fibre Reinforced Concrete in Practice*; the 2nd Congress in Hradec Kralove (Czech Republic) 2006 had the main topic *Concrete Structures for Traffic Network*. The 3rd Congress in Visegrád (Hungary) 2007 focuses on *Innovative Materials and Technologies for Concrete Structures.*

CONGRESS TOPICS

Contributions focused on the following topics are invited.

TOPIC 1: TAILORED PROPERITIES OF CONCRETE

Environmentally compatible cements. New types of Aggregates. High performance admixtures. High strength and high performance concretes. Fibre reinforced concrete. Lightweight concrete. Green concrete. Applications.

TOPIC 2: ADVANCED REINFORCING AND PRESTRESSING MATERIALS AND TECHNOLOGIES

Metallic and non-metallic reinforcements. Internal and external reinforcements. Applications.

TOPIC 3: ADVANCED PRODUCTION AND CONSTRUCTION TECHNOLOGIES

Concrete structures meeting high requirements. Prefabrication. Design aspects. Applications.

IMPORTANT DATES

30 June 2007 Submission of manuscripts

Accepted contributions will be either presented orally or in the Poster Session. Both oral and poster presentations will be published in the Congress Proceedings. The Poster Session will be continuously open during the Congress.

OFFICIAL LANGUAGE

The official language of the Congress is English. In addition, simultaneous translation into Hungarian and other languages can be organized.

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Tel: +36-1-463 4068 Fax: +36-1-463 3450
e-mail: ccc2007@eik.bme.hu
Symposium website: www.fib.bme.hu/ccc2007

EXHIBITION

A technical/commercial exhibition will be organized during the Congress to demonstrate new materials, products, technologies and services. The exhibition will be directly next to the Session rooms. You are kindly asked to indicate your interest to the organizers to make part of the exhibition.

SPONSORSHIP OPPORTUNITIES

Sponsorship of the Congress is welcome. Appreciation of sponsorship will be given several ways during the Congress.

TECHNICAL EXCURSIONS

Technical excursion will be organized to the new Danube Bride on the North section of the M0 motorway around Budapest under construction.

CONGRESS BANQUET

As a part of the Congress Dinner knight's tournament and royal fest will be presented in medieval atmosphere.

ACCOMPANYING PERSON'S PROGRAMME

Accompanying persons are warmly welcome to attend the Congress. Special programme will be organized for them.

VISEGRÁD

Visegrád is situated in the picturesque Danube Bend 50 km North to Budapest. Visegrád was a royal residence of Hungarian Kings in early renaissance. Visegrád is not only of historical interest but offers also natural beauty. Visegrád is best approachable by car through Budapest, Vienna or Bratislava.

REGISTRATION for 3rd CCC CONGRESS VISEGRÁD 2007

"Innovative materials and technologies for concrete structures"

17–18 September 2007, Visegrád, Hungary

fib CEB-FIP Hungary

Please type or fill in BLOCK LETTERS *and send via e-mail or via fax. Please use separate form per person.*

Please mark the status of your registration:

☐ *Member of CCC Association* ☐ *Non member* ☐ *Student, PhD student* ☐ *Accompanying person*

Participation in events

☐ Welcome drink – *evening 16 Sept. 2007*

☐ Congress (17 Sept. – 18 Sept. 2007) - *Opening 9.00 o'clock*

☐ Congress Banquet – *evening 17 Sept. 2007*

☐ Technical Excursion – *afternoon 18 Sept. 2007*

☐ Accompanying person's programme

Name, affiliation and contact details

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Congress Secretariat

"The 3rd Central European Congress on Concrete Engineering" Secretariat

Hungarian Group of ***fib***

c/o Budapest University of Technology and Economics

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Tel: +36-1-463-4068 Fax: +36-1-463-3450

e-mail: ccc2007@eik.bme.hu ; www.fib.bme.hu/ccc2007

Congress registration fees (incl. 20% VAT)

Status	*by 30 June 2007*	*after 30 June 2007*
Member of CCC Association	300 €	400 €
Non member	350 €	450 €
Student, PhD Student	30 €	30 €
Accompanying	60 €	60 €
Congress Banquet	50 €	50 €
Technical Excursion	10 €	10 €

It is requested that at least one author of every paper or poster registers (and transfers the reg. fee) **by 30 June 2007.**

Registration fee includes participation in Opening and Closing ceremonies, participation in Technical Sessions, one copy of the Symposium Proceedings (for delegates), coffees and refreshments in coffee breaks, lunches during the Congress, participation in Welcome.

Accompanying person's programme.

Indicate the lunches (included in registration fee) which you intend to take: 17 Sept ☐ 18 Sept ☐

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☐ **Bank transfer** **VOLKSBANK Hungary** Rakoczi str. 1-3. H-1088 Budapest, **IBAN**: HU 22 14100457-85520449-02000003, **Account No.**: 85520449-02000003, **Swift**: MAVO HUHB, **Beneficiary name:** Hungarian Group of ***fib***. Transfer has to carry the purpose Congress "**CCC2007 Visegrad**" and your name. Any extra costs occurred in bank transfer is at charge of the participant.

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There is a block reservation in the Congress Venue, Thermal Hotel Visegrád for Congress participants. Every participant is kindly asked to make his or her room reservation directly by the hotel using the **keyword: CCC2007**. The block reservation is valid till **15 August 2007**. After this date rooms can be reserved on availability. Internet address of the hotel: ***www.thv.hu*** and e-mail address of the hotel: ***info@thv.hu,*** fax: ***36-26-801-914***

EUR/pers./night	Forest-side room	Danube-side room
Double Room	70	76
Double Room for single use	95	101
Suite for single or double use	195	195

Conference room prices include: accommodation for 1 night with rich buffet breakfast, unlimited use of the hotel's wellness centre and bath: indoor and outdoor fun bath, whirlpool, thermal pools, outdoor swimming pool with sundeck, tepidarium (steam bath, sauna, aroma-bath, fun-showers, relax room), fitness room, bathrobe use, use of the parking lot, local tax, VAT.

Cancellations

All cancellations must be received in writing. Full refund reduced by 10% handling if cancellation is made before **1 August 2007**. No refund if cancellation is made after **1 August 2007**.

Date: .... Signature: ....