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CONCRETE STRUCTURES

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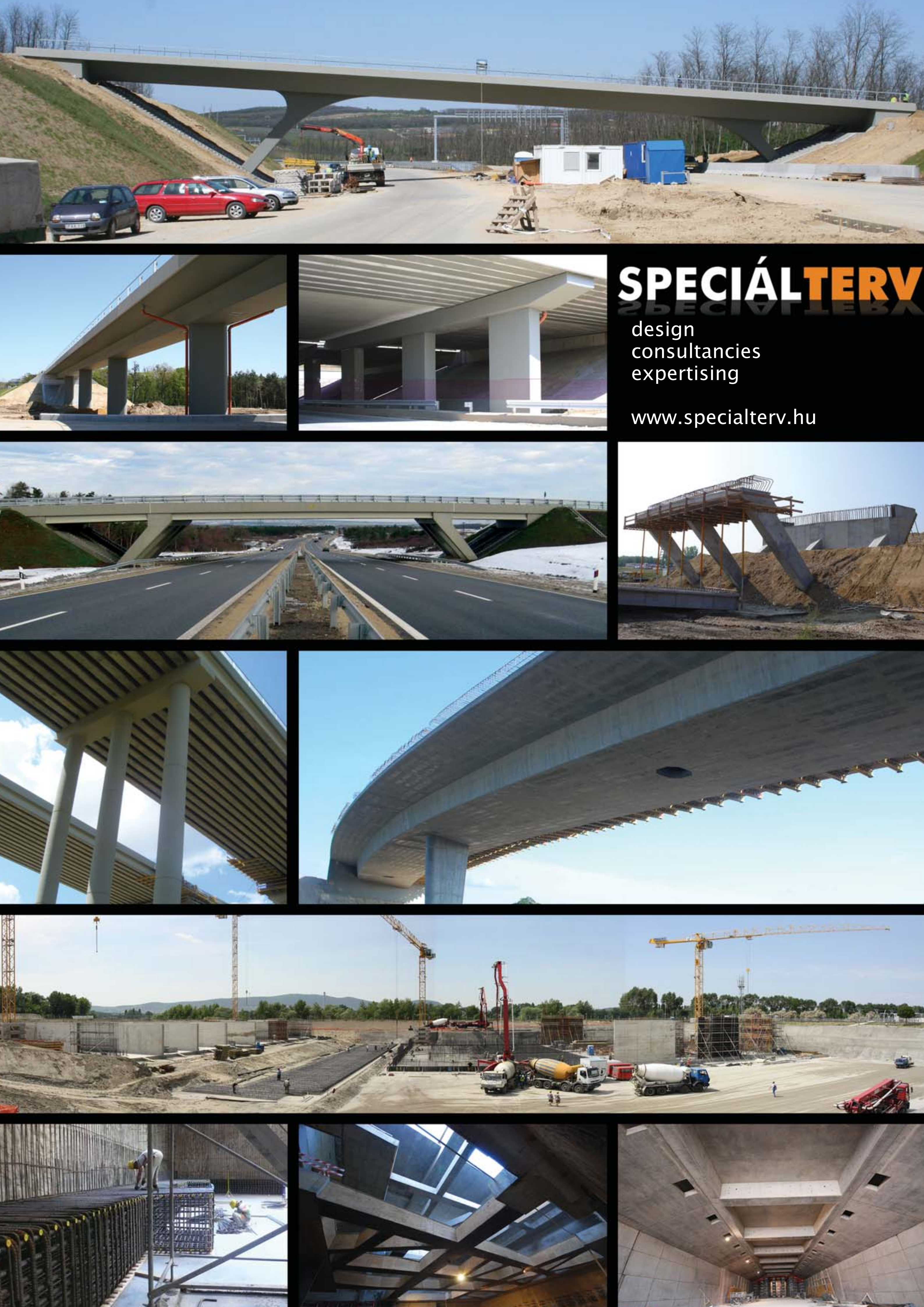
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SWEDES AND HUNGARIANS IN THE WHIRLWIND OF TIME – FROM HISTORY TO CONCRETE

Géza Tassi – György L. Balázs

It may be said that a tradition of presenting a leading article on the relations between the organizing country of the yearly international event of fib and Hungary has been created by our journal. The fib Symposium Stockholm 2012 provides an opportunity to overview the close connections between our homeland and the host country. We hope that this symposium improves the friendship and ties between specialists of our two countries and between all other member groups.

Keywords: Swedes and Hungarians, history, art and science, industry, concrete

1. INTRODUCTION

Despite the geographical distance, there are several links between historical events in Sweden and in Hungary. The connections in science and arts are worthy of notice as are connections in industry and commerce and particularly cooperation in the field of concrete technology over the last five to six decades.

Let our first sentences reflect on engagements between Hungarians with Swedes. Both historical and cultural connections between Sweden and Hungary are disconnected and sporadic due to the basically different ethnogeny of the two people and the relatively great distances between the lands in which they have settled. Possibly Hungarian children first hear of Swedes ("svédek" in Hungarian) listening to news on international sporting events. The country of Sweden ("*Svéd*ország") becomes known to schoolchildren most certainly when learning geography and history.

The languages of the two folks are very different and for lack of essential connections there are however a few words occurring in both languages: "ombudsman" (parliamentary commissioner of citizen rights) and "skanzen" (open air museum). Hungarians have borrowed both from the Swedes (Lakó, 1980). There are also some adjectival constructions in the Hungarian language that indicate either a Swedish origin or at least some Swedish relationship, such as "*svéd*acél" (Swedish steel), "svédtorna" (Swedish gymnastics), "*svéd*csavar" (reverse screw or Swedish throw) in water polo, "*svéd*kulcs" (crescent wrench - tool), "*svéd*toll" (a special adjustable ruling pen), "*svéd*asztal" (Smorgasbord), and in English, the Swedish word "smörgåsbord" is used.

The "*svéd*gyufa" (safety match) really deserves a special mention (see Point 3.1) because of its Hungarian origin.

Strange as it may be, the name "Swede" (in Hungarian: "*Svéd*") as a family name does occur, though rather seldom, as in the case of the world famous Hungarian baritone singer, Sándor *Svéd* (1906-1979), or the distinguished Hungarian born professor of engineering at the University of Adelaide (Australia), George (György) *Svéd* (1910-1994). In the Budapest phone directory there are presently only nine subscribers named "*Svéd*".

2. LINKS IN SWEDISH-HUNGARIAN HISTORY

2.1 First Contacts

After such an introduction it may be considered to be a hopeless task to write something on the subject, however, the facts are contradictory to all negative expectations.

Archaeological findings in the territory of Hungary have revealed characteristic types of swords called Viking swords dating from the early settlement of the Hungarian people in the Carpathian basin, thus demonstrating the early connection between Scandinavia and Hungary (*Fig.1*). Beautiful examples of this type of sword are in the Statens Historika Museum in Stockholm (*Fig. 2*), matching exactly to the then contemporary sword of the first Hungarian King, St. Stephen (István) (969-1038), of which an illustration is presented in this periodical (Tassi, Balázs. 2011). Another definite evidence of the 9th century's connections is the famous Viking lance found in the territory of Budapest.

The Vikings, being the forefathers of the Swedes, were an adventurous, warlike people having covered long distances both to the East and to the West. On their voyages eastwards they came into contact with the Slavs. One of their leaders

Fig. 1: Viking sword and lance findings, Budapest

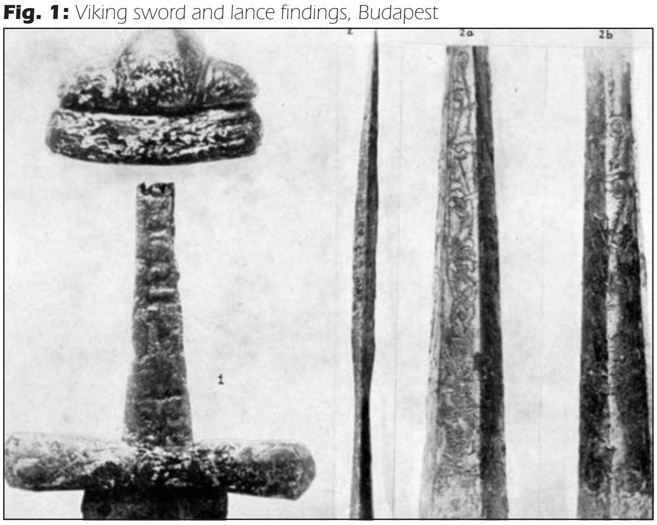
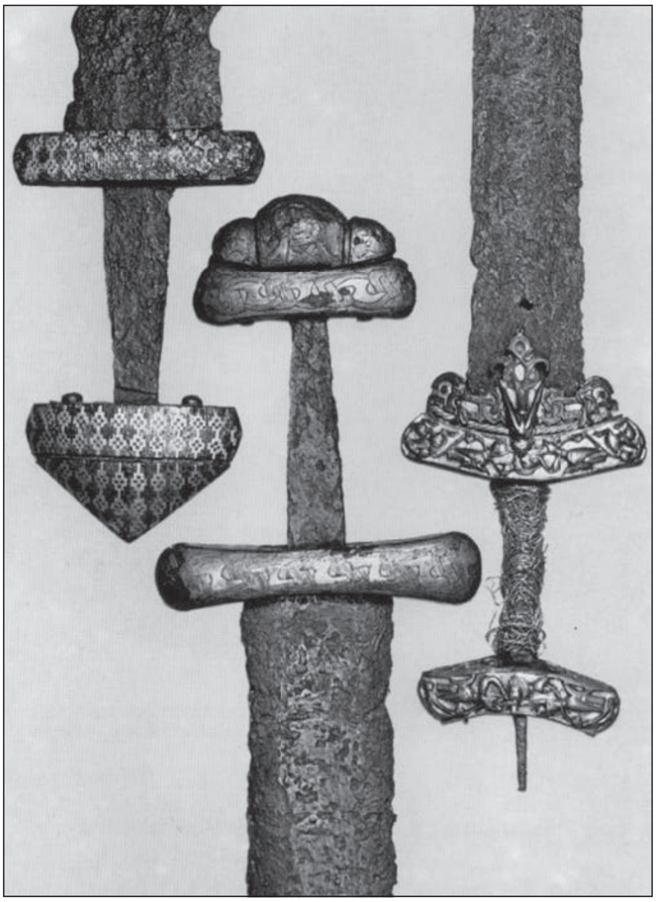


Fig. 2: Viking swords, Statens Historika Museum, Stockholm



called Rjurik (~830-879) (Scandinavian Hrøikr) founded the first Russian principality (Avella-Widhalm, 2003). The Vikings also reached the land surrounding the Black Sea, where Hungarian tribes had been living for some time prior to the conquest of their present land (896 A.D.). Influences common to both must surely have stamped the ornamental arts of both people, characterised by sophisticated foliated scrolls (*Fig. 3* and *4*) (László, 1970). Though considered to be of different origins, the runic writing of the ancestors of the Swedes is formally very similar to the old Székely runic writing in Hungary.

Fig. 3: Bag plate, Hungarian National Museum, Budapest

Fig. 4: Ivory box from the South of Scandinavia, Bavarian National Museum, Munich

2.2 Early Connections

The late 9th and the early 10th century is a very significant era in the life of both the Swedes and Hungarians. Olof III Skötkonung (~980-1022) ascended the throne of Sweden as the son of Erik VI Segersäll (~932-994) and the third king of the Yngling dynasty. He converted to Christianity and founded the first episcopate in his land.

A parallel with events mentioned above was the first Hungarian King, St. Stephen, who converted the pagan Hungarian people to Christianity and furnished the land with the first Hungarian coins. Interestingly the coins of early Hungary were highly valued and accordingly widely accepted. Though not in large quantities, their distribution is demonstrated in Swedish archaeological findings dating back to the time of the Hungarian King St. Stephen and until the reign of Anjou Louis the Great (Nagy Lajos) (14th century). It is noteworthy that the largest part of the Hungarian coins found in Sweden were minted under the rule of King St. Stephen (66 pieces) and of King Andrew (András) I (1013-1060), both belonging to the first Hungarian royal dynasty, the Árpádian house (Jonsson, 2001-2002).

There was indeed even a personal connection between the first Hungarian royal dynasty and its counterpart in Sweden, that being the wife of the fourth Hungarian King, Andrew I. His wife was Anastasia, the daughter of the Russian (from Kiev) Grand Duke Yaroslav I (~979-1054) and the granddaughter of the above mentioned Swedish King Olof III Skötkonung (Wertner, 1892).

There are other connections between early Hungary and Scandinavia. A well-known fact is that Scandinavian warriors, called varangians, served alongside the first Russian princes and the Byzantine Empire with which Hungary maintained a rather long and versatile intercourse.

Having long lived in the Byzantine Court with the waning possibility of becoming an emperor there, the Hungarian King Béla III (~1148-1196) summoned the above mentioned varangian warriors (Greek*: varangoi*) to his Hungarian court. He allowed them to settle in the countryside, where their name is preserved in the Hungarian place-name *Várong* (Tolna County).

The Middle Ages of Sweden cannot be examined without reflecting on the person and works of the great saint of the North, St. Brigitta (Birgitta Birgersdotter) (1303-1373). St Brigitta was a Swedish noblewoman by birth and marriage, and after the death of her husband she became known as a typical medieval religious visionary. Christ revealed himself to her in a vision in which He laid down the rules for a new Order which she was to find in Vadstena. Brigitta's first step was to enlist the support of her kinsman, King Magnus IV Eriksson (1316-1374). In 1346 the king and his consort Blanche of Namur (1316-1363) bestowed on her the royal estate of Vadstena, there to establish a nunnery. Thereafter Brigitta went to Rome to obtain the Papal sanction for her intention. This was finally granted to her in 1367 by Urban V (1309-1370). At the age of 70 Brigitta commenced a pilgrimage to the Holy Land, however, she died in Cyprus in 1373. Her body was brought back home in order to be buried there. Her tomb at Vadstena came to be a much frequented place of religious honour and pilgrimage. Brigitta put down in writing her religious visions which became not only valuable treasures of early Swedish literature but they soon also became widely disseminated in translated form. Accordingly, they were welcome and regularly read in Hungarian monasteries of the following centuries (Andersson, s. a.).

2.3 Incipit Reformation

In the following couple of centuries both Sweden and Hungary existed independently without significant or only loose connections. A prelude of the reformation was the Council of Basle (1431-49) presided by the Hungarian King and head of the Holy Roman Empire, Sigismund (Zsigmond) of Luxemburg (1368-1437), and Sweden was represented rather fiercely by Bishop Nikolaus Ragvaldi (Nils Ragvaldsson) of Växjö (1380-1448). He and the first two rulers of the Union of Kalmar, Queen Margareta I (1353-1412) and King Erik XIII of Pomerania (~1381-1459) were first cousins of the Hungarian King Sigismund. Queen Margareta contributed greatly to the victory of the reformation in Sweden, severing herself finally from Rome in 1530. The last Catholic Archbishop of Sweden, Johannes Magnus (Johan Månsson) (1488-1544) retired to Italy, where he composed his famous history of the Kings of the Goths and the Svear with the title: *Historia de omnibus Gothorum Sueonumque regibus*. At the same time Laurentius Petri (1499-1573) was enthroned as Sweden's first Protestant Archbishop. (Avella-Widhalm, 2003, Andersson, s. a.). As a consequence of these events the so called "Gustav Vasa's Bible" was published in 1541, thus laying the foundation for the "Earlier New Swedish" language. The translators had shaken off some of the Danish influences of the late Middle Ages and they endowed the language with a new firmness and clarity.

Plagued by the Turkish occupation Hungary also responded similarly in this respect. The first and complete Bible translated by Gáspár Károli (~1529-1591) into Hungarian was published in 1590 after some fifty years, contributing thus to the development of literary language. Some decades before these substantial decisions, King Sten I Sture (1440-1503) established the first Swedish university in Uppsala. Not only the university but also its library is very important and famous. Among many old prints of Hungarian origin resides an exceptional item in its collection, a manuscript older than one and a half thousand years called Codex Argenteus. It is the only existing copy of the Bible translated by the bishop Wulfila (~310-383 A.D.) into the Gothic language (Ballagi, 1925, Pöppelmann, 2008).

2.4 The Thirty Years War and its Aftermath

The Reformation brought into reality those irreconcilable controversies between the old and the new faiths resulting in the miserable Thirty Years War (1618-1648). On behalf of the Protestant League Gustavus II Adolphus (Gustav II Adolf) (1594-1632), King of Sweden, fought against the Catholic League side by side with György Rákóczi II (1621-1660), the Hungarian Prince of Transylvania. During the course of this war and following the consequences of this war, Sweden became a great power, a reflection on the merit and efforts of King Gustavus Adolphus. In one of his campaigns conducted in Germany, King Gustavus Adolphus seized the head reliquary of St. Elizabeth (Erzsébet) (1207-1231), daughter of the Hungarian King Andreas (András) II (1176-1235). The reliquary in question is now exhibited in the Statens Historiska Museum, in Stockholm (*Fig. 5*) (Sz. Jónás, 1996).

Fig. 5: Head reliquary of St. Elizabeth, Statens Historika Museum, Stockholm

King Gustavus Adolphus proved to be an outstanding military leader and his reputation became well known throughout Europe. He was regarded as the incarnation of the "Lion of the North", long prophesied by seers and astrologers as being the one to bring salvation from afar. This vision was strengthened by the moral qualities and discipline of his army. Tragically at the age of 38 years he died in action at the triumphal battle of Lützen (1632).

Following the death of King Gustavus Adolphus II (1594-1632), his daughter Christina (1626-1689) succeeded her father to the throne. Being still an infant during the first years of her reign the actual government was placed in the hands of the powerful chancellor, Axel Oxenstierna (1583-1654).

Both Sweden and Transylvania were rich in copper mines. Accordingly, both Oxenstierna and Prince György Rákóczi II, as allies in the Protestant League, attempted unsuccessfully to establish a joint copper monopoly in Europe. Inexplicably Christina, the Queen of protestant Sweden, converted to Roman Catholicism and abdicated the throne in 1654. Moving to Rome, she died there in 1689 and was buried in St. Peter's Cathedral in the first part of the 20th century.

The successor to the throne of Queen Christina was her cousin, Carolus X Gustavus (Karl X Gustav) (1622-1660) and he, together with Prince György Rákóczi II participated in the siege of Brest-Litovsk in 1657. (Andersson, s. a., Köpeczi, 1987) (*Fig. 6*).

Fig. 6: King Carolus X Gustavus and Prince György Rákóczi II in Brest-Litovsk

Fig. 7: Memorial tablet of the King Carolus XII in Budapest

Fig. 8: Title-page of a Hungarian book written on the King Carolus XII

2.5 Recurring Struggles

With the long and devastating Thirty Years' War the hopes of Hungary for independence from Habsburg rule was extinguished. During the reign of King Carolus (Karl) XI, Pope Innocence XI (1611-1689) demonstrated tremendous effort to establish the Holy League with the aim of driving the Turks out of Europe. As a result Buda was liberated in 1686 and some 1200 soldiers, sent by the King of Sweden, contributed greatly to the successful siege of the Buda castle (Károlyi, 1936).

Some years later the Great Northern War became the devastating heritage of the next King of Sweden, Carolus (Karl) XII (1682-1718). With a sense of vocation, Carolus (Karl) XII was a good and indefatigable soldier. After spectacularly winning the battle at Narva in 1700, he suffered a disastrous defeat at Poltava in 1709 at the hands of the army of the Russian Tsar, Peter the Great (1672-1725). Nevertheless, Carolus XII succeeded to refute with a contingent of his army into Turkey. He was exiled in Turkey until November 6th 1714, when, disguised as a simple captain wearing a dark wig (*en mörk Peruque*) and accompanied by a couple of his intimates, he embarked on a fourteen day ride to Stralsund to the amazement of all of Europe. During this journey Carolus XII stopped for a night in Pest and the house in which he lodged is now furnished with a memorial tablet (*Fig. 7*). The Swedish inscription of the tablet in question is slightly different from the Hungarian one and it runs as follows: "Carl XII rastade här på Ridt från Turkiet till Stralsund nov. 1714." (Carl XII rested here on riding from Turkey to Stralsund Nov. 1714.)

At that time there was still no permanent bridge over the Danube so on the following day the king rode over the so called flying bridge towards Vienna. On 22nd November the king wrote to his grandmother "fourteen days ago (*för fiorton dagar*) I started my ride". For long time this accomplishment had no equal one (Ballagi, 1922). François M. A. Voltaire (1694-1778) commemorated the ride of this king, and some decades later a Hungarian novelist, the retired general of cavalry, Count József Gvadányi (1725-1801) did similarly (*Fig. 8*). Tragically, Carolus XII never saw Stockholm again. He was killed by a stray bullet during the siege of the fortress at Fredrikssten.

Mention must be made also of the Swedish contingent of soldiers who remained in Turkey as refugees after the battle of Poltava (1709) together with their king. Some years before their king, the majority of them returned to Sweden. Passing through Hungary, they acquiesced to the demand of Prince Ferenc Rákóczi II (1676-1735) and fought against the Habsburgs in two battles of the Hungarian War of Liberation during the early 18th century. The scene of one of these battles was at Romhány 1710, not too far distant from today's city of Budapest. Two hundred years later a memorial column was erected there in memory of the fallen warriors, among which were the Swedish soldiers who had given their lives for the liberty of Hungary. ("*gåvo…sitt liv för Ungerns frihet*") (Hóman, Szekfű, 1935, Gustafsson, s. a.)

However, not all surviving Swedish soldiers returned home. Due to a lingering illness a young man called Albert Olofsson remained in Hungary indefinitely. After recovering he married a Hungarian maid and he thus became the ancestor of the eminent Benedictine friar Placid Olofsson (1916-), who is still serving in Budapest at a venerable age (Ézsiás, 2004).

2.6 The Last Two Centuries

Due to the efforts and merits of Hans Järta (1774-1847), on 6th June, 1809, the Swedish *Riksdag* passed a new constitution which remains the constitution of Sweden today. This constitution was established along the lines laid down by Montesquieu (1689-1755).

Meanwhile, Hungary passed silent years until the middle of the 19th century. On 15th March 1848 a revolution broke out in Pest, followed by a new war of liberty against the Habsburg realm. The fights and endeavours of the country for independence and freedom gained the sympathy of many countries and people.

Among the warriors of the Hungarian revolutionary army were several Swedish volunteers. Of them a very colourful personality was Fredrik Theodor Krook, a saddler by profession. He served in the army of General József Z. Bem (1794-1850) together with the Hungarian poet Sándor Petőfi (1823-1849) (Gustafsson, s. a.). A few Hungarians from among Krook's comrades emigrated to Stockholm. Samu Manovil became a journalist for Aftonbladet, organized a Kossuth evening in the 1850s and arranged for Lajos Kossuth (1802-1894), leader of the Hungarian fight for freedom 1848-49, to publish an article in the journal.

Sweden remained neutral in WW I, providing an advantageous position for both sides in commercial activities between Sweden and Austria-Hungary.

With the outbreak of WW II Sweden declared its neutrality and succeeded to preserve this status despite the expansion of Hitler's Germany.

At the end of the war Sweden was in a good economic position and preserved its democratic political environment. This positioned Sweden well to give political and charitable aid to the people living in Nazi allied and occupied countries, including Hungary.

2.7 The end of World War II

There were many activities of Swedish authorities and civil organizations to help people who survived Nazi cruelties.

One of these initiatives was the program of "White Buses". This refers to a course undertaken by the Swedish Red Cross and others in the spring of 1945 to rescue concentration camp inmates in areas under Nazi control and transport them to Sweden. Although the program was initially targeted at saving citizens of Scandinavian countries, it rapidly expanded to include people deported from other countries. Those saved included more than 15,000 prisoners in mortal peril held in concentration camps, of which about 7,500 were from other countries which included Hungary. (The term "white buses" originates from the buses having been painted white with red crosses.)

Because Sweden held neutral status in the war, it was not easy for the Swedish Embassy in Budapest to help Hungarian people, particularly after the occupation of the country by Hitler's troops in March 1944. Swedish diplomats did however attempt to give support to people in difficult situations, including among others, the Nobel-Prize winning renowned Hungarian biologist, Albert Szent-Györgyi (1893-1986) who succeeded in surviving the hardships of the last years of the war as a consequence of protection from Swedish diplomats.

Valdemar (Waldemar) G. Langlet (1872-1960) *(Fig. 9)* (see also Point 3.2) was appointed to Hungary as the Chief

Delegate of the Swedish Red Cross during the period of WW II. In this capacity from March 1944 onwards, he and his wife, supported by a group organized by them, did much to protect Hungarian Jews and other persecuted people. He could not hinder the deportation to concentration camps of many hundreds of thousand people of the Hungarian countryside, but he did save hundreds of people by supplying them with Swedish documents. It was to his merit that he sent authentic information about the Auschwitz death-camp to governments of the allied forces and to the state of the Vatican. It was his initiative to provide "Schutzbrief" (protecting documents) and to create protected houses. His intervention contributed to stopping the deportations from the capital in August 1944.

In 1965, Valdemar and Nina Langlet were recognized as "Righteous Among Nations", and their name is commemorated by the naming after them of a street and a school in Budapest *(Fig. 10)*.

Fig. 9: Valdemar Langlet

Fig. 10: Tablet in the street named after Langlet, Budapest

Fig. 11: Raoul Wallenberg

Fig. 12: Wallenberg monument in Budapest

Their compatriot, the diplomat Raoul Wallenberg (1912-1947) *(Fig. 11)* joined and then continued this work and saved the lives of many thousands of Hungarian Jews (Gustafsson, s. a.).

Raoul Wallenberg was an outstanding individual in the Swedish diplomatic corps. He studied architecture, and following his work in international trade, he worked in Stockholm on a committee formed for the purpose of saving people consigned to German concentration camps. His career continued as secretary of the Swedish embassy in Budapest from July 1944.

He contributed to saving many thousand people, after

Fig. 13: *Relief in Raoul Wallenberg Street, Budapest*

tragically more than 400,000 had already been deported from the currently recognised borders of Hungary, as well as many thousands of Gipsy citizens.

August-September 1944 was a relatively "mild" period of the Nazi occupation. On 15th October, when the men of the arrow-cross (Hungarian Nazis) came to full power, those Jewish people who still could remain in Budapest in so called Yellow Star marked houses were constrained in a "black ghetto" created by the Hungarian Nazi authorities. Many people were killed in the streets around the city, or shot at the river bank into the waters of the Danube.

This time Wallenberg helped the persecuted people, providing all possible assistance based on his Swedish diplomatic position, spurred by his sense of humanity and personal courage. He saved more thousand Hungarian citizens who were declared as Jewish according to the laws at that time.

In mid January 1945, when the headquarters of the Second Ukrainian Front of the Red Army was in East Hungary, Wallenberg wished to personally contact the Soviet command. To this day it remains unknown as to how and why he was arrested and taken to Moscow. It is assumed, though unconfirmed, that he died in prison in 1947.

The grateful Hungarian people preserve Wallenberg's memory with a sculpture by the famous sculptor Imre Varga (1923-) mounted in the capital *(Fig. 12)*; the naming of the street *(Fig. 13)* in the district where he established Swedish protected houses, as well as a commemorative plaque at the Józsefváros railway station from where trains left with the deportees, destined for the Nazi extermination camps. Wallenberg was himself present at the station, saving many people before they were locked into the wagons.

This year of 2012 is denoted in Hungary as "Wallenberg year" celebrating the 100th anniversary of the birth of this outstanding Swedish diplomat.

In the years following 1944-1945 many Hungarian citizens emigrated to Sweden, including many who shuddered at the past Nazi terror, as well as those who were fearful of the increasing Soviet influence.

2.8 Post World War II

Following the war there was relatively good connection between neutral Sweden and the Eastern European block to which Hungary then belonged.

It was clear that Sweden showed a deep sympathy to the 1956 Hungarian uprising, an unsuccessful attempt to reach independence and a multi-party political system. After the defeat of the Hungarian Revolution by Soviet intervention, about 8,000 Hungarian refugees emigrated to Sweden and found shelter and a future there.

The assassination of the Prime Minister of Sweden, Olof Palme (1927-1986) shocked the Hungarian public sentiment. The House of Hungarian Creative Artists held a promenade in a Budapest City Park where a memorial was established in commemoration.

Following political changes in Hungary in 1989, the relationship between Sweden and Hungary has become much closer. Sweden entered the European Union in 1995, and since the time that Hungary joined (2004) we belong to the same economic community.

We may say that the peak of international relations between Sweden and Hungary was achieved in 1991 with the state visit from King Carolus XVI Gustavus (Carl XVI Gustaf) (1946-) to Hungary (*Fig. 14*) (Gustavsson, s. a.). Anecdotally, the

Fig. 14: *King Carolus Gustavus XVI and the Hungarian President Árpád Göncz in the company of the Royal Consort and the Hungarian First Lady at the entrance of Hungarian Parliament*

Swedish king, being a passionate hunter, pays unofficial visits to Hungary more frequently.

2.9 Reflections on the past and hopes for the future

As we discussed, the histories of Sweden and Hungary were largely divergent. However, there has been a long period of sympathetic alignment between the two populations. We may be thankful that there has always been cooperation and friendship between the two nations.

Presently the commercial, industrial, agricultural connections are increasing. Friendship in sports is traditional. In the field of science the connections can be said to be excellent. Collaboration and exchange in the fields of music, art and literature is exemplary.

We can look forward to the future and hope that links between Sweden and Hungary will develop in a peaceful European environment, enjoying mutual benefit, following the traditional friendship and helpfulness.

We are sure that the approximately 35,000 inhabitants in Sweden of Hungarian origin, preserving the attitude of their ancestors, will contribute advantageously in their work to their Northern homeland, and enhance the good relations with their former home country.

3. SCIENCE AND CULTURE

Swedish-Hungarian links were plentiful in spite of the geographical distances and differing economic and political environments spanning several historical periods. One could fill volumes, but in this article of our technical journal it is only possible to highlight some randomly selected examples with no exact thematic or chronological order and no possibility of being comprehensive. The following highlights connections in music, in the different branches of art and sports.

3.1 Mutual effects in various fields of natural science

As early as 1807 a book on herbs was published by two Hungarian gentlemen of enduring eminence. One of them was Mihály Fazekas (1766-1828), lieutenant by profession and a poet for pleasure. The other author of the book was Sámuel Diószegi (1761-1813), a protestant clergyman. This book on herbs is the first Hungarian work composed according to the taxonomy of Carl Linnaeus (Linné) (1707-1778) (*Fig. 15*), the great Swedish pioneer of the universal humane science and contemporary of Christopher Polhem (see below). The

Fig. 15: Portrait of Carl Linnaeus by A. Roslin

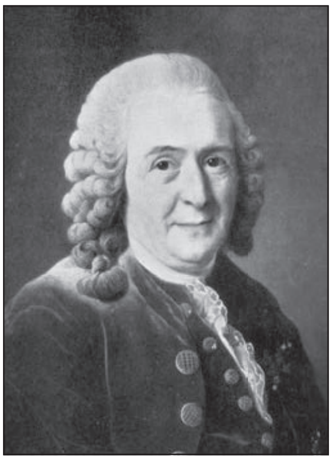
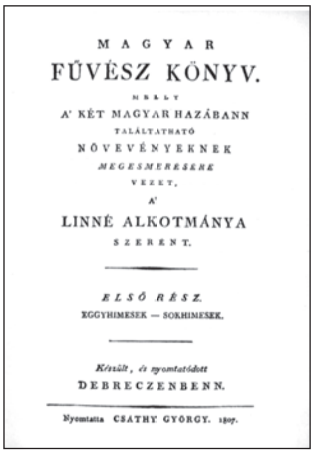


Fig. 16: Title page of a Hungarian book following the taxonomy of Linnaeus



MAGYAR
FŰVÉSZ KÖNYV.
MELLY
A' KÉT MAGYAR HAZÁBANN
TALÁLTATHATÓ
NÖVEVÉNYEKNEK
MEGESMERÉSÉRE
VEZET,
A'
LINNÉ ALKOTMÁNYA
SZERÉNT.
ELSŐ RÉSZ.
EGGYHIMESEK — SOKHIMESEK.
Készült, és nyomtatódott
DEBRECZENBENN.
Nyomtatta CSÁTHY GYÖRGY. 1807.

principal merit of the above mentioned Hungarian book is the substantial contribution to the development of Hungarian botanic language (*Fig. 16*).

At the conclusion of the Thirty Years War Sweden was able to utilize the fruits of her victory in substantial economic and scientific achievements. Many of these endeavours were forged by Christopher Polhem (1661-1751), whose forefathers are deemed to be of Hungarian origin (Mihalovits, 1938). His merits are acknowledged by a memorial tablet in the Hungarian Museum of Metallurgy in Miskolc (*Fig. 17*). Christopher Polhem is well-known as a scientist, inventor and industrialist of Sweden, who greatly contributed to the economic and industrial development of the country. He is also considered to be the father of Swedish engineering.

Fig. 17: Memorial tablet of Christopher Polhem in Miskolc, Hungary

We should not overlook the story of the humble safety match. Actually it was invented by a young Hungarian chemist called János Irinyi (1817-1895), who did not anticipate its future importance and so sold the licence to an Austrian businessman for almost nothing. The businessman in question passed on the licence to Sweden, where the invention was developed by Gustav Erik Pasch (1788-1862) in Jönköping. The first successful match factory was erected there in 1845, and decades later the product returned back to Hungary with the name of "Swedish Safety Match" (see Point 1).

From among the Hungarian Nobel Prize Winners there were the following who left for Sweden: Róbert Bárány (1876-1936) won the medical-physiology prize in 1914, and lived in Sweden after WW I. György Hevesy (1885-1966) Nobel prize winner in 1943 in the field of chemistry (radioactive isotopes) lived in Sweden from 1933. György Békesy (1899-1972), prize winner in 1961 in physiology, lived in Sweden from 1946 for a relatively short time.

Lars (László) Ernster (1920-1998) was born in Hungary, emigrated to Sweden in 1946. He was professor of biochemistry and member of the Royal Swedish Academy of Science and the Board of the Nobel Foundation. The Hungarian born Georg (György) Klein (1925-) microbiologist has been living in Sweden since 1947 and was professor of Karolinska Institut. Among many other memberships he was member of the Royal Swedish Academy of Sciences as well as the Nobel Prize Committee for physiological and medical sciences. Marcell Riesz (1886-1969) graduated from the Budapest University of Science. He arrived to Sweden in 1913, worked in Stockholm, and was later appointed as professor of mathematics at the University of Lund.

These are only a few examples. There were many other individuals who were of Hungarian origin and enriched the world of science under the flag of Sweden.

3.2 Literature, theatre, art

Connections in the field of linguistics, prose, poetry and fine art are wide ranging. Following are a few examples.

Béla Leffler (1887-1936) was first active as an instructor of the Swedish language at the Budapest University of Sciences, then, in 1919 he moved to Sweden where he performed translations from Hungarian to Swedish including "The

Tragedy of Man" by Imre Madách (1823-1864), and from Swedish to Hungarian the works of Selma Lagerlöf and August Strindberg.

János Lotz (1913-1973) founded an international language institute with a centre in Sweden. He had been teaching Hungarian language at Swedish universities. He later found employment at the Hungarian Institute of the Stockholm University as an instructor of the Hungarian language.

Valdemar (Waldemar) G. Langlet (1872-1960) moved from Sweden to Hungary in the 1930s. He was instructor of the Swedish language at the Budapest University of Science (His other activity see Point 2.6.)

We may say that the most popular Swedish writer enjoyed by Hungarian readers was Selma Lagerlöf (1858-1940). Her first novel, "Göste Berlings-Saga", was translated into Hungarian in 1912 by Marcell Benedek (1885-1969). Following this, the translation of her novels came in series. Dramas, novels and poems of August Strindberg (1849-1912) were present in the Hungarian language from the 1880s (Köpeczy, Pók, 1976).

Poems of Swedish poets were translated into Hungarian and published by Vilmos Győry (1838-1885) in a collection *(Fig. 18)* almost two decades before the end of 19th century (Győry, 1882).

Sándor Petőfi (1823-1849) was one of the ever greatest poets of Hungary, of whose poetry became well-known also in other countries, among them in Sweden. A selection of his poems had been published rather early, there (*Fig. 19*). A great propagator of Petőfi's poems in Sweden was Birger Schöldström (1840-1910), who had learned even the Hungarian language and became a devoted interpreter of the Hungarian poetry in Sweden, resulting thus his membership in the Hungarian Academy of Sciences. Besides, his home in Stockholm was a real centre of Petőfi's culture.

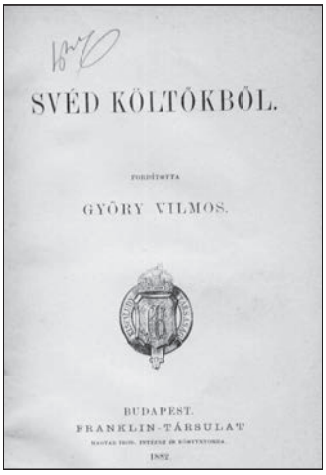


Fig. 18: Title page of an early volume containing poems of Swedish poets translated to Hungarian

Fig. 19: Title page of early Swedish translations of Petőfi's poems

In Hungary many translations from Swedish poets were published in 1960s and 70s (Hajdu, 1964., Bernát, 1967., Thinsz, 1974).

It is noteworthy that Ove Berglund (1940-), a doctor of medicine as well as a man of letters, translated into Swedish famous Hungarian poets such as Attila József (1905-1937), Mihály Babits (1883-1941), Miklós Radnóti (1904-1944), Sándor Kányádi (1929-), Sándor Weöres (1913-1989).

By the end of the 19th century Hungarian literature became well known in Sweden. More than fifteen works by one of the greatest of Hungarian novelists, Mór Jókai (1825-1904) had been translated in Swedish. The play by Ferenc Molnár (1878-1952) entitled "The Swan" (In Hungarian "Hattyú") became a great hit on the stages of Stockholm. Mention must also be made of the connection between the Hungarian poet and novelist Frigyes Karinthy (1887-1938) and Sweden. Karinthy suffered a severe cerebral tumour which required an urgent operation. In the first part of the 20th century there was an outstanding master of such operations in Sweden, the surgeon Herbert Olivecrona (1891-1980). He operated on Karinthy and this event was commemorated by the writing of a very successful novel entitled "A journey around my skull" (In Hungarian "Utazás a koponyám körül").

Prosaic Swedish works were widely translated into Hungarian (e.g. Horváth, 1942., Lontay, 1965).

Georg (György) Klein (See Point 3.1) besides being an excellent scientist in cell biology, is also acknowledged as Swedish writer of excellence.

László Sall (1961-) lives in Göteborg since 1990. He is founder of the Cultural Club "Kőrösi Csoma Sándor" in Göteborg, and is editor of "Two Seasons" (Két évszak) cultural CD series. He is author of several papers on Hungarian culture and history (Sall, 2011).

Let us now glean in the world of theatre and cinema.

Greta Garbo (1905-1990), as a world famous actress, was well known in Hungary by such films as Grand Hotel, Queen Christina, Anna Karenina, Conquest, Ninochka and Two Face Woman. The authors of the script of her film "Grand Hotel" were William A. Brake and the Hungarian, Béla Balázs (1884-1949).

The films of Ingrid Bergman (1915-1982) were widely screened in Hungary, including "For whom the Bell Tolls" and "Murder on the Orient Express". There were other films with Hungarian connections such as "Casablanca", directed by the Hungarian Mihály Kertész (1886-1962) and "Gaslight", under direction of George (György) Cukor (1899-1983). The films of Ingmar Bergman (1918-2007) were popular with the Hungarian public: "Fanny and Alexander", "Cries and Whispers", "Scenes from a Marriage", "Wild Strawberries" and others.

In 1990 a film was created as a Swedish-Hungarian co-production on Raoul Wallenberg (See Point 2.7).

Judit Benedek (1951-), who left Hungary with her parents for Sweden after the 1956 Revolution, founded in Stockholm an actors' college in 1974 and study of stage-management in 1984. Since 2004 she has been directing plays of Swedish playwrights in the Budapest Kolibri Theatre. Plays for children and youths were translated into Hungarian by the well known Hungarian professor and translator, László Kúnos (1947-). He translated many works of Per Olov Enquist (1934-) who in 2011 received the Budapest Grand Prize. Kúnos also translated other Swedish authors such as Selma Lagerlöf and Ingmar Bergman (Benedek, 2012).

János Herskó (1926-2011) was a famous Hungarian movie-director who directed popular films in Hungary and left for Sweden in 1970 where he continued his work and engaged in the training of moving picture specialists.

In the fine arts represented by sculpture and painting, there are many mutual links between the two countries. The famous Hungarian painter László Paál (1846-1879) formed a friendship in Paris with Swedish colleague Carl Fredrik Hill (1849-1911). Both profited artistically from this connection. The art and design of the Swedish architect Ragnar Östberg (1866-1945) was recognised and acknowledged in Hungary also. A resemblance of his style is recognizable on the central building of the University of Szeged, Hungary, constructed by the Hungarian architect Béla Rerrich (1881-1932).

In contemporary times there are regular exhibitions in Stockholm organized by György Tamás displaying the works of Hungarian artists. There exists a Universal Hungarian Association in Stockholm of which two most active people are Tamás Gergely (Stockholm) and Károly Tar (Lund).

Budapest houses a Foundation "Scandinavian House". The aim is to foster cultural and economic activities between Hungary and Sweden as well as with its neighbours.

There are other fields of art, such as music, ballet and circus in which there are extensive links, but insufficient space in this article to detail.

4. TRADE, COMMERCE

There are wide ranging industrial and commercial links between the two countries. Examples of significance follow:

The rolling bearings factory SKF (Svenska Kullager Fabriken) was founded in 1907 in Göteborg and soon developed a very strong influence on Hungarian mechanical industry. From 1917 a distributor of the products of this factory was established in Hungary. In the 1920's the links were strengthened and a joint Swedish-Hungarian company was established in 1929. In the 1930's production capacity in Hungary was significant. In 1948 SKF set up Hungarian headquarters in Budapest (which moved recently to Budaörs, a suburb of the capital). There was a contract between SKF and the Hungarian State which insured the continuation of cooperation until the present time. SKF contributed to the Hungarian rolling bearing factory in Debrecen.

A long tradition of Swedish export to Hungary exists in the field of electricity. One of the main organizations was the Elektromekaniska AB, founded in Stockholm 1918. Extensive export activity existed between Sweden and Hungary in the period between the two world wars. A major trading firm was named Elektrolux. In 1991 the Swedish company acquired the Hungarian firm Lehel. The factory in Jászberény continued and improved the production of the excellent and famous Lehel refrigerators. Elektrolux vacuum cleaners, mixers, washing machines and dishwashers were well known in Hungary. The Elektrolux-Lehel, (later called Elektrolux-Zanussi) continued the trade between Sweden and Hungary.

Swedish industry in railway rolling stock, heavy vehicles and aircraft plays an important role.

The installation of NOHAB locomotives on Hungarian railways was of great importance. In Hungary during the late 1950's the proportion of steam operated locomotives was about 85%. There was need to modernize but the Hungarian railway vehicle industry did not have the capacity to produce up-to-date Diesel engines. NOHAB (Nydqvist och Holm Aktiebolag) won the tender, and the first locomotives had their test run in 1960. MÁV (Hungarian State Railway Company) purchased 20 NOHAB locomotives. They moved first between Budapest and the Eastern region of the country and after the electrification of those lines NOHABs were running in the surroundings of the Lake Balaton. After upgrades, these vehicles served rail traffic until 2000. The original engine is exhibited in the Hungarian Railway History Park.

The SAAB motor vehicle factory delivered from 1947 a range of specialized utility vehicles. After political-economic changes in Hungary domestic vehicles were also brought in to Hungary.

There is another product of Swedish industry which plays an important role. The Hungarian Air Force started leasing in 2006 SBS 39 type Gripen aircrafts for 12 years.

Volvo cars left the Göteborg factory already in 1927. Models were available for purchase through an Austrian-Hungarian firm since 1991. The first Volvo dealers in Hungary were opened in 1992. In 1995 the Volvo Car Hungary was established. Now there are 15 Volvo dealers and 18 service centres in the country, partly in the capital, partly in major cities. Almost 1200 Volvo cars were sold in Hungary during 2011. Volvo buses, trucks and other vehicles are also popular.

It should be noted that Volvo takes part in Hungarian social life by supporting Hungarian children in the Volvo Adventure competition, which is an environmental award program organized each year jointly by Volvo and the UN Environment Program. Volvo Car Hungary has been sponsoring the Hungarian Water Polo Association (Bodrogai, 2012).

There are many commercial organizations, such as IKEA, which are well known in Hungarian life. The traditional Hungarian export to Sweden has been mainly in the area of agriculture, but industrial sell to Sweden is also developing.

5. CONCRETE

5.1 Hungarian engineers in Sweden

Following are a few examples of Hungarian born and educated engineers who were close to concrete technology and contributed to Swedish science and construction.

5.1.1 Hungarians who contributed to the Swedish building industry

There were many engineers of Hungarian origin in different fields of construction who settled in Sweden. Among them are architects having interest in concrete. One such professional is Kornél Pajor (1923-) who attended the Technical University of Budapest until 1949. In that year he became skating world champion in Oslo, from where he left for Sweden. There he worked for air navigation services contributing to the design of airport buildings as well as residential houses and other structures. Tibor Hottovy (1923-) is an outstanding specialist in town planning and informatics. He lives in Sweden and contributes significantly to international development.

Ferenc Rathing, civil engineer attended Budapest University of Technology and graduated in 1931. He worked in Hungarian concrete bridge construction and prefabrication. In 1956 he emigrated to Sweden and was a staff member of the Stockholm Gatukontoret (city management).

5.1.2 A famous Hungarian scientist in Sweden

Gábor Kazinczy (1889-1964) *(Fig. 20)* was a ground breaking engineer in the theory of plasticity. He graduated in 1911 from the József Technical University in Budapest with a Doctorate in 1931, followed by a Dr.-habil degree in 1939.

During his employment in the municipality of Budapest, he was head of the material and structural testing laboratory. He was then appointed as deputy chief of the city's department of construction and finally Chief Councillor of Engineering until 1943.

He carried out extensive research work and developed a reputation for his theories in many countries.

Based on his experimental works and knowledge of theory he published a famous report in 1914 on the plastic

Fig. 20: Gábor Kazinczy

behaviour of statically indeterminate beams. He extended his theory for steel girders to ones of reinforced concrete. After 1921 he continued his work in the field of reinforced concrete slabs. His doctoral thesis discussed the residual strains in constraint beams and he carried out research work in the field of folded plates and shells. Among the many other fields in which his knowledge impacted, he improved the application of reinforcing bars strengthened by cold working.

Gábor Kazinczy was one of the first researchers who, based on his experimental work, applied the principle of plastic hinges and the theorem of limit state analysis. His early investigations in 1914 proved that the flexural load capacity of members is not exhausted when reaching the yield point at any one place along the beam, but there are reserves until the development of yield mechanism and the element loses its load bearing capacity.

Gábor Kazinczy contributed to the early Hungarian codes for load carrying structures. Contemporary European codes of structural concrete follow the essence of his very early original findings.

After WW II Kazinczy left Hungary for Denmark. From 1947 until his retirement in 1959 he was active in Sweden, where he worked at the Kooperativa Förbundet Arkitektkontor. He designed grain silos, shell structures, prestressed concrete floors among other notable engineering works. He continued research work, publishing scientific papers and he lectured at international professional meetings.

In 1957 the Association of Hungarian Engineers and Architects was founded in Sweden, and Dr. Kazinczy became the president of this organization. It was his initiative to found the World Federation of Hungarian Engineers and Architects in 1958, headquartered in the United States and to which he was elected as deputy president.

Living in Sweden he did his utmost to maintain contact with Hungarian scientists. A token of this was his meeting with the Hungarian delegation on the occasion of the 1960 congress of IABSE in Stockholm. Károly Széchy, professor at the Technical University of Budapest, greeted Dr. Kazinczy as a valued old acquaintance and the author1 of this paper was also fortunate to shake hands with the distinguished and internationally acclaimed engineer.

It is to be noted that Endre Reuss (1900-1968), famous professor of technical mechanics in Budapest, informed his students in 1946 of the merits of Dr. Gábor Kazinczy as an epoch-making scientist in field of plasticity.

The name of Kazinczy is well known in Hungary. Ferenc Kazinczy (1759-1831), great-grandfather of Gábor Kazinczy, was a leading contributor to Hungarian literary life and an important person in the enlightenment movement as well as in the reform of the Hungarian language.

The son of Gábor Kazinczy, tekn. dr. Ferenc Kazinczy (1929-), is a recognized engineer in Sweden, who acknowledges his Hungarian origins. He has contributed to this remembrance of his father. Both authors of this paper were honoured to meet Ferenc Kazinczy at the reception of the FIP Symposium in Budapest in 1992, held at the National Gallery located in Buda Castle. It was an impressive feeling to meet and greet tekn. dr. Ferenc Kazinczy, standing adjacent to the portrait of his ancestor, the legendary Hungarian linguist and man of letters of the 18th-19th centuries (Kaliszky, 1984, 2007; F. Kazinczy, 2012, Lenkei, 2000, Mihailich, Haviár, 1966).

Fig. 21: László Frigyes

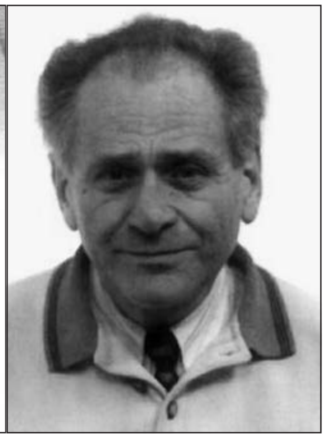


Fig. 22: Gábor Kalmár

5.1.3 Two engineers in design, construction and research

Below is a brief curriculum of two Hungarian born and educated engineers serving the Swedish building industry and concrete science.

László Frigyes (1926-1992) *(Fig. 21)* was born in Hungary. He graduated as civil engineer from the Technical University of Budapest (BME). He was active in various design tasks of the M2 metro line in Budapest. After the 1956 Revolution he continued his work in Sweden. Between 1956 and 1961 he was employed by Skanska Cement in Malmö. His main work was construction of highways, buildings and establishments for power stations. 1961-1989 alongside of his engagement in construction of roads and waterway lines at Syedkraft Malmö (later E-on), he took part in many other projects and their contracts. Among these were the Malmö sport stadium, buildings of the Barsebäck (South Sweden) nuclear power plant, wind power stations North to Trelleborg. He created engineering structures for solar energy equipment in Limnhamn. From late 1980's his work was limited due to illness but he remained active almost until his death in 1992.

Gábor Kalmár *(Fig. 22)* was born in Budapest in 1930. He attended the Civil Engineering Faculty of the Technical University of Budapest from 1949 to 1953. After graduating he worked as an engineer at the Budapest Water Plant. He left for Sweden in 1956. After various technical activities he continued his career in the concrete industry in 1962. He was employed as research engineer at A-Betong and A-System, later at Ytong AB. His principal field of interest was the application, properties and development of lightweight concrete. Following a post-graduate course he achieved the tekn. lic. degree in 1966. From 1982 he was active as research officer at the Swedish State Building Research Institute (SIB) and after restructure, at the Gävle Branch of the Royal Technical University (KTH BMG). He retired in 1997. Gábor Kalmár continued his work at this same institution as project employee. In 1998 he founded a private enterprise "GK-Råd". Meanwhile he gave lectures on construction materials at KTH-BMG.

Some topics of his research and design work were: heat insulation of sandwich panels; refurbishment of lightweight concrete façades; room units using high performance concrete.

5.2 Mutual and common works in concrete

In this paper we can only demonstrate selected incidences where Swedish and Hungarian cooperation was successful. The merit in great extent is due to international professional associations, for us most importantly, the ***fib***.

5.2.1 Prefabrication

Leaders of BVM (Concrete and Reinforced Concrete Works, Hungary) studied the best open building systems and selected the Swedish Strängbetong system. BVM became the system owner and TTI (Design Office for type buildings) became the designer of the BVM-TIP. A building catalogue for BVM-TIP was published in two volumes by TTI in 1988.

There was cooperation in railway sleeper technology. After the political changes in Hungary the BVM factories were privatized. Gy. Fogarasi, previous leading staff member of BVM, was at that time active in Omaha, USA. In 1990 the American Wilson concrete company decided to develop railway sleeper technologies. Fogarasi consequently visited a number of manufacturing plants including two factories: the A-Betong Sabema long line technology as well as Strängbetong plant in Veddige.

In 1981 the steel moulds for the extension for the Veddige plant were manufactured by BVM Hungary (Fogarasi, 2012).

5.2.2 Slipforming

5.2.2.1 The SVETHO System

One of the most advanced branches of Hungarian concrete construction has been the application of slipform technology. This was performed to a large extent with Swedish cooperation.

AB Bygging (Stockholm) was founded in 1942. In 1944 they invented the hydraulic slipform jack and developed centrally controlled hydraulic equipment. The first one was developed in 1947. Uddemann Byggteknik AB was founded in 1955. The company supplied slipform in 1965. In 1980 the two companies merged to become Bygging-Uddemann AB.

This Swedish company extended its activity to many countries including Hungary where it found very good partners. Mélyépterv Engineering Design Bureau and ÁÉV 31 contracting firm have made steps to develop slipforming. A joint firm was founded in Hungary and Sweden in 1969 named Bygging-Ungern 31. AB *(Fig. 23)*, within the framework of the Hungarian state building company ÁÉV 31.

The main feature of the SVETHO-TM system (an improvement on previous versions SVETHO-U, -S and –E) was the expansion of productivity through centric situated hydro-mechanical equipment. The prescribed changes of designed parameters, like the diameter and wall thickness, are controlled in all units at the same time. The automatic system enabled the reduction in employment of blue collar workers and further resulted in higher speed of the construction work and greater accuracy. It is to be mentioned that the Hungarian engineer in industrial surveying, Gyula Holéczy (1925-1994), designed and applied the controlling device and method to minimize the tolerance in geometrical data.

The Bygging-Ungern 31. AB produced in Hungary and delivered slipform technology equipment to many countries.

The cooling towers, water-towers, chimneys and TV towers constructed by the SVETHO system are found in many countries today. Their height reaches 250-300 m, diameter over 100 m with inclination of wall up to 25%.

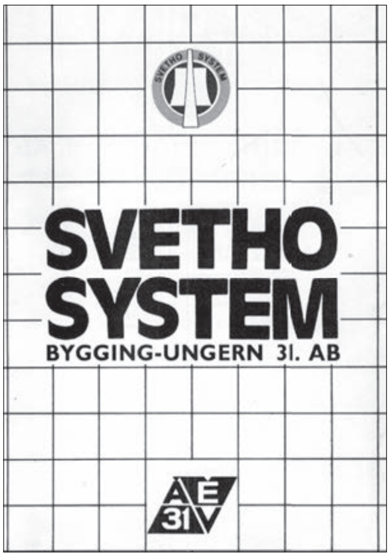


Fig. 23: Logo of SVETHO System

In more than 20 countries and on all continents more than 100 giant slipformed concrete structures announce the results of Swedish-Hungarian cooperation.

Joint work with Sweden and the merits of Sven-Erik Svensson in cooperation with József Thoma had an extraordinarily high value.

As an example *Fig. 24* shows the cooling towers and chimney of the Gyöngyösvisonta power plant in Hungary.

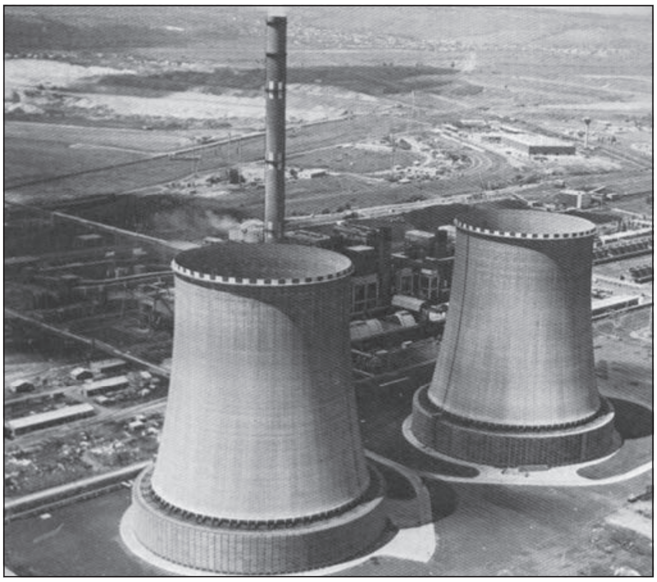


Fig. 24: Cooling towers and chimney of the power station at Gyöngyösvisonta, Hungary

5.2.2.2 Eponyms of the SVETHO system

Sven-Erik Svensson (1915-1997) *(Fig. 25)* graduated in 1940 in civil engineering at the Royal Institute of Technology (KTH), Stockholm. During his professional career he made particular contributions to the development of the slipforming technique for various forms of concrete construction. Additionally he developed techniques for the handling of heavy loads using special hydraulic jacks and related devices. These achievements have resulted in economic benefits and improved safety in the construction of a variety of reinforced concrete structures such as water towers, suspension bridge towers and nuclear

Fig. 25: Sven-Erik Svensson

containments; perhaps the best known being the tapered CN-tower in Toronto.

Svensson was active as a construction consultant on sites in more than 70 countries. In addition he regularly lectured at Swedish and international universities and engineering associations (Spratt, 1982).

Sven-Erik Svensson played a definitive role in the Swedish-Hungarian links in the field of concrete technology. His experience enhanced collaboration with the Hungarian, József Thoma resulting in the creation of the SVETHO slipforming system and founding the Bygging-Ungern 31. AB joint enterprise.

It was a pleasure for Author1 to be present in Stockholm during 1982, when Miklós Márkus – later general manager of Bygging-Ungern 31. AB - who headed the Hungarian delegation at the FIP Congress Stockholm in that year, congratulated Sven-Erik Svensson when awarded the FIP Medal.

József Thoma (1922-2009) *(Fig. 26)* graduated from the Technical University of Budapest (BME) in 1946. Worked as assistant to professor at the Department of Hydraulic Constructions No. 1. From 1948 he was one of the founders of the Mélyépterv engineering design bureau in Budapest, and from 1993 was owner and managing director of Mélyépterv Consulting Ltd.

Fig. 26: József Thoma

During the late 1950's Thoma was active in the design and development of various prestressed concrete structures. In 1957 he was awarded for his achievements the Kossuth-Prize (shared with Béla Gnädig), the highest professional distinction in Hungary.

From 1955 he directed his talent mainly to the development of slipform technology. He improved the procedure mainly in field of structures with changing diameter and thickness (e.g. see Fig. 24).

From 1969 significant international developments were formed in fields of design and construction. His best known patent was for the equipment SVETHO in Swedish-Hungarian cooperation which resulted in the foundation of Bygging-Ungern 31. AB. He worked in many other countries (e.g. Germany and Turkey) and held 17 different valuable patents in Hungary and abroad. From among approximately 500 of his diverse works 350 structures are standing abroad. The most attractive engineering works were completed using the SVETHO system. He was awarded the Széchenyi-Prize in 1993 (Balázs, Borosnyói, Tóth, 2007). During his life and engineering activity Thoma has been a beacon of fruitful cooperation between Sweden and Hungary.

6. CONTACTS IN INTERNATIONAL ORGNIZATIONS AND EXCHANGE PROGRAMS

6.1 Events of societies dealing with concrete

The congress of IABSE-IVBH-AIPC took place in Stockholm in 1960. This was one of the first occasions after WW II for Hungarian engineers to have the possibility to engage in international professional life. The environment for good links between specialists coming from different countries was excellent within the walls of the Technical University in Stockholm (Kungliga Tekniska Högskolan, KTH). The well organized event was an early forum to build enduring connections between Swedish and Hungarian engineers.

Károly Széchy, professor of Technical University of Budapest (BME) headed the Hungarian delegation. Author1 was fortunate to attend the congress and all of its social events. Besides the technical sessions, the opening ceremony in the Konserthuset *(Fig. 27)*, the Orgelandakt i Storkyrkan and the banquet in the Stadshuset all provided good oppportunities to meet Swedish colleagues. The civil engineering objects of the Swedish capital, the underground, bridges, up-to-date concrete housing in suburbs and other structures were edifying.

The post congress tour to the northern territory in the Arctic Circle left very deep impressions on the participants. The Sandö bridge – the world's largest concrete structure at that time – , the arches across the Lule River, the structures of the Harsprånget hydraulic power plant, structures of Kiruna and Malmberget iron ore mining estates were all very informative. Author1 together with Árpád Apáthy, later head of Bridge Department of Hungarian Ministry of Transportation, related in several Hungarian cities the achievements of the Swedish construction industry.

Herbert Träger (1927-), past president of Hungarian IABSE Group was, through this organisation, in contact with Werner von Olnhausen who was chief engineer of Swedish central road administration. They both had occasion to contribute to work at different meetings of the association.

The IABSE Symposium 2006 took place in Budapest. Peter Collins from Sweden was a member of the scientific committee and four contributions were presented by Swedish delegates.

The 9th FIP Congress, held in 1982, was an important event in Stockholm. The congress was rich in its technical program. From among 11 Hungarian delegates, four papers were presented (M. Márkus, J. Illésy and Author1 presenting two papers). This congress was a good occasion to widen the professional contact between the Swedish specialists and the Hungarian participants. In 1992 Karl-Gustav Bernander (see Point 6.2) was invited to be the chairman of the first session of the FIP Symposium in Budapest. Johan Silfwerbrand, who is now the chairman of the Organizing Committee as well as Deputy Chairman of the Scientific Committee of *fib* Symposium 2012 Stockholm, contributed much to the FIP Symposium in 1992. In the following decades he became a leader in Swedish concrete technology and nurtured contacts with his Hungarian counterparts.

Fig. 27: Members of the Hungarian delegation to IABSE Congress 1960 at the entrance of Konserthuset, Stockholm

The CEB+FIP=***fib*** association has played a significant role for long decades in concrete technology on all continents. The 31st CEB General Assembly in Stockholm 1997, following a suggestion made by a common CEB-FIP implementation group, approved the merger with FIP to create *fib,* in which Hungary was represented by Author2.

6.2 Two Swedish leading personalities of professional organizations supporting international links

Ralejs Tepfers *(Fig. 28)* was born in Rezekne, Latvia in 1933 and lived in Sweden since 1944. He graduated in 1958 from the Chalmers University of Technology and received tekn. lic. title in 1966, and a PhD in 1973. Since 1996 he is Dr.-Ing. h. c. Latvian Academy of Sciences where he is an external member since 2010. He has many awards, among them lifetime honorary member of *fib*, and his distinguished work is proven by his 300 scientific papers. He championed Swedish-Hungarian cooperation.

Ralejs Tepfers was at the forefront of new and improved standards in several fields of concrete science. The bond between concrete and reinforcement is an enduring problem for reinforced and prestressed concrete. The extensive experience of Ralejs Tapers realised the importance of dealing with bond questions in the frame of CEB. He became convener of CEB TG VI/1, the task group dealing with bond and anchorage zone questions. Among his many merits he recognized that there were engineers in the east, who were well experienced and ready to contribute to this international work. He built professional relationships with Jerzy L. Zielinski (Warsaw), Alberts Skudra (Riga), and Vladimír Urban (Prague).

Both authors of this paper were fortunate – together with other countrymen - to be able to participate in the initiatives of Ralejs Tepfers. Through this a close cooperation between Göteborg and Budapest was established.

The lion's part of the work was performed by the convener. He built the task group with experts from Latvia, Czech Republic, Poland, The Netherlands, UK, Germany (from where also Andor Windisch is of Hungarian origin), Italy, USA (including the Hungarian born Péter Gergely), Greece and Hungary. This demonstrates the organisational force of the convener. Nevertheless, we may say without immodesty, that the Hungarian part of the task group was a significant contributor to the work. This was shown mainly in the publication work creating CEB and *fib* Bulletins in the field of bond problem.

This task group worked for 20 years. Ralejs Tepfers made it a rule that several meetings should take place in an Eastern European country and the meetings should always be low budget. There were meetings in Warsaw, Prague, Budapest, as well as in Rotterdam, Milano, Athens, Treviso, Stuttgart, Munich and Göteborg. Some meetings were connected to CEB Plenary Sessions.

Later in the 1990's, following political changes in Eastern Europe, contacts became easier. The international activity of Ralejs Tepfers has continued and the conference in Riga (1992) and – together with Author2 - Budapest (2002) are examples.

Let us refer to the words of Ralejs Tepfers (Tepfers, 2011): "It should be stated that the success of the collaboration on bond of reinforcement in concrete was a result not only of scientific excellence but also on deep friendships between the task group members and a mutual understanding which developed during the years of common work and collaboration."

Fig. 28: Ralejs Tepfers

Fig. 29: Karl-Gustav Bernander

Indeed, the common work of the convener of a task group from Gothenburg, Sweden and group members from Budapest, Hungary created a sincere friendship between participants. This is a good example that concrete can bring people close to each other. In this the authors of this paper are honouring Ralejs Tepfers, who did very much for strong connections between Sweden and Hungary.

Karl-Gustav Bernander *(Fig. 29)*, a respected engineer, was born in 1924 and graduated from the Chalmers University, Göteborg, in 1950. He carried out research work at Chalmers until 1954. Since then he has been with Strängbetong AB and was technical director until 1964. The majority of research works for precast pre-tensioned concrete within the company was carried out and published by him. From 1974 to 1983 he was professor at Chalmers University. He contributed much to Swedish codes on concrete construction. Presently he is general manager of Bernander Prefab Konsult AB in Stockholm.

Karl-Gustav Bernander started his work in FIP in 1962, and from 1970 he was chairman of FIP Commission 5 "Prefabrication". He was also vice-president for Sweden.

Hungarian experts, M. Márkus, J. Beluzsár, Gy. Gecsényi, were working in the commission under his chairmanship. From among the meetings of the commission we would mention the sessions of significance concerning Swedish-Hungarian links, i.e. those in Budapest 1974,, Göteborg 1975, Budapest 1977 and 1992.

K-G. Bernander was awarded the FIP Medal in 1984 (Shacklock, 1984). Under the chairmanships of K-G Bernander task groups were convened with other Swedish specialists and with Hungarian collaboration, e.g. the task group on concrete railway sleepers was headed by Kent Gylltoft, and Author1 also worked in this task group (Gylltoft, 1986).

K-G. Bernander was nominated as Chairman of the first session of the FIP Symposium, Budapest 1992. Author1 was fortunate to greet K-G. Bernander and his commission members in the Laboratory for Building at the Technical University of Budapest, and in the same year to be guided by him in Sweden through factories of Strängbetong. K-G Bernander played an enduring role in Swedish-Hungarian links in field of concrete technology.

6.3 Contact between organizations and people

As shown in Point 6.1 above, the international professional societies have helped in great measure the dialogue and collaboration between Swedish and Hungarian engineers. Additionally there were exchange programs based on cultural agreements. Within the framework of the Swedish-Hungarian cultural program and organized by the Svenska Institutet, Stockholm, Author1 had the opportunity to study the achievements of concrete research and technology in Sweden.

It is only possible to list the visited firms and institutions:

Kungliga Tekniska Högsolan, Instutitionen för Brobyggnad; Gatukontor Stockholm; Statensvägverk Brosektionen; Strängbetong AB, Stockholm; Cement och Betonginstitut (CBI), Stockholm; Chalmers Tekniska Högskola, Institutionen för Konstruktionsteknik, Betongbyggnad, Göteborg; Gatukontoret, Göteborg; Lunds Universitet, Byggnadsteknik II.; Strängbetong, Kungsör, Sunbyberg.

There was also opportunity to visit construction works of concrete bridges and buildings.

To demonstrate how many Swedish concrete specialists can become acquainted *one* visitor in a short time, a list of names is presented here:

Olof Berge, Karl-Gustav Bernander, Peter Björlin,, Ulf Bjuggren, Stune Brodin, Christer Cederwall, Lennart Elfgren, Georg von Gegerfelt, Hjalmar Granholm, , S. Gustavsson, Kent Gylltoft, Björn Hellström, Arne Hill, Arne Hilleberg, Åke Holmberg, Hans Ingvarson, Hans, Sven Kinnunen, Åke Kjelsson, Lars Larsson, N. O. Larsson, Tommy Liefvendahl, Bernart Lindbladh, Bertil Lindeberg, Sten Ljungqvist, Mogens Lorentsen, Anders Losberg, N. Lyckeberg, T. Muregård, T, Sven Nilsson, Lars Östlund, Arne Rinkert, Johan Silfwerbrand, Ralejs Tepfers, Valentinas Vilkenas, Kurt Wenger, Lloyd Willberg.

The following lists a limited number of Hungarian experts who went on professional tours in Sweden during the last decades of the 20th century.

Árpád Apáthy, Ernő Burka, László Enyingi, László Fazekas, Péter Fodor. György Deák, Mihály Gyenge, Gyula Holéczy, Mihály Huszár, József Illésy, Lajos Imre, Géza Jolánkai, Endre Juhász, Pál Kékedy, László Kolin, Miklós Kozák, Gyula Márkus, Miklós Márkus Gábor Medved, Miklós Merényi, Miklós Molnár, József Novoszáth, András Protzner, Kornél Rados, László Rákóczy, Károly Sághi, Andor Sasvári, Ferenc Sebők, Antal Szabó, Zoltán Szigyártó, József Thoma, Lenke Tóth, Irén Túri, Imre V. Nagy, Tibor Váradi, György Varga.

7. CONCLUSION

The survey presented here attempts to provide an impression on a range of connections between Hungary and the host country of the *fib* Symposium, Stockholm 2012. We believe it is worthwhile to learn more about each-other and hope that we provide interesting examples to all participants of the event. In this spirit we hope that this meeting helps to highlight the achievements of Sweden and of other *fib* group countries, improving the better understanding and friendship of all participants.

8. ACKNOWLEDGEMENT

It is already a tradition that Eng. Dr. L. Bajzik (Kecskemét) contributes very much to the introductory article of our journal and Mrs. K. Haworth-Litvai (Sydney) revises the text. We express our gratitude to both of them. Further thanks are due to tekn. lic. G. Kalmár (Gävle) for his conscientious help.

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REINFORCED CONCRETE FRAME OF THE PAINT SHOP
MERCEDES-BENZ FACTORY COMPLEX IN THE CITY OF KECSKEMÉT, HUNGARY

József Almási – László Polgár – Pál Sterner – Péter Varvasovszky

The article is presenting the modified structural frame design of a paint shop which was comprised of heavy precast concrete elements with moment bearing joints at the time tendering. The modified structural design provided a lighter structural framing system accommodating the application of the locally more employed construction and production technologies, meeting all the requirements of the tendered technical requirements by using lighter precast elements, composite structural framing system, more reliable construction methods for moment bearing connections accommodating much greater manufacturing and erection tolerances for the precast elements, therefore, reducing the construction cost and time.

Keywords: reinforced concrete frame, precast concrete elements, moment bearing connections, modified construction technology, optimized construction cost

1. INTRODUCTION

Mercedes-Benz invested 800 million Euros to build a new car manufacturing plant located in the city of Kecskemét, Hungary. The total built in areas of buildings are 300.000 m^2.

One of the technologically most complex buildings of the new car manufacturing plant is the paint shop.

In the conceptual phase of the project the architectural design was prepared by the German Kohlbecker Ltd. and the Hungarian CÉH Co. The conceptual structural design was the work of the German BKSi. In the later stage of the design the structural drawings for permit and construction were prepared by the Hungarian Sterner Ltd.

The bid of KÉSZ Co. won the tender. The entire construction of the project was awarded to KÉSZ Co. – as general contractor - and CAEC Ltd. – as consulting engineers – was hired by the general contractor to provide alternative structural solution for the framing of the tendered paint shop with restriction as follows:

- the alternative structural solution has to preserve the architectural and technological content of the tender document.
- the owner explicitly asked for normal reinforced concrete structure excluding any pre/post tensioning option.
- the geometry of the building must remain and the alternative structural system has to bring cost, time and constructability benefits to the owner and the general contractor.

CAEC Ltd. proposed a composite reinforced concrete structure comprised of precast concrete elements without pre- or post-tensioning and cast in situ concrete options by including the future structural sub-contractors, namely PAMINVEST Co., BAMTEC Ltd. and DVB Ltd. The newly assembled team of structural consultants, subcontractors worked very closely with the consulting team to develop optimal and practical structure for the paint shop to achieve the desired outcome of cost and time saving. It is important to note here that the authors would like to express their appreciation to all participants who were part of the team and contributed to the successful completion of the paint shop. The meticulous listing of the participants of the design team and their contribution is important due to another reason, namely it accentuate the broad range of international cooperation, the application of the common Eurocode standards which was unquestionably the most important aspect of the success of the design work.

2. GENERAL DESCRIPTION OF THE STRUCTURAL SYSTEM OF PAINT SHOP

The planar dimensions of the paint shop are 63.0×333.0 m and the orthogonal grid spacing determining one panel dimension bordered by four columns is 12 × 10 m generally.

Due to the length of the building the structural consultant created four independent structural units having length of 70 to 100 m (*Fig.1*). These planar units were separated by expansion joints running along the full short length of the building.

There are three functionally distinctive floors in the paint shop. The first level located around 7.5 m above the ground floor was constructed as precast and cast-in-situ composite floor slab covering the entire foot print of the building.

The second and third floors have structural steel primary framing (beams and columns) and cast-in-situ concrete slab is the secondary framing element supported by the primary framing. It has to be noted that the third floor does not cover the entire foot print of the paint shop. The finished structural floor elevations of the second and third floor plates are 20 m and 28 m above ground floor elevation.

The scope of the article covers only the structural aspects of the first floor slab (+7.50 m) of the building as shown in *Fig. 2*

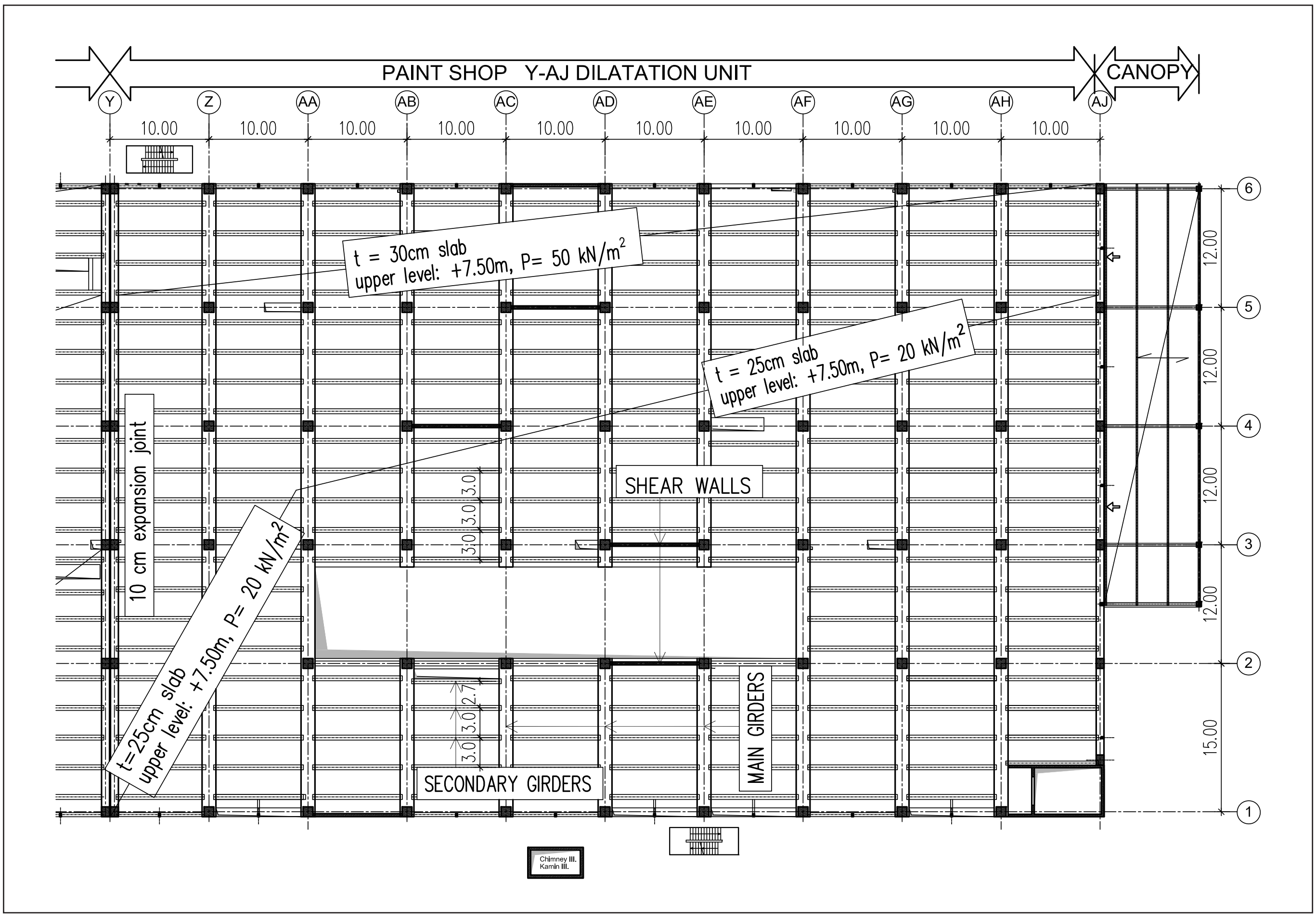


Fig. 1: *Layout of a structural unit on +7.50 m level*

marked with dotted line wire frame. The structural description of the two level steel frame is beyond the scope this paper. We are only referring to the structural steel frame because it is supported by the reinforced concrete frame of the first floor.

The magnitude of service (un-factored) level live load of the first floor is p = 20 kN/m² generally. There is a 15 m wide band of the first floor where the magnitude of live load is p = 50 kN/m². This high intensity load was due to the location of the painting robots. Their operational requirement allows very small magnitude of slab deflection.

3. DESCRIPTION OF THE ORIGINAL STRUCTURAL CONCEPT DESIGN

The original structural concept envisioned the application of a moment bearing sway type framing system for the paint shop which was assembled of heavy precast structural components.

The architectural and technological requirements allowed placing shear walls running only parallel to the longitudinal axis of the building. Therefore, sway type moment bearing multi-level (2D) frames secured the cross directional stiffening of the building and the frame action was perpendicular to the shear walls. This combined system provides adequate resistance against lateral forces induced by wind, seismic force corresponding to maximum horizontal acceleration (a_{gr} = 1 m/s²). There was no special consideration given to the vertical acceleration component of the seismic – Raleigh type – waves. The foundation consultant selected pile foundation for the building due to the uncertain load bearing characteristic of the soil (Loess type) and the magnitude of the column forces.

The long foundation piles (6, 8 or 9 piles in one group) were connected at top with large cast-in-situ reinforced concrete pile cape. A typical cross section of the building is shown in *Fig. 2*.

The characteristic dimensions of the precast components from the tender document are as follows:

Columns: 1.00 × 1.00 m or 0.80 × 1.00 m cross section, G = 25 t
Main girders:1.40 × (1.55+0.25) m, G = 60 t.
Secondary girders: 0.50 × (0.75+0.25) m or 0.50× (1.00+0.25) m, G = 11.2 or 14.0 t.

The floor slab comprised of 8 cm thick precast cradle panels forming elements and 17 cm thick cast-in-situ reinforced concrete topping.

Fig. 3 shows the typical cross section of the building slab located at elevation +7.50 m and *Fig. 4* shows the connection details.

The moment bearing frame connections were designed by using GEWI bolted connections and the gaps between the intersecting members were injected with speciality grout according to the tender document shown in *Fig. 4*.

4. INTRODUCTION OF AN ALTERNATIVE STRUCTURAL SOLUTION

CAEC Ltd. as structural consultant with a group of Hungarian subcontractors collaboratively worked out a new structural design which provided structural solution for reducing the construction time, cost, adapted better to the Hungarian construction practice and resources than the sturctural scheme provided in tender documentatiton. Priority requirement was to reduce the weight of precast concrete main girders because

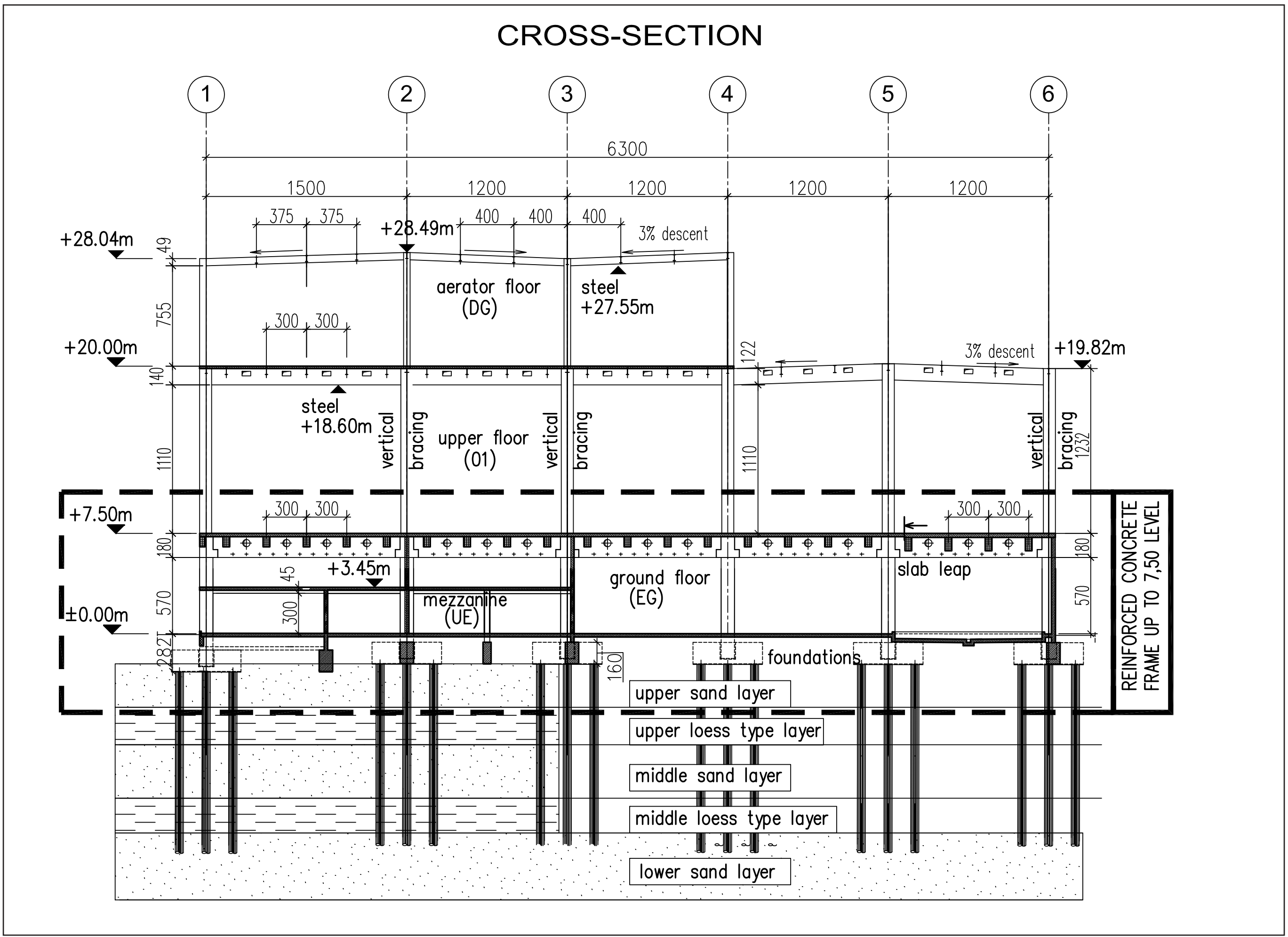


Fig. 2: *Cross section of the building - schema of the structural frame (detail from tender document)*

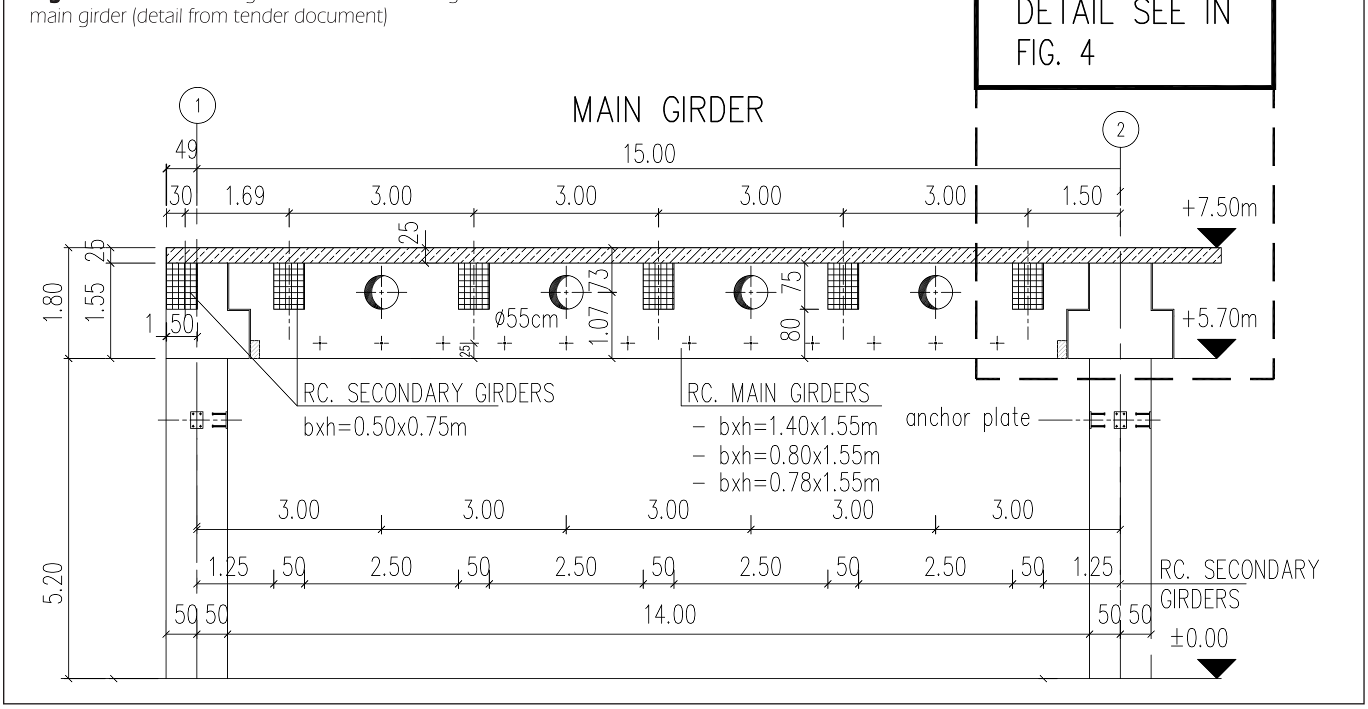
the weight of originally designed girder was so excessive that it created an insurmountable logistical problem with the transportation and lifting.

The new corrected structural design kept the concept of the tender meaning the rigid two way system concept and it changed rather the building method of structural frame of the paint shop. The dimensions of the primary frames remained unchanged because technological design issued with the tender document.

According to the new structural version the design team used Partially Precast System.

This means a composite system, which comprises precast elements and cast-in-situ additional structural enhancement which will be explained in detail later in the paper.

Fig. 3: *Details of building cross section according to tender document – side view of the main girder (detail from tender document)*



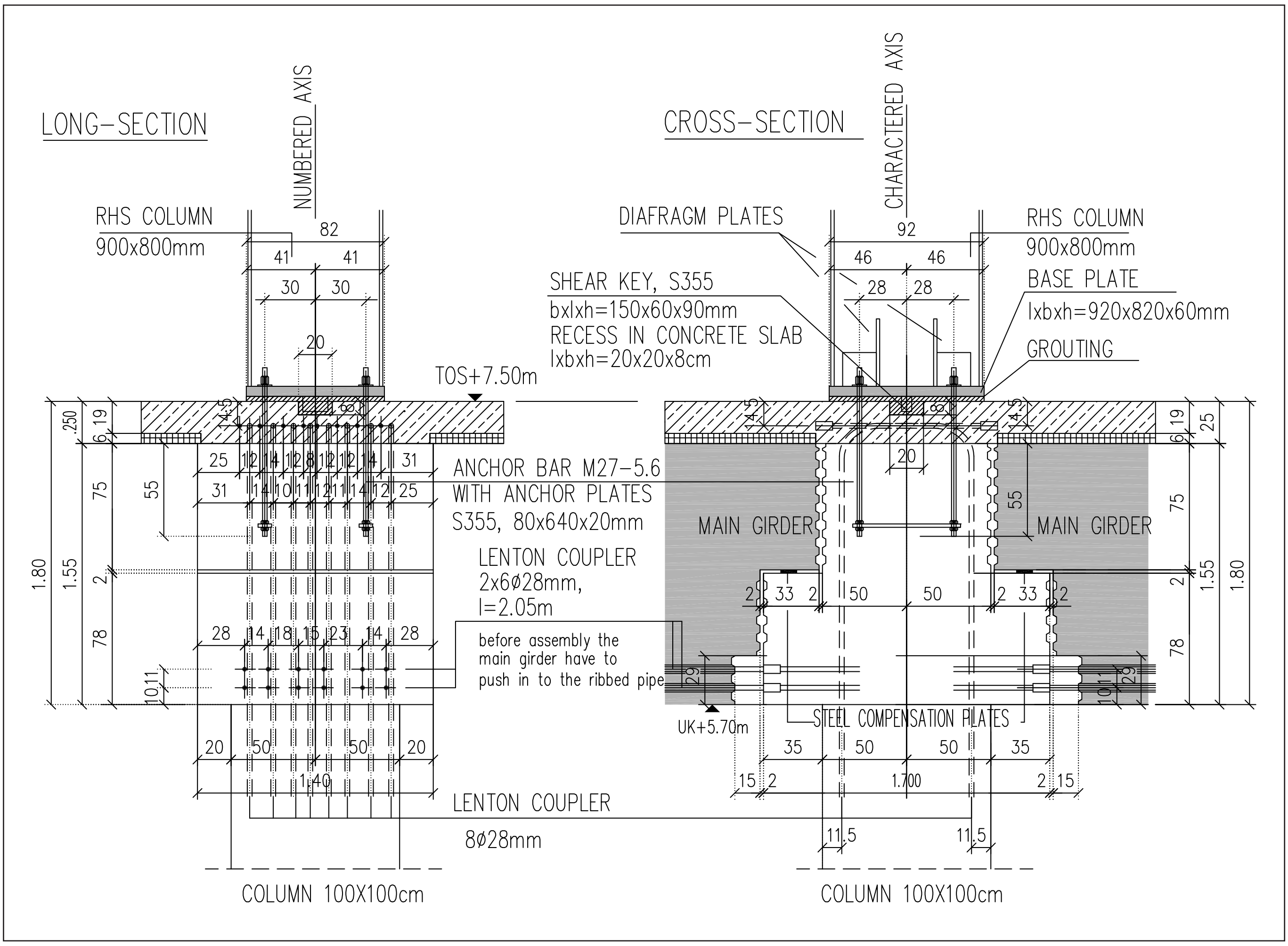


Fig. 4: *Column-main girder rigid connection detail (detail from tender document)*

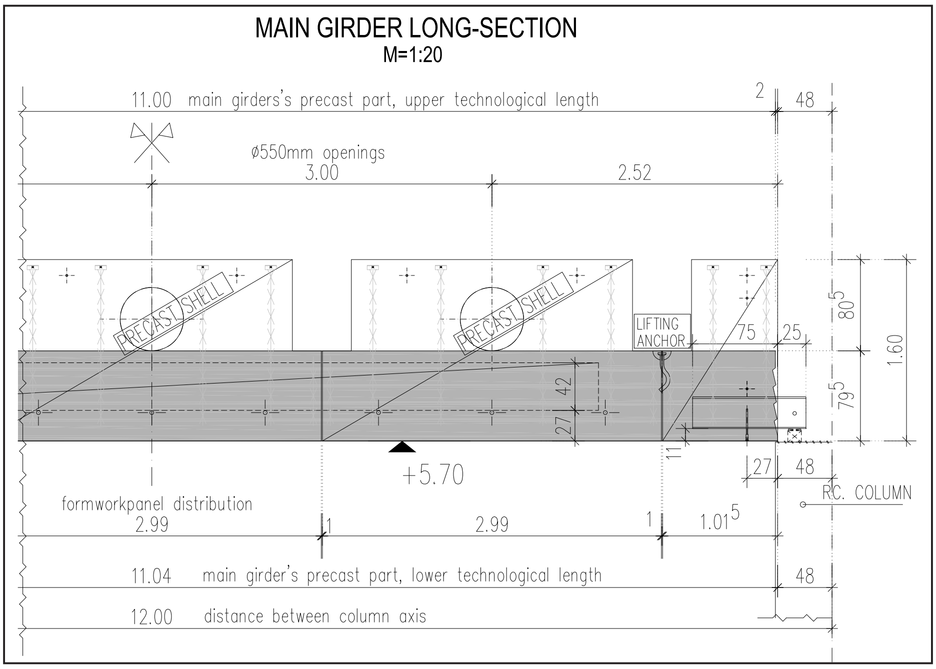


Fig. 5: *Part elevation of the main girder at column support*

With this method the weight of the precast concrete main girder was substantially reduced and the use of the precast formwork enabled the contractor to speed up the schedule timing greatly. The columns were poured on site and the upper 80 cm high part of the main girders was also a cast-in-situ reinforced concrete. This condition is shown in *Figs. 5 and 6*. The new structural detail allowed the construction of more reliable and constructible rigid moment bearing beam/column connections than the tender document provided structural design.

Fig. 6 shows the cross and longitudinal sections of the alternative beam/column connections.

Geometrical modification was only possible for the foundation. The new structural design favoured the application of the cast-in-situ columns as opposed to the precast columns by the tender document. With the construction of the cast-in-situ columns the contractor had the opportunity to avoid the construction of the cup shaped sockets for the connections of foundation and precast columns. The cast-in-situ option provided great opportunity to reduce the dimensions of pile caps and to make the connections simpler and more reliable without changing anything in the spacing and numbers of piles which was one of the owner's requirements. The new structural design kept the original dimensions of the main girders (1.40 × 1.50 m), however only the lower half part of the girder is fully completed and the remaining part has only outer shell to facilitate the on-site concrete pouring. With this arrangement the weight of the main girder became manageable for transportation and erection. Fig. 8 shows the end segment of

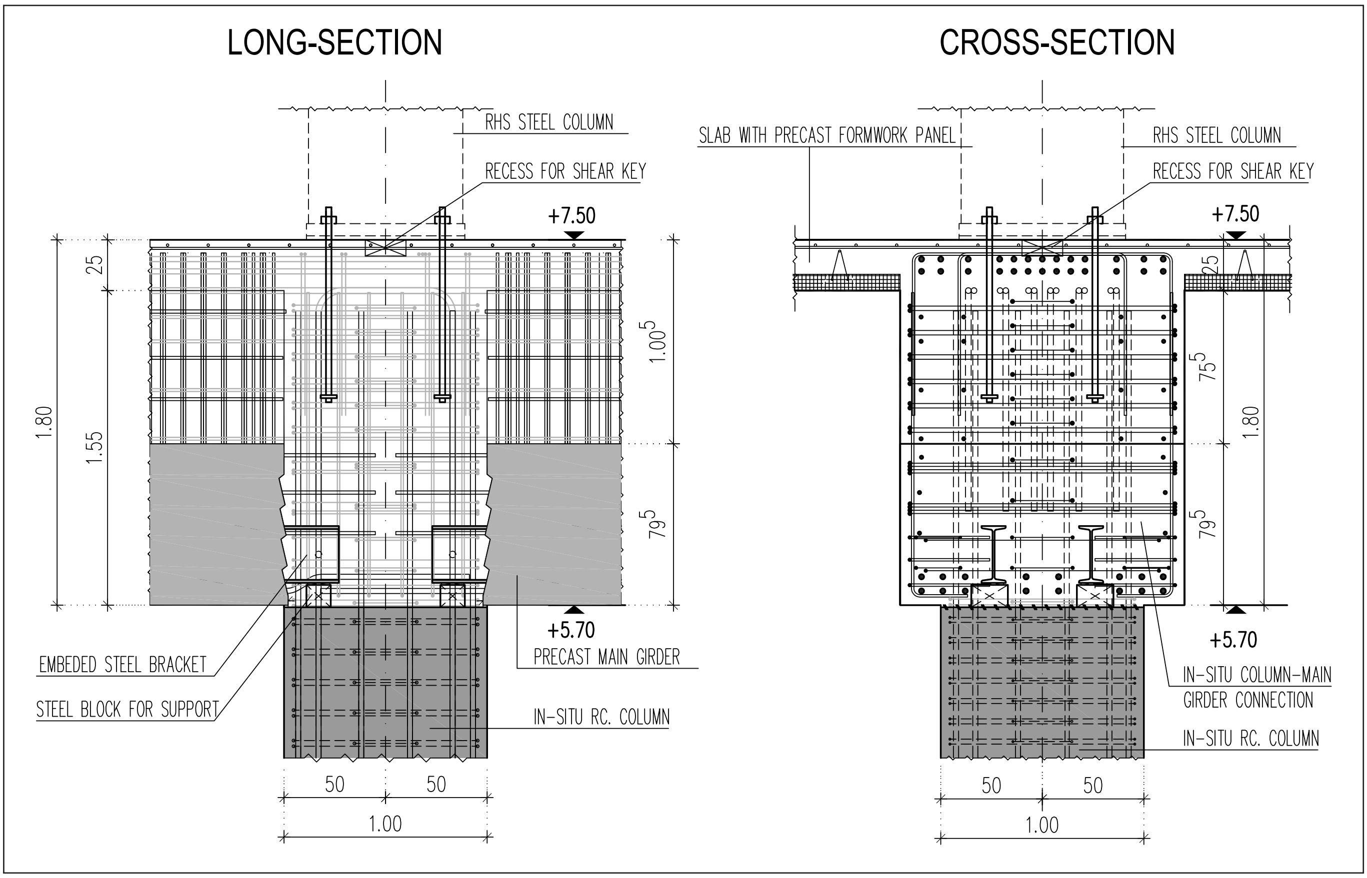


Fig. 6: Connection details of column and main girders

the main girder as it was stored on the yard of the manufacturing facility. The exact cross sectional dimensions of the main girder are shown in *Fig. 7*.

The precast concrete shell of the main girder enabled the contractor to get easy access to the column / girders intersections and it also allowed the simple connection between the secondary girders framing into the main girder and it allowed the on-site concrete pour without any further formwork construction. In order to make the precast bottom segment of the main girder lighter the consultant designed longitudinal

Fig. 7: Cross sections of the main girder

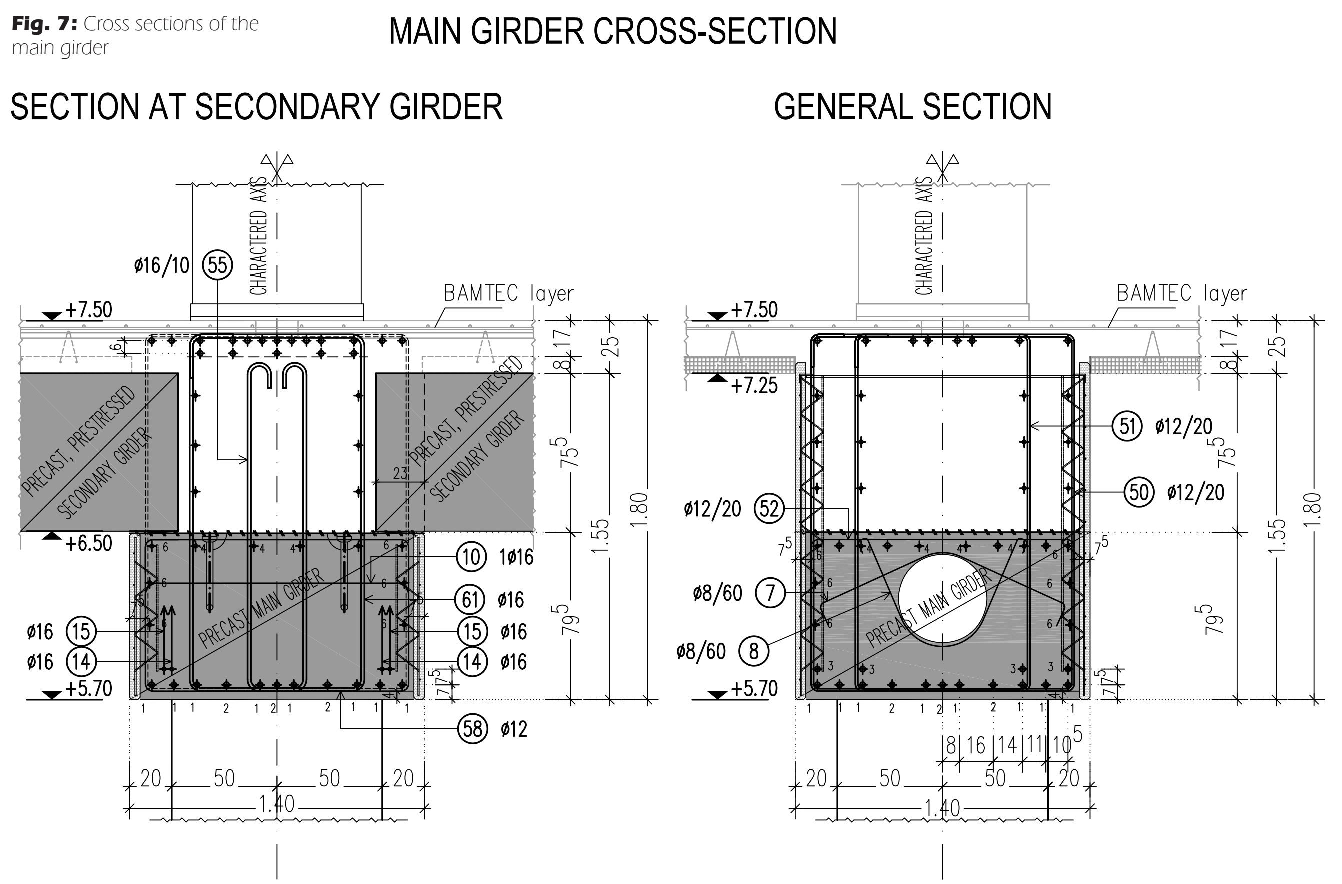


Fig. 8: Precast concrete main girder in the manufacturing yard

Fig. 9: 100 cm deep secondary girders with upper reinforcement before assembling on site

Fig. 10: Precast structure of the slab before concreting with temporary shoring structures

Fig. 11: Placing BAMTEC reinforcement on precast formwork elements

placed pipe duct. Other circular ducts were perpondicular to the axis of the main girder allowing the free crossing of various electrical and mechanical installations.

The extension of the vertical legs of shear and torsion stirrups extended above the precast bottom part of girder and enabled the composite action between the precast and the cast-in-situ segments along the horizontal cold joints running along the entire length of the main girder.

The presented solution was the first case used in Hungary according to the present knowledge of the authors.

The formwork and the construction expediency did not allow designing traditional short cantilevers providing support for the main girders. Therefore, the design team used embedded I shaped steel sections which were projecting beyond the ends of the main girder into the supporting main cast-in-situ concrete columns. In order to assure the formation of the adequate bearing surface the projecting steel beams were placed on solid bearing blocks. The spacing of the column vertical reinforcement made it feasible to avoid the interference with the bearing blocks and the steel I shaped short beams. The precast ends of the main girder have series of grooves and projections designed to act as shear keys. Therefore, the shear force transfer has two path ways. One of them is the shear capacity of the embedded I shaped steel beams and the second the shear key formation at the ends of the precast girders. There was a coordinated effort of all team members that connection between the precast main girders and the cast-in-situ concrete columns to meet the stringent requirement of the on-site construction tolerance.

Secondary girders were either 75 cm or 100 cm deep, depending on the live load. They were supported by the precast part of the main girder (*Fig. 7*). The ends of secondary girders were indented (*Fig. 9*) in order to achieve better connection between precast and in-situ concrete. The upper reinforcement bars of secondary girder were already placed inside the stirrups before assembling (*Fig. 9*). As the secondary girders were already placed, upper reinforcement had to be pulled properly above the main girders.

Due to the magnitudes of seismic action induced horizontal loads acting on the rigidly connected column/girder connections the Code EN-1998 requires a substantial amount of shear reinforcement in order to attain the adequate level of two way shear and moment bearing capacities of the connections. This reinforcement is shown in *Fig. 6*.

The density of the shear reinforcement caused difficulties during the rebar placement but with correct detailing and keeping the set placing sequences the contractor managed to install the correct rebar cage without any problem in exemplary manner.

The main girders had to be supported by temporary shoring system having load bearing capacity of 900 to 1000 kN because the half dimension precast girder did not have the load bearing capacity to carry the self-weight of the cast-in-situ concrete including the weights of precast secondary girders, precast formwork elements, and the cast-in-situ concrete slab topping.

Fig. 10 shows the temporary shoring of the main girders during construction phase. The temporary shoring structures were placed on temporary foundation made of precast concrete elements.

The floor slab formwork was made of stay in place precast thin slabs and the cast-in-situ concrete topping having thickness of 17 cm was reinforced with rolled out carpet reinforcement patented by BAMTEC Ltd. (*Fig. 11*). This patented reinforcing arrangement can assure that the optimal

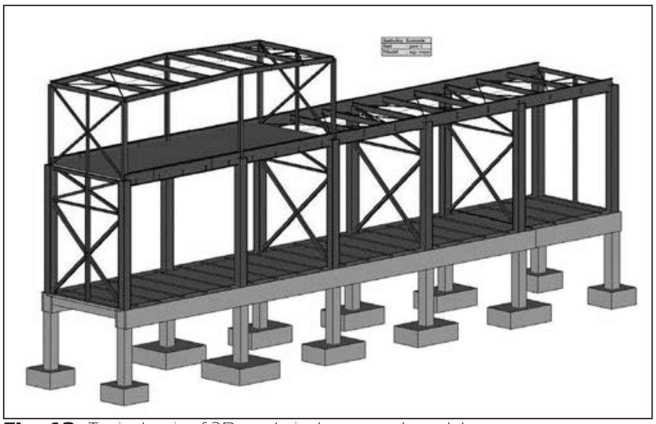


Fig. 12: Typical unit of 3D analytical structural model

amount of reinforcement is placed in the slab and BAMTEC method is vastly reducing the installation time of the slab reinforcement providing substantial saving on the material and labour cost sides.

5. DESCRIPTION OF THE ANALYTICAL STRUCTURAL MODEL

The structural consultants used 3D finite element model which was created by the structural engineers of Sterner Ltd. during the permit phase. They used the AXIS VM 9.0 finite element software for setting up the analytical model of the building. Each building part separated by expansion joints was handled as separate and independent structure during the analytical phase of design process. The structural engineers created precise models taking into account the soil and superstructure interaction during seismic action and modelled the piles as spring supports for the pile caps. The upper two level steel frame was modelled as linear element with the proper restraining conditions like pinned, partially restrained or fully restrained 3D structure. The reinforced concrete part of the building was modelled as shell and slab elements. Under such modelling technique the dynamic properties of the building like eigen-frequencies, eigen-vectors, acceleration and velocity vectors were adequately captured. The structural model did not venture into the time history analysis or even more sophisticated realm of the analysis like plastic behaviour of the system components or site specific seismic data application.

The magnitudes of the spring supports for the pile caps were given by the geotechnical consultant (Smolczyk & Partner GmbH) specifically related to the physical properties of the loess type soil matrix. The models considered that the connections between the pile caps and cast-in-situ concrete columns are fully restrained and rigid.

The analytical model of the 1st floor slab (+ 7.50 m) comprised identical size rectangular shell elements supported and stiffened by the main and secondary girders with full cross sectional contribution to the structural stiffness.

Fig.12 shows a segment of the analytical model located between two grid lines as typical unit of the analytical model.

Modelling of the superstructure steel and of the reinforced concrete frames was not extremely challenging task. However, the correct analytical capturing of the foundation comprising piles and connecting head caps left many disputable issues

Fig. 13: Finished structure inside

because of many uncertainties arising from the soil/structure interaction. It needed considerable degree of engineering judgment from all sides of the consulting engineers to arrive to a mutually acceptable conclusion. Eventually it became necessary that CAEC engineers corrected a few aspects of the design during the construction phase.

The horizontal and vertical spring bedding constants of the individual piles are different under static and dynamic loading conditions. Addition to this the load bearing characteristics of individual pile is grossly influenced by the group action of piles related to the pile spacing, number of piles connected to one pile cap.The pile arrangement and other site conditions are influencing the eigen-frequencies of the building frame and in return the response of the structure (like acceleration, velocity, displacement) for the seismic action.

Therefore, the rigidities of the supports were determined in many steps taking into account the vertical and horizontal displacements of the piles under short and long duration loads, the actual pile capacities under displaced geometry and the response of the superstructure.

The internal forces of the structural components were determined by the 3D analytical model but for the actual member proportioning in house developed softwares were used frequently with the utilization of the Friedrich Lochner software package.

6. SUMMARY AND CONCLUSIONS

In the tender document of the paint shop building of Mercedes-Benz new manufacturing plant the structural engineers envisioned a structural framing system assembled of heavy precast concrete components (columns and girders) rigidly connected together with the application of complicated details and construction methods.

Due to the very short construction time allocated to the project and the capability of the general- and subcontractors we decided to seek out a more practical, reliably constructible structural system meeting all the basic requirement of the tender document and provides benefit for the owner and the general contractor. The new structural frame fully considered the originally designed framing system up to the 1st floor (+7.5 m) uses all advantages of the partially precast concrete and cast-in-situ system and integrates the steel building frame located above the 1st floor without any deviation from the requirement of the tender document considering the full exclusion the possibilities of any additional forces or deflections might be caused in the steel superstructure (*Fig.13*).

Among the chief considerations there were the following items like adverse climatic/weather conditions during the construction, capacity of the available and existing lifting equipment's for the precast concrete manufactures, the availability of large capacity mobile cranes for site erection, improvement of quality of the site work, improvement of the quality of the coordination, scheduling and monitoring of the construction and the assurance of the overall quality of the structure.

In order to reach the above listed goals there was an exemplary cooperation among the stake holders including the manufacturers, erectors, design teams, construction and project managers.

7. ACKNOWLEDGEMENTS

The authors are expressing their sincere appreciation and thankfulness to all participants who contributed to the successful completion of the paint shop building including the original engineering group BKSI GmbH and the construction management group of the owner.

This building project proves again that the design and construction technology aspect shall work in a synchronized way to attain the optimum solution in complex building structure like the present paint shop.

This building is a proof for the international cooperation among all stake holders by using the unified codes, advanced applications of various computer softwares, and project management techniques.

The authors are very grateful for all those organizations and individuals who assisted in the project with their ideas, practical solutions, advices and their diligent works bringing this project to a successful completion.

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STRUCTURES, DESIGN AND REALIZATION OF ÁRKÁD SHOPPING CENTRE IN SZEGED, HUNGARY

András Szabó

The load bearing structure of the new shopping centre in Szeged was built with a combination of precast and cast-in-situ reinforced concrete structures. The difficult planning decisions encountered the possibilities and limitations of precast reinforced concrete structural elements. The article demonstrates this specific manifestation, the constraints of redesign, the difficulties in creating breakthroughs. The problems derived specifically from the fact that the co-operating partners in the planning (from the client of the building through the architect until the installation engineer - all) consider the possibilities and limits of precasting the structural designer's own problem. Regarding the relevant building type, the intervention needs of the emerging structural changes during and after the building due to regular interior remodelling are very specific. In the article, we would like to draw attention to the mutuality of these - seemingly minor – decisions, which in fact influence the pureness of the structure.

Keywords: *shopping centre, precast reinforced concrete structural elements, installation breakthroughs*

1. INTRODUCTION

A new shopping centre was built by ECE Einkauf-Center Szeged GmbH&Co.KG, developed by ECE Budapest Projektmanagement Ltd. in Szeged, Hungary, at the London Boulevard. The department store lies on a site of 23,000 m^2 and its useful area is almost 100,000 m^2. The stores occupy the ground floor, the first floor and a part of the basement level, while the second and third floors primarily provide for the car park.

The construction designer of the building permission plans was Béla Balogh, the plans were supervised by Árpád Dunai, and the adviser of the owner was Prof. Dr.-Ing. M. Fastabend. Our office (System Steel Design Bureau Ltd., head designer András Szabó) made the modified building permit and the final construction drawings of the structure *(Fig. 1).*

2. FOUNDATION ISSUES

Due to the notoriously well-known unfavourable subsoil of Szeged, the choice of the foundation type caused some conflicts. The Hungarian soil experts (B. Vásárhelyi, R. Szepesházi, L. Szilvágyi), being familiar with the area, suggested raft compound pile foundation, while the German experts of the owner (M. Fastabend, M. Kowalów) reasoned against the piles. The structural expert published his observations about the sealing basement block ´"Weisse Wanne" applied by the ECE department stores built in Poland and Germany (Fastabend, 2010), and proposed its adaptation in Szeged as well.

To clarify the features of the soil and the circumstances of the foundation, we studied them carefully in advance, in view of the experiences of previous constructions in Szeged. The low compression modulus of the soft fat clay soil made the team and structural experts precautious in more structural questions, however, we also had to avoid unrealistic overdesign because of the magnitude of the building. To take the bedding factor for sizing the foundation raft, settlement calculations were made (involving B. Vásárhelyi and R. Szepesházi) and projecting its results to the wide foundation raft. The sinkage rate was significant, about 60-80 mm. This possibility could not be neglected from the point of view of the elastic strains of the foundation raft, however, by calculating the final probable building sinkage, the uplift effect of the high groundwater and the lower consolidation of the non-drainaged load bearing soil layer was also highly taken into account.

The basement block built with watertight reinforced concrete boundary constructions has to assure waterproofing without any bituminous sheet („Weisse Wanne"). Assuring the waterproofing of the 80 cm thick foundation raft caused no problem, however we could not agree with the owner's consultant regarding the interpretation of the crack-width.

As the final decision is always made by the client, the compromise was made by accepting the suggestion of the German expert, because originally the concept had been to apply the "Weisse Wanne" solution. This interpretation meant more use of steel for the contractor, but he did not have a choice.

A critical question of the sealing of the foundation raft is the creation of construction joints. By concreting of the foundation raft, the order of the concreting of each field or rather its proper choice reduces the opening of the working gaps caused by shrinkage. However, the planned chessboard order was overwritten by logistic necessity. Contaflex active construction joint band was placed in the working joints, the bentonite coat of which, effected by water, fills the gaps allowing the water to get through *(Fig. 2).*

During the construction the groundwater was continuously pumped, and the maximal permissible water level was calculated based on uplift balance. Under the foundation raft, a drain-system and pumps were installed, the hole of which had to be closed in the foundation raft in posterior.

Fig. 1: Atrium under the rotunda

Fig. 2: Construction joint in the raft

Fig. 3: Column-beam-slab joint in the mall

3. THE SUPERSTRUCTURE

The important aspect at the structure of the department store building was the most possible prefabrication – beyond the functionally adequate raster size, of course. The foundation raft is naturally cast-in-situ. The possibility of prefabricating the columns arose, but the time saved is not so significant with this structural element. The problem was caused by the space need at the joint of the columns and the beams (the depth of the beams were only 65 cm). In case of prefabricated columns more stiffening walls would have been needed to assure spatial stiffness – because of the hinged connections –, which could not be solved without compromising the flexibility of the function. That is why the columns were made monolithically.

The planned structural grid of the three-floor building is 10.0 × 8.25 m. The columns of the building were made monolithically, with fixed end to the foundation raft, creating a frame of more floors with the joists. The prefabricated, 10.0 m span beams were made with conventional reinforcement. In order to avoid cumulative reinforcement on the column capital, the compressed bottom reinforcement (at the brace) was led through the column capital only partly (corresponding to the sizing) while the tension bars completely, and additional tension reinforcement was built in the top concrete. The continuous slab panel lying on more braces was made with core concrete shuttering elements and compound top concrete. We paid special attention to the composition of prefabricated and monolithic structural parts *(Fig. 3)*.

The columns take place in this distribution on the car park floors as well as on the store levels. The shuttering panels with 8.25 m effective span lean on the prefabricated main beams lying in line with the longitudinal axis, on 10 m span. The basic raster had to be changed with individual elements because of functional requirements in more places, which are mainly the areas of the staircases and the lifts, going by the line of the slanted street front. The panels connect to the walls of the slanted street façade with shortening. The floor slab disc is interrupted in the middle axis by the bay of the mall *(Fig. 4)*. Because of the different, trapezoidal slab panels, the panels were created in hundreds of versions regarding size and load bearing capacity.

Fig. 4: Corridors of the mall lying on precast cantilevers

Fig. 5: Scaffolding of the core panels. Column-beam-slab joint before pouring

Fig. 6: Cantilever of the parking deck

Fig. 7: Bridges of the mall

However, cast-in-situ slab fields are connected to the staircase cores in oblique position, in order to allow binding the coupling reinforcement concreted in the walls of the lift and staircase cores built in full height with slip-form construction method. This solution was constrained by the change of the panel type. The slab core (shuttering) elements were planned with conventional reinforcement, according to the basic idea, 8 cm reinforced concrete panel on both sides flat. The three-chord reinforcement truss connected to its lower reinforcement mesh with a top chord compression member holding the upper mat. The truss reinforcement allowed the free lead of the bars in the 17 cm top concrete of the 25 cm thick slab, and the free leading into the sheared zone of the coupling bars folded from the staircase walls. By changing the product the construction of the panel altered, the prestressed concrete panel ensured the needed stiffness and the support of the on-site upper flange reinforcement with solid concrete ribs instead of truss spacer *(Fig. 5).* However, in contrast with the truss reinforcement, the solid rib greatly impede the free lead of the cross rebars. The distribution reinforcement ensuring the composite behaviour of the panels and the cross rigidity could be put, but the opening of the starter pack reinforcement and the needed distribution bars cannot be assembled because of the ribs. Therefore, we had to go without prefabrication in this part, and the field was allocated to cast-in-situ reinforced concrete slab.

On the top floor, the architectural functions significantly differed from the others, transformers, boiler room, ventilating machines were placed here. Due to this change of function, and according to the architectural intention, the column distribution on the top floor was offset with half an axis distance from the lower ones, the columns leant on the midpoint of the beams, which were not possible to solve with prefabricated beams, therefore cast-in-situ beams had to be designed for the lintel, and on the cantilevers protruding out of the raster *(Fig. 6).*

The columns were placed on lintels also above the downward ramp of the basement car park and at the main entrance, which needed trimming in more raster width. The ground-floor discharges were even more complicated, because beyond more floors of slab, also the clinker brick façade cladding loads upon them, not once throughout the 10.0 m height.

4. CHANGE OF THE PANEL TYPE

In the phase of the permission plan, the core (shuttering) elements designed by Béla Balogh were panels made with conventional reinforcement and we planned two floors according to this on the final construction drawings as well. We put polystyrene filling into the monolithic top concrete in order to diminish the dead load. However, in the course of the consultations with the fabricator and the contractor, we came to a conclusion which affected the on-the-spot implementation's need of time favourably. With prestressing reinforcement and steam curing of the core (shuttering) panels, a bigger load capacity can be expected also in the state of fresh concrete of the in situ top concrete, so the number of braces can be cut down. The total weight to be supported did not diminish, but the spacing of the braces meant saving manual labour *(Fig. 7).* Despite the intervening changes in the plans, the building of the structure was finished according to the original schedule. (In fact, the shopping centre opened one month before the proposed date).

However, changing the manufacturing technology caused a partial change of structural model, the consequences of which had to be considered and resolved underway. (For example: the manufacturing width of the prestressed concrete panel was different, the layout had to be redivided, instead of truss reinforcement stiffening ribs, concrete ribs were made, which made the cross reinforcement impossible on the lower flange, the cross rigidity diminished. The slab connections at the elevator and staircase cores could only be guaranteed with monolithic stripes. Last but not least, the pre-tensioning strands are much denser in the panel, and therefore the openings had to be planned more carefully, especially the posterior slab openings had to be thought over better.) The diminution of the useful cross-section of the concrete topping made us think as well at the binding of the prefabricated cantilevers *(Fig. 8).*

5. THE INSTALLATION BREAKTHROUGHS

The allocation of the stores and the (so called rental) areas, the exact type of the shops and their special requirements were known only in a limited way by the time of the construction design. In consequence, as more and more stores were let out, the breakthroughs of the slabs or even of the stiffening walls changed, while the leasing was already in progress, causing special structural problems.

Another peculiarity of the slab panels with prestressing reinforcement demanded also particular foresight from the designer. The load carrying capacity of the designed panels considered the standard load and safety level, accounting into the designed breakings. The panel fabricator provided many different but restricted possibility for posterior attenuating for each field of the panels in a detailed description (Thek, 2010), explained with the quite dense position of the prestressing strands. Statistically analyzing the load carrying capacity reducing effect caused by the breakthrough of each prestressing strand, made it possible to draw conclusion from the remaining bearing capacity reserves about the safety level after the breakthrough.

Most of the intervening changes of the technical solutions were launched by the general contractor, who considered it the designer's evident task to redesign the plans with these modifications. It manifested itself in the radical change of the slab panelling plans (in addition, the prestressed concrete panels were made 1.25 m wide instead of the original 1.20 m wide panels, and therefore the distribution of the elements had to be redesigned, all the particular accessories, all the corner connections changed. The distribution of the slab openings of the rental areas caused significant additional work (beyond the contract). The original plan during the structure-building was based on the installation engineer's data evolved from the documentation of the contractor's tender. The slab breakthroughs needed for this engineering demand were represented panel by panel in the panel consignation, the load bearing capacity of the panels were raised in the ratio of the omitted tension boom, often slab breakthroughs of a whole panel loaded on seemingly standard adjacent panels. This resulted in a significant number of new panel versions *(Fig. 9).*

The appearance of tenants often led to radical changes of engineering demands. On the one hand, unused slab openings were left on the structure, on the other hand, new holes had to be enclosed, sometimes only a few spans from the old one.

The general contractor executed the modifications. From us he expected the services of adapting the structure to the changes of the installation plan. This changes require weakening of the slab making more openings or bigger breakthroughs. To make it with precision and responsibility this task is sometimes even harder than making the original plans. The structural engineer cannot turn his back in these cases, because uncontrolled changes on the structure can lead to reduction of safety level, or even damages in extreme case. The structural engineer is responsible for the safety of the users of the structure (store tenants, customers), who rightly expect to have a safe structure above their head. Thus, we provided technical help by the changes, designed plans, made site inspection gratis, and the general contractor expected it as part of the initial base service.

However, this is not the lesson of the many posterior changes. The original installation plan seems to be based on an immature architectural conception. Regarding the kind of building, it can be foreseen that either before the handover, during furnishing and creating the stores' own internal design conception, or later when the renters might change, even the interior design and thus the mechanical systems change, and slab openings will be enclosed and demolished for new ventilating pipes or cables. In our opinion, these changes cannot be dealt one by one occasionally, because the structural engineer and interior designer, structural and building execution engineer will inevitably get into conflict again and again, since possible changes were not taken into consideration when creating either the original concept, or the used product (prestressed concrete shuttering panel). The fabricator provided (at a later date, when the demand of the owner became obvious) a statistic method based on risk analysis, about how many tensioning strands can yet be drilled through keeping the prescribed safety of the panel. However, this could not be applied even knowing every inch of the structure. So how can an engineer, dealing with only one given retail unit, use it?

The nearly 200 m long building consists of three dilatational units. The structure is connected by shear bolts at

Fig. 8: Cantilever on support

Fig. 9: Before handover

the expansion joints, because stiffening walls were only put on one side of the expansion joints, according to the demand of the owner (each m^2 surface is to be rented out).

The function of the building creates special requirements as well. The above mentioned problem, namely that the installation plan cannot be final by the time of building construction, derives from the timing of the contracts with the renters, but neither the designers, nor the owner can help it, because new renters will come during the whole life of the establishment. The solution could be an all-purpose vertical pipe string system and a renter service corridor the line alignment of which makes it possible to serve a flexible layout system. Based on a summary of the experiences gained so far, the possible rental types could be defined. Such a solution could guarantee avoiding solutions complicated by compromises neither at the first arrangement, nor by later changes.

A similar problem complicates the placement of the stiffening walls, because, lacking an adequate system, the owner does not want any obstacles for the arrangement of the possible rentals. Even the net layout occupied by the stiffening walls was considered a painful loss for the owner, so we had to accept the smallest possible stiffening wall system. These could be placed principally on the façade walls, and „exceptionally" some walls along the expansion joint. The same wall serves the stiffening of two adjacent units, and both units were connected by shear bolts *(Fig. 10).*

6. THE ROOF

The architectural conception to put the public car park on the roof slab and the slab underneath proved to be disadvantageous from the point of view of natural lighting of the mall, therefore at the middle nave and at the centrally placed rotunda, the thermo insulated second-floor slab was broken upwards as a reinforced concrete wall, and the so formed basilica broke through the roof slab, structurally completely partitioned from it. These reinforced concrete walls, in their whole length, lean on bracket ends. The cylindrical walls of the rotunda, due to architectural difficulties, were put together from prefabricated panels. On the top of the reinforced concrete walls, steel and glass-steel roofing was built *(Fig. 11).*

The cylindrical walls of the rotunda were planned with precast flat reinforced concrete elements connected to each other with outgoing binders along the two edges and concreted. The slightly segmental concrete wall was coated with curved plasterboard. The precast rotunda elements are held together with on-site concrete cornice, on top of which steel beams are laid on concreted steel bearings to support the glass roof.

Almost expectedly, the difference in the measure tolerance at the connection of the reinforced concrete structures and the steel constructions needed to be overcome. The most significant difference showed up at the six-meter extent glass penthouse over the entrance from London Boulevard. The cantilevered canopy ended up on the wall of the ramp leading to the top, which is not stiffened with slab, and this architectural handicap could not be corrected in the phase of implementation. The mooring points of the penthouse running on ascendant floor fell on different heights of the wall, which moved when the concrete was laid down.

The joining elements of cast-in-place concrete structures and connecting steel structures often cause problems, not only because of the different allowance of the two structure parts. Inaccurate measurements within standard limits, meaning not more than 10-20 mm, can be compensated with long hole bolt connections or spacers. However, here we had to deal with bigger, 100-150 mm differences, and the worst thing was that

Fig. 10: *Dead load reducing light fills between the ribs of the panels*

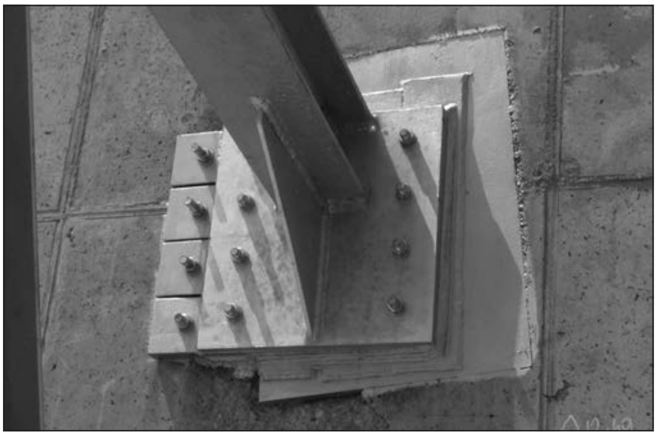

Fig. 12: *Penthouse cantilever corrected joint*

Fig. 11: *Rotunda compiled with precast wall elements*

Fig. 13: *Penthouse cantilevers*

Fig. 14: Second floor covered car park level

in several cases the weld plates sank slantwise in the concrete, demanding the most artistic corrections in order to be able to put the canopy cantilever into place *(Fig. 12)*.

The correction had to be planned from point to point, based on data recorded with three dimensional sizing, a kind of photogrammetry, and given to the locksmiths „waiting on the scaffold". Our original linking solution had been more elegant, but the owner rejected it due to financial reasons *(Fig. 13)*.

According to the original idea, the anchor bolts would have been concreted. It had been a successful solution in various cases, although it causes extra expenses, but it cannot be compared to the work input and time loss of patching. The bolt packet has to be led through the shuttering raft (the damage of which causes the extra cost), but the shift during concreting is only a couple of mm.

The concreting of the plinths of the steel frames above the mall was not more accurate, either. Even 60 mm height difference occurred between the steel plates receiving the right and left legs of the frame, or the plate was simply missing. On top of the reinforced concrete wall however, we could not plan screws glued into borehole (anchoring), since there is not enough lateral distance for building in anchors with adequate carrying capacity.

7. EXECUTION

The concrete technology design of the monolithic reinforced concrete structures was planned by István Zsigovics. Both concrete plants in Szeged took part in the concrete production, because neither of them could fulfil alone the daily amount of concrete needed. There were hardly any structural problems due to the quality of concrete. Concrete technology played an important part from the foundation raft to the floor panels. We created a carefully planned system of joints to allow lengthwise changes for the surface of the car parks' reinforced concrete floor panels lying on the top thermo-insulation, partly shaded, partly exposed to the sun.

Defining the gap size of the joints the most important point of view was that the movable joint aperture ratio cannot exceed the 25% of the gap width, because the waterproofing of the joint sealing material is only be guaranteed under this percentage *(Fig. 14)*.

Due to the fast-paced construction timing, reinforced concrete technology caused the greatest challenge, when post-controlling showed that the concrete of the column planned as C30/37 did not reach the planned strength,. We made long calculations to find out if the reduced carrying capacity of the column can bear the local load. Assuming that a less strong concrete has a lower modulus of elasticity as well, it will show less resistance from the columns of the same group, at establishing the horizontal forces. Several columns could be accepted with this theory. Besides, we could not avoid the strengthening of some columns with steel stirrup, placed within the planned outline, after graving back from the column.

Due to the above mentioned the structural designer's site inspections – unlike a typical construction process - lasted almost until the handover *(Fig. 15)*.

8. CONCLUSION

It follows from the function of the mall that, in some stores already during the constructions, but with later changes

Fig. 15: Façade before handover

of tenants for sure, new requirements can come forward, which differ from the original engineering programme. As a consequence, new openings must be created on the slab instead of passages of drains or ventilating pipes created during planning, while the old ones will be covered, but these slab parts cannot take part in the load bearing any more. Such remodelling cause reduction in the nominal carrying capacity of the slab in various cases, and it cannot be excluded that they repeat several times during the lifespan of the building. The problem of reduction in carrying capacity shows up more definitely by the precast panels, which ensure less the cross distribution than the cast-in-situ reinforced concrete slabs. This often demands deliberation beyond the structural designer's competences, since any reduction of the working slab section reduces the useful carrying capacity of the deck slab. If there is no reserve carrying capacity of the same extent in the slab, it has to be strengthened. The expenditure is often not commensurate with the benefit, but the structural designer cannot influence the interior design, which is often based on company standards. Rational thinking would suggest that reducing the useful load with some per-cents will not influence the usage safety, but the structural designer cannot overwrite the values required by the standard. Two possibilities must be considered to solve the problem. First, it is not useless to involve the structural designer in the decisions of the architectural programme, and second, the planning programme of the mall must be created with a system approach which takes into account also the statical viewpoints of any further remodelling.

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CONSTRUCTION OF A MOTORWAY BRIDGE IN NITRA

János Barta

The R1 motorway, which until September, 2011 ended at the city of Nitra, branches off from the D1 highway near Trnava. The next section of R1 towards Banska Bystrica – including the section passing by the city of Nitra from the south – was constructed as a PPP project. A-Híd Co. Ltd. took part in this work as the contractor for the largest, almost 1.2 km long structure, i.e. bridge No. 209, which consists of two separate structures, and was constructed by two different methods.

Keywords: motorway bridge, incremental launching, balanced cantilevering, prestressing

1. INTRODUCTION

One of the most significant works to be performed by A-HÍD C. Ltd. in 2010-11, which was simultaneously its first real challenging work ordered from abroad, was bridge No. 209 in Nitra. The first steps had been taken as early as in December 2008 when Granvia Construction s.r.o. approached A-HÍD C. Ltd in connection with the construction of a substantial bridge. A-HÍD owed the invitation to its past history and considerable references well-known in the Czech Republic and Slovakia as well. The negotiations, which were needed to clarify numerous technical and financial issues, began soon. As a result of these negotiations, which became successful in August 2009, an agreement was reached on the construction of the bridge in a value of EUR 30 million followed by the conclusion of the contract in early 2010.

2. THE PROJECT

The four separate sections of R1 motorway were implemented as a single PPP project in cooperation of the Government of Slovakia and Granvia Construction. The operating period of the PPP project is 30 years, i.e. the State has to pay operating charges to Granvia and the contractor undertakes a guarantee for the construction works for that period. The overall length of the four sections is 52 km, including 84 bridges in a total length of 6843 m, of which A-HÍD constructed 1166 m. The deadline for completion was 28th September 2011.

3. DESIGN

The substructures and the prestressed concrete superstructures of bridge No. 209 were designed by Dopravoprojekt while the temporary structures (necessitated by the special technologies) by the Technical Department of M-HÍD C. Ltd.

According to the original conception the complete bridge would have consisted of three separate dilatation units lengthwise. The superstructures of the first two units would have been multispan RC girders consisting of a slab with two prestressed concrete webs underneath. The superstructure of the third unit would have been a multispan hollow box girder strengthened by internal post tensioning and extradosed cables as well.

Fig. 2: Cross section of the first dilatation unit (DC1)

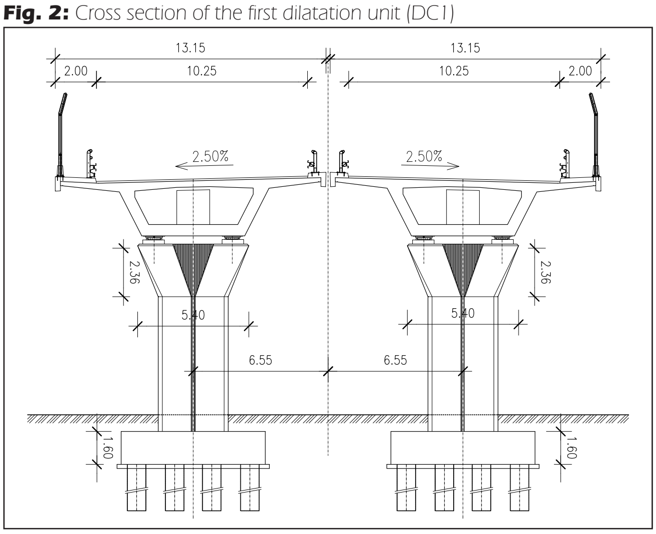
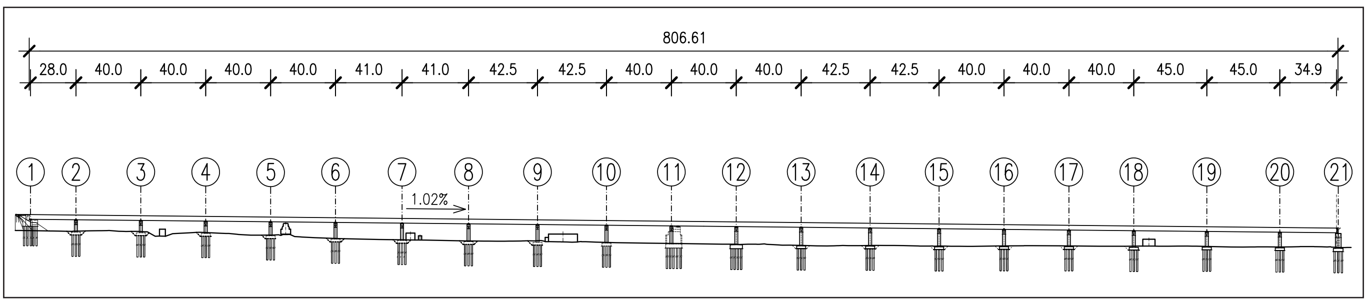


Fig. 1: Side view of the first dilatation unit (DC1)



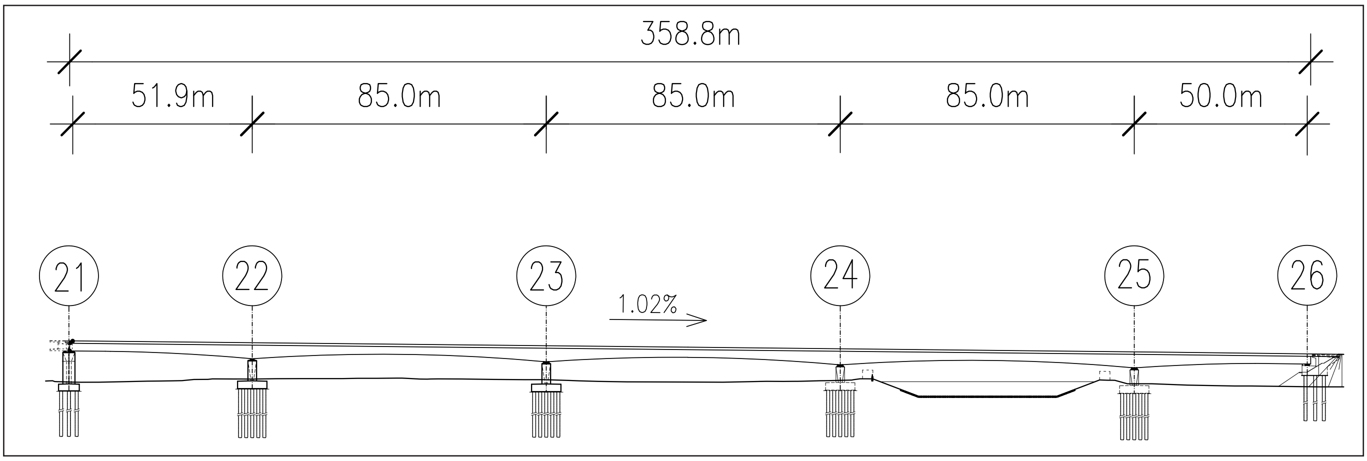


Fig. 3: *Side view of the second dilatation unit (DC2)*

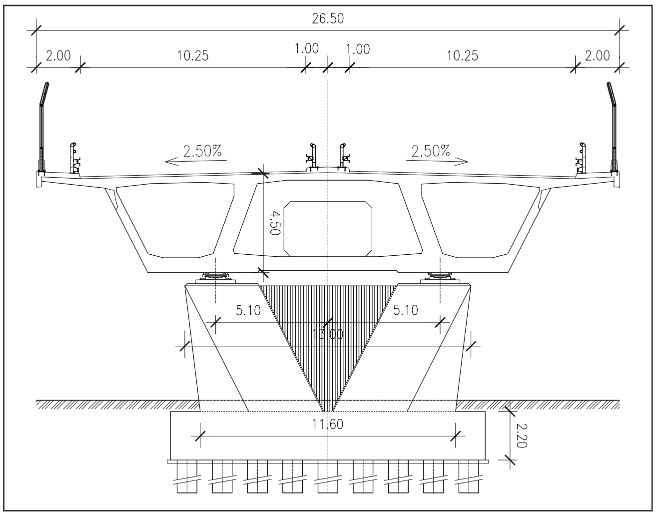


Fig. 4: *Cross section of the second dilatation unit (DC2)*

Later, because of constructability, speed of contstruction and financial reasons the design has changed to two dilation units (called DC1 and DC2) with two different types of superstructures, and two totally different methods of construction.

4. DESCRIPTION OF THE STRUCTURES

The final design of the bridge-structure was significantly influenced by the site conditions. The bridge crosses a railway line, a main-road, various in-town roads, streets and the river Nitra. A part of the R1 expressway itself is situated in the industrial zone of the town with a number of plants. Within the given locality the precondition was to minimize the land required, which resulted in increasing the bridge spans. Due to this reason the bridge was divided into two separate structures with different length of spans.

The DC1 section *(Fig. 1)* consists of two separate, parallel structures, both leading the traffic of one direction of the motorway. Their superstructures are single-cell hollow-box girders with the height of 2.67 m and the width of 13.15 m *(Fig. 2)*. Their spans vary from 40 to 45 m. From the structural point of view the superstructure represents a continuous 20-span girder.

The DC2 section *(Fig. 3)* has a compact structure leading both directions of traffic through. Its cross section is formed by a 3-cell structure with a haunch above each pier. Here its height is 4.50 m, while in the middle of the spans 2.80 m *(Fig. 4)*. The total width is 26.50 m. The lengths of the spans are generally 85 m, but they are 50 m in case of the two side-spans. From the structural point of view it represents a 5-span continuous girder.

Fig. 5: *Common pier 21 with expansion joint*

At each pier the superstructure rests on a pair of bearings. The result of this is an indirect support of the outer webs. For the transfer of resulting shear forces at the diaphragm it was necessary to use transverse bonded tendons, which were prestressed gradually in two stages, depending on the intensity of shear forces in the diaphragm area.

The forms of the substructures of the bridge were influenced by the lack of space for the foundations between the river Nitra flood protection dykes and by the necessity to minimize the land required within the industrial zone. The resulting permanent land required, at the given locality is "only" the outline of the piers.

The substructure of the bridge consists of abutments N° 1 and 26, and 24 piers. In axis 21 a common pier with expansion joint is situated in order to enable movement of the two different superstructures *(Fig. 5)*. Design of the cross sections

Fig. 6: Constructing deck of DC1 with the launching nose ready for starting

Fig. 7: Pushing of the DC1 bridge section above Nové Zamky road

Fig. 8: Completed starting segment with anchor heads of transversal prestressing supported by stabilizing concrete walls

Fig. 9: Formwork wagon of the DC2 bridge section

of the aforementioned structures was influenced by aesthetic reasons, so their junction did not form a "disturbing feature".

The superstructures rest on spherical bearings.

5. THE CONSTRUCTION TECHNOLOGIES

The 806 m long DC1 section *(Fig. 1)* was constructed by incremental launching. When deciding about the construction method the time factor was crucial. By using this technology the superstructure of DC1 was completed in 320 days.

The 40 segments with varying length from 12.0 – 22.5 m were manufactured in the constructing deck behind abutment No. 1. The segments were concreted in two phases: first the bottom slab with the webs, later the top slab. After the hardening of the concrete they were fastened to the previously completed bridge section by prestressing and then pushed from the constructing deck onto the earlier completed piers, by means of hydraulic jacks. Important part of this construction technology is the steel launching nose, which serves for reducing the exceptional big stresses in the concrete cantilever structure which arise when the structure has left the previous pier, but still has not reached the next one. As a result, the spans with maximum 45 m length could be bridged.

The superstructure of the second, 360 m long dilatation unit DC2 *(Fig. 2)* was built by on-site concreting balanced cantilever method, starting from the middle piers. The site conditions, the need to minimize the land required during construction works and the crossing of the river Nitra were the main reasons for using this technology. The total time for the superstructure completion was 250 days.

The 4.75 and 5 m long segments of the four balanced cantilevers were manufactured in one phase in form travellers

Fig. 10: Construction of the last segment of DC2 at abutment 26 in hanged formwork

Fig. 11: Connection of the two arms of the DC2 bridge section above the river

Fig. 12: Construction of the last segment of DC 2 at common pier 21, on heavy scaffolding

Fig. 13: Disassembly in pieces of the launching nose of DC1 just before reaching the common pier

supported on the ready superstructure. For the sake of the balance of the scales, the cantilever fabrication was going on simultaneously at both sides of the piers. The prestressing of the bridge took place in two phases. Immediately after the cantilever segments had been manufactured, they were stressed together using internal prestressing cables, and after the 3 m long closing segments connecting the arms of the bridge had been concreted and had hardened, the entire bridge was prestressed by external cables which were placed inside the box.

6. CONSTRUCTING PROCESS

The worksite was handed over for starting construction works finally on 26th December 2009 instead of the originally planned September, later October, due to a protracted relocation of a 110 kV electric aerial cable at the location of the bridge. Work could be started with the construction of the bored piles of the deep foundation in January 2010. In the subsoil, the quality of which varied but was nevertheless mostly inadequate for the purposes of foundation works, more boring methods were needed to be used (Soil-Mec, CFA). Construction of the waterside supports necessitated making an artificial peninsula, supported by Larssen sheet piles.

At DC1 work started with the constructing decks. The core of the two structures independent of each other was the movable RC grid and the droppable bottom, loaded on the R/C grating supported by piles and connected with the pile cap

Fig. 14: Final works on DC1 bridge part: making of the curbs

of abutment No. 1. The formwork of the constructing deck, supplied by PERI, was built on the concrete grid (Fig. 6). The launching equipment (i.e. a classic lifting-pushing jack), that had to move the bridge structure into its final place, was located on the abutment. Due to the substantial length of the bridge, two of these equipments were needed to be used "from half way on", so a temporary concrete structure, a so called launching-support, capable of taking up large horizontal loads, was needed also, to be built near pier 11. The moving superstructure only passed over the other substructures. To facilitate this, temporary launching structures – slides and lateral guides – were placed on the top of the pier heads.

One of the great advantages of this technology was seen when the superstructure passed above the crossed railway line and main road: the construction works of the bridge structure caused no disturbance at all, in the traffic beneath *(Fig. 7)*.

In line with the superstructure, the substructures of DC2 are squatter, and form a single structure, but their design is in harmony with the slimmer piers of DC1. The first element of the superstructure, i.e. the 12.5 m long starting segment, was built upon a steel platform, mounted on two "blade" walls and on the pier body itself. From this point on, these blade walls were serving also to stabilize the bridge arms being built *(Fig. 8)*.

Construction of the superstructure was started above the two waterside piers at the same time. The formwork wagons were supplied by DOKA, compiled mostly from module units. Their structure consisted of four diamond-shaped deck units (one above each web of the superstructure), of which the two outer ones travelled on longitudinal rails *(Fig. 9)*.

Using these wagons, each 5 m long segment pair (7 pairs per arm) was constructed during 8 or 9 days (the segments were concreted in one phase). The last segment at the end of the bridge was constructed on scaffolds suspended at the free end of the bridge arm and on the abutment 26 *(Fig. 10)*. The middle closing segments were built by the formwork wagon *(Fig. 11)* and the segment at the other end of DC2 was constructed on a classic heavy scaffold, neighbouring the common pier 21 *(Fig. 12)*. As this part of the structure was completed earlier than the superstructure of DC1, the launching nose of DC1 had to be disassembled in pieces before it reached the common pier 21 *(Fig. 13)*.

After the RC structures had been completed, the works were finished by the insulation of the deck with 2 layers of bituminous sheet covered by a 9 cm thick asphalt layer. The final form of the bridge was given by two inspection sidewalks at the outer sides of the bridges on monolithic RC curbs *(Fig. 14)*, provided with noise protection walls, and H3

Fig. 15: *The completed bridge*

safety protecting railing, while in the middle H2 rails were installed on the curbs. After the load tests had been carried out to demonstrate the behaviour of the bridge structures being in compliance with the statical calculations, the technical hand-over process was closed and the bridge was opened for traffic. The project was finally completed *(Fig. 15).*

7. CONCLUSIONS

As usually, Hídépítő ZRt (A-HÍD Építő ZRt) attempted a difficult task again, constructing a more than 1 km long motorway bridge using two completely different technologies, moreover, outside the borders of Hungary. Due to the above fact, it had to tackle not only the usual (engineering, technological) difficulties but also other difficulties arising from different labour, official and other conditions. Despite these difficulties, our attempts resulted in full success and we were able to hand over the bridge to our Client in due time and in excellent quality.

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János BARTA (1968), MSc. Civil Eng. is the head of Technical Department of Hídépítő (M-Híd) Co. Graduated at the Civil Engineering Faculty of the Budapest University of Technology. After working for five years at a statical design company (designing mainly building structures) in 1997 he moved to Hídépítő. There he took part in the design works of the sub- and superstructures of several bridges and a pier structure of the Port of Ploče, Croatia. Some of the designed bridges were constructed by incremental launching such as the viaducts on the Hungarian-Slovenian railway line, the Homokkert overpass in Debrecen and two viaducts on the M7 highway. He was the statical designer of Hungary's first extradosed bridge structure at Letenye. He took part in the design process of Central-Europe's largest RC bridge structure, the Köröshegy viaduct. Since 2008 he has been leading the department. He is member of the Hungarian Group of ***fib.***

FRAME BRIDGES ON V-SHAPED SUPPORTS


Antónia Királyföldi - Gábor Pál

In this paper, we present the bridges built in Hungary on V-shaped supports from the World War II until today. The frame structures of this type of bridges have structural and economic advantages compared to conventional structures. The unique partly pre-cast partly monolithic structures built in the last years have combined the benefits of the two construction methods. The article explains the design aspects and the construction solutions of the bridges.

Keywords: bridge, design, frame bridges on V-shaped supports

1. INTRODUCTION

During the years after the World War II the intensity and volume of bridge building was so great, that several new types of structures appeared and were applied. The reason was mainly the reconstruction of the demolished bridges, but building new ones were necessary, too, for the modern highways, railway lines and channels.

One of the new structure types was the frame bridge on V-shaped substructure. It proved to be convenient to bridge over railway lines for sake of the bigger and safer highway traffic. Hungary was a pioneer in construction of such bridges. Of course, it is possible, that in other countries - mainly among mountainous ones - similar structures were applied formerly (Arup, 1964).

The advantages of a frame on V-shaped support are: smaller construction height, the middle span is shorter. The three spans are supported by two foundation bodies. These are not allowed to sink or slip on the ground and the connection to the pavement of the road is fix. At the beginning of its existence it was regarded as a monolithic reinforced concrete structure.

Let us mention here, that bridges with inclined legs have similar advantages (Bölcskei, 1953). Furthermore, it may be proved that at multi span structures with repeatedly changing spans a more convenient moment distribution can be reached (Tassi, Rózsa, Schlotter, 2006).

Fig. 1: The first bridge with V-shaped supports (Photo: P. Gyukics)

2. V-SHAPED SUPPORT FRAME BRIDGES IN HUNGARY

2.1 The first bridge with V-shaped supports

The first bridge on V-shaped supports was a 9.0+18.7+9.0 m span reinforced concrete slab over a railway line, near Dunaújváros, to give a two level crossing for the new highway No. 6 (*Fig. 1*) (Bölcskei, 1951, 1956).

2.2 Footbridge over M7 motorway near Velence

The building of motorway M7 started in 1961, causing new structural requirements for bridges. All the crossings of a motorway must have two levels, the width of a motorway is big (28.0 or 35.5 m), the crossings are mainly skew, there are high embankments and deep cuttings, the scaffolding for monolithic bridges is costly and hinder for road-building. That time the existing prefabricated, prestressed concrete girders had 3.0-12.0 m length.

In 1965 the motorway building reached the northern shore of Velence lake, by a deep cutting of a hill, previously wine yard. The claim, that in this beautiful surrounding an elegant rest-place should be built, which should be reached from the left and the right traffic lanes alike, to be realized by a foot bridge, which follows the 8% slope of the hillside, and crosses the 2×3 traffic lanes of M7 with one span (Királyföldi, 2000).

The soil of the hill is a sandy silt with low load-bearing capacity, not more than 100 kN/m^2.

The possible structures were a beam, an arch or a frame

Fig. 2: Footbridge over M7 motorway near Velence

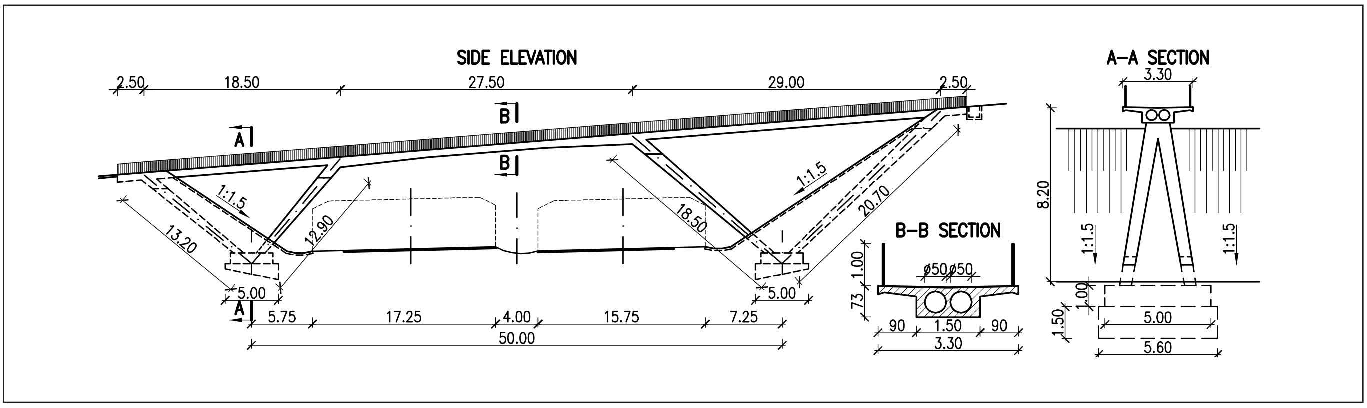


Fig. 3: Drawing of the footbridge over M7 motorway near Velence

bridge, the material had to be monolithic reinforced concrete. The beam and the asymmetric arch proved to be strong and heavy structures with complicated deep foundations, showing to the traffic of M7 an unbecoming picture. So remained the frame, slender, asymmetric and easy standing on its high V-shaped supports (*Fig. 2*).

The superstructure has a beam**,** with 18.6+27.5+29.0 m span, the inner piers are 12.9/18.5 m high, the outer ones 13.2/20.7 m high, the piers are double to give stability. The distance of hinges is 50.0 m. The foundation bodies are 5.0×6.2 m^2, with 75 kN/m^2 compression stress under them (*Fig. 3*). The outer piers are under the slope of the cutting with rubble-stone covering. The left side gully gets the rainwater from the bridge, the right-side one from the hillside.

The plans and the calculations of the foot-bridge were made following the requirements of the Hungarian Standard of Highway Bridges 1956 (live load 4 kN/m^2). The calculations were made by use of Cross-method, the independent control by force method.

Up to now only the left side of elegant rest-place was built: a coffee bar, the right side restaurant and look-out tower is only a plan.

2.3 Bridge over the road to railway station in Balatonvilágos

The M7 motorway crosses in its 93 km point the road in 70° skew direction. The road is running in 7% slope, in deep cutting (*Fig. 4*). The soil is dry, fat clay under a thin humus cover. The bridge of the M7 is a reinforced concrete slab over V-shaped supports, designed and built in 1968; designed by the rules of Standard 1956, calculated by use of deformation method. The slab is 70 cm thick with 11.0+14.0+11.0 m spans. The total width is 28.0 m but built in two phases: I. 15.5 m, II. 12.5 m. The foundation bodies are 4.0 m wide (*Fig. 5*). The plans of the V-supported bridges of M7 were designed by the Bridge-1 Department of the Road and Railway Designing Bureau (UVATERV Co.), built the bridge construction company (HÍDÉPÍTŐ Co.).

2.4 Bridge over M0 motorway under earth-road and footpath

The bridge stands in the rest-place of Anna-mountain (Fig. 6). It has a 7.0 m wide carriageway between two footpaths, each 2.1 m wide. The superstructure is a 5.7 m wide, 1.9 m deep box girder with 2×2.25 m cantilevers, the spans are 21.5+35.6+12.3 m. The asymmetry comes from the 6% slope of the road. The V-shaped supports have 5.3×0.7 m^2 slabs for the inner, and two-two 1.0×1.0 m^2 columns for the outer legs (*Fig. 7*). The distance of the hinges is 48 m, they stand on simple foundation. The monolithic structure was built 6.7 cm higher, to compensate the calculated deformation of the box girder. The detail designs and the calculation were made by the Bridge Department of FŐMTERV in the years 1991-1992, following the prescriptions

Fig. 4: Bridge over the road to railway station in Balatonvilágos

Fig. 5: Drawing of the bridge over the road to railway station in Balatonvilágos

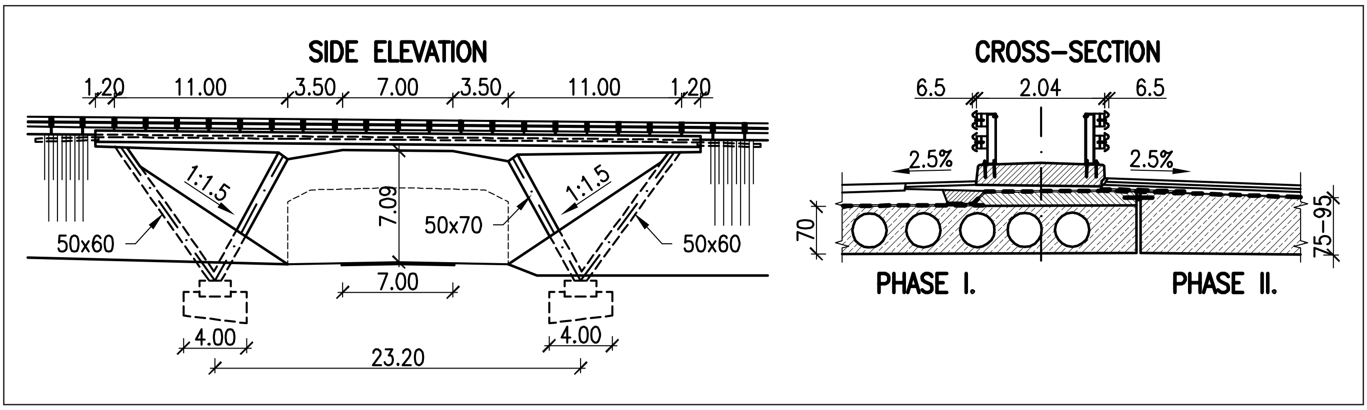


Fig. 6: Bridge over M0 motorway under earth-road and footpath

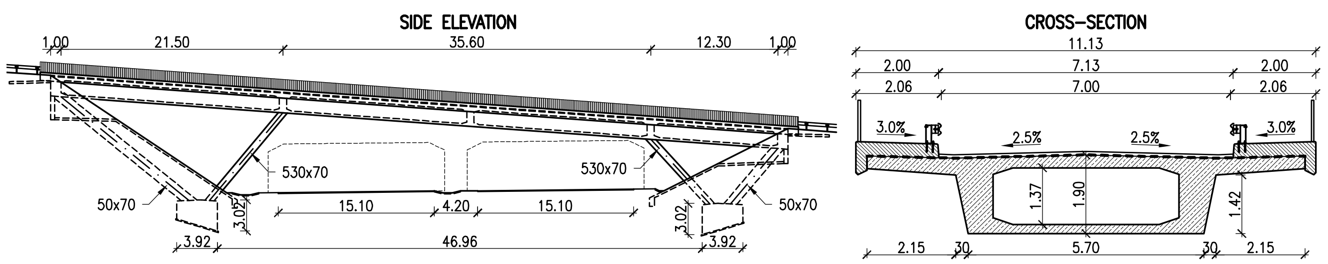


Fig. 7: Design of bridge over M0 motorway under earth-road and footpath

of MSZ 07-3701-86 Hungarian Standard. The bridge was built in 1992-1994 by Asphalt-Road Building Company.

2.5 Bridge over the junction of highways 8 and 72

Till the year 2008 the connection of highway 72 to highway 8 was a complicated one level junction. For the safety of traffic it had to be reconstructed to two levels. The building of the necessary bridge was not permitted to hinder the traffic, so it cannot be a monolithic structure (Királyföldi, Tassi, 2009). The crossing is skew, α=75° (*Fig. 8*).

The junction is on the eastern slopes of Bakony mountain. The southern-side of the highway 8 is showing a cutting slope of big dolomite-slabs. So the intention of the bridge designer was obvious that this is the convenient place for a frame bridge supported by V-shaped legs. The monolithic supports can be built out of the clearance of highway 8, and prefabricated, prestressed concrete girders can be put over them in a few hours. The traffic can bear such a short hindrance. It means a new situation, that the connection between the monolithic support and the prefabricated superstructure must be perfect. The idea and the design for permission were made by Speciálterv Ltd.

Fig. 8: Bridge over the junction of highways 8 and 72

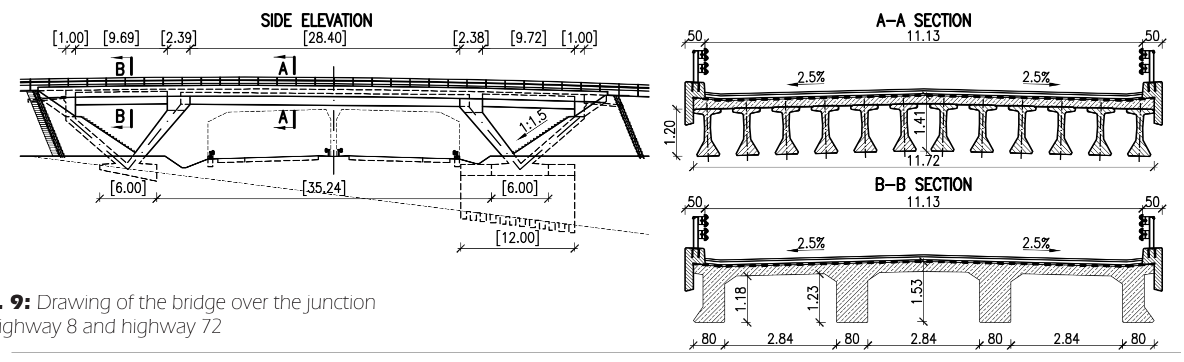


Fig. 9: Drawing of the bridge over the junction of highway 8 and highway 72

The spans of superstructure are 11.8+30.0+11.8 m, the distance of hinges is 41.2 m, the foundation bodies are 6.0×12.0×1.2 m³ blocks, the inner and outer columns are pegged down.

The detail design was won by Kánya Ltd, and the first step was a survey on the spot. The left side of highway 8 was beautiful with the big slabs of dolomite, but on right side there was a line of 15-20 m high pine trees, and behind them a strong slope downwards. There was the question: is the level of rock horizontal or does it lay in a slope. It proved that the level of dolomite rock lies in a 12% slope under the pavement of highway 8, so under the foundation level of right side V-support is a 4.0 m thick cover of wet soil, which has a slight load bearing capacity (*Fig. 9*).

This problem could cause a change in the symmetric structure of the design for permission: a left support 5.5 m high, 30.0 m span bridged with prefabricated girders and a right support with 9.5 m height causing a big asymmetry - when the foundation would be over a rock. To dig out such a deep pit beside the right traffic-lane of highway 8 was a bit dangerous work. So there remained a deep foundation and symmetry was preserved.

The deep foundation could be a double pile-wall or slurry-wall. The soil mechanics specialist's proposal was the pile-wall, with Ø60 cm bored piles, bored 80 cm deep into the rock. So the monolithic V-supports were built, the piles were bored from the level of the carriageway, without any accident, their longitudinal bars are bound into the foundation block.

Well known task was to lifting and placing the 12 prefabricated girders - each has a 23.1 t mass - over the cross-beam of the inner skew columns, not causing the calculated 15 cm deformation. It was not a big problem that the scaffolding of the monolithic V-support was built strong enough to bear the weight of the girders and the slab over them with only 1-2 mm deformation. The traffic was hindered for 3.5 hours.

The structural cap-beam over the inner columns has a great importance. Even without the weight of the prefabricated girders and the monolithic slab it has to resist vertical and horizontal moments and shear forces. The reinforcement of the end cross girder and the longitudinal bars of the slab have to produce the convenient connection between the supporting elements and the superstructure. This construction of bars, then the concreting process had to be made over the traffic.

When the concrete of the slab became hardened the scaffolding was demolished, no deformation could be measured. The results of the load test verified the terms of the design. The calculations were made by use of UT 2-3.401-414 Standard by use of AXIS VM9 program, the independent control calculation was made by the Department of Structural Engineering of Budapest University of Technology and Economics. The bridge was built by Colas Hungary Company.

2.6 Frame bridge over the M31 motorway

The three-span motorway-overbridge spanning over the M31 at the cross section 7+073.58 km (*Figs. 10-11*) was designed by the Speciálterv Ltd. in 2008.

The road going through the bridge has a regulation width of 8.50 m, and a straight horizontal alignment with a right angle to

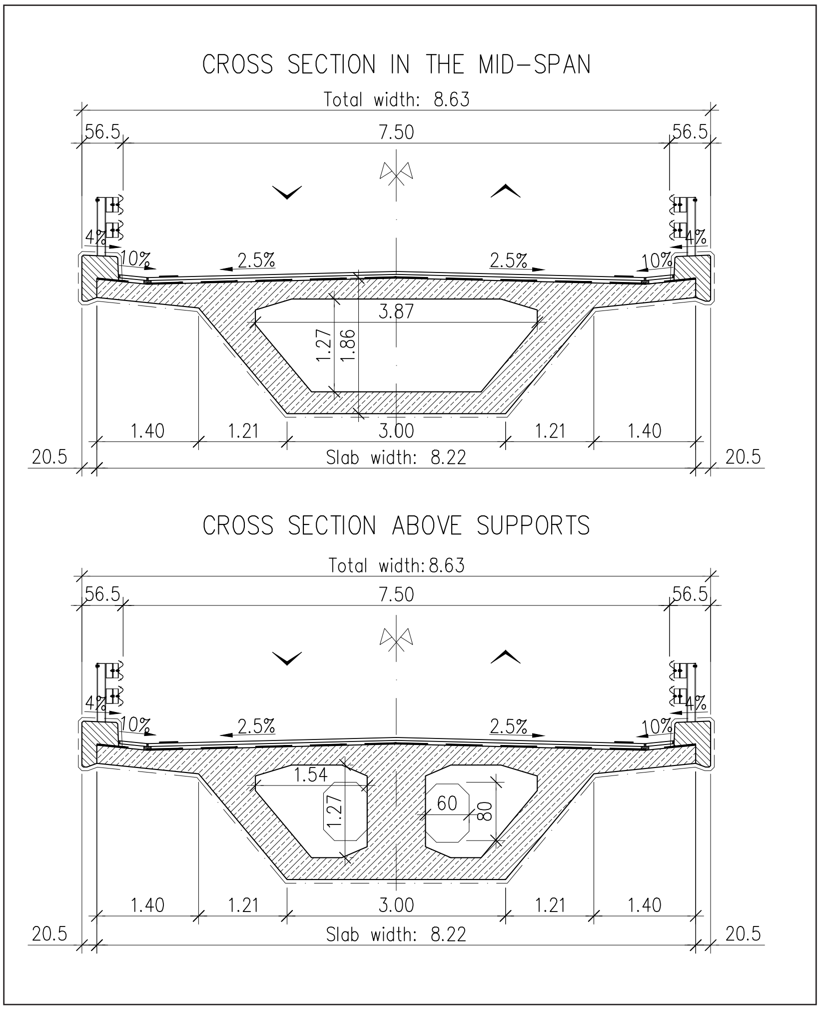
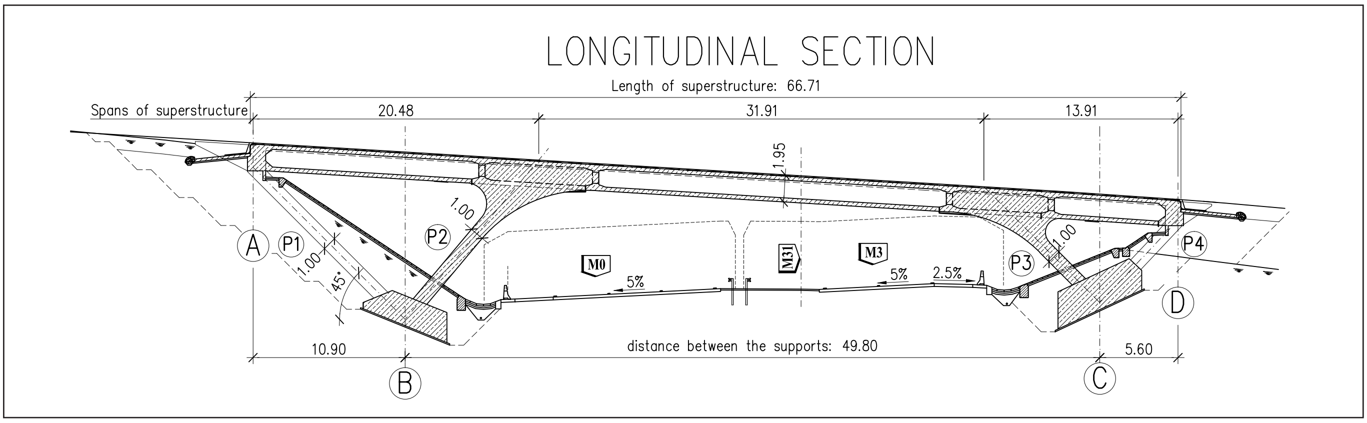


Fig. 11: *Cross-section of the M31 bridge*

Fig. 10: *Longitudinal section of the M31 bridge*



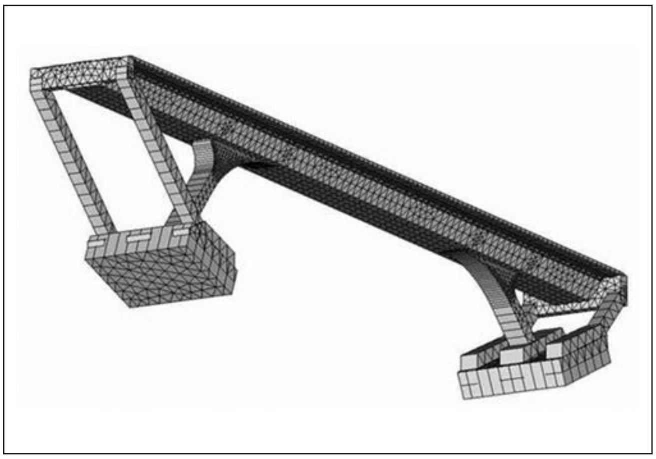


Fig. 12: The model of the M31 bridge in 3D by the Finite element software

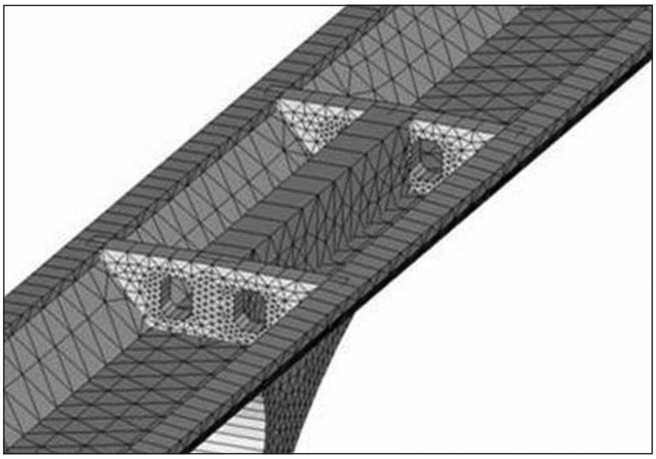


Fig. 13: The part of the M31 bridge deck underneath, above the internal leg. (from the finite element software)

the obstacle. The motorway under the bridge has a regulation width of 34.82 m and has the longitudinal alignment at cut. Due to the 6% gradient of the longitudinal axis of the bridge deck, the arrangement of the supports are asymmetrical 10.90 + 49.80 + 5.60 m. The total length of the bridge is 68.70 m.

The inclined pillars (legs), the bridge deck and the supports joins in rigid connections. The end points A and D are supported by angled columns. The end structural beams are joint with the wing walls as perpendicular cantilevers. These beams have a width of 130 cm.

From the end points of the superstructure to the abutments the load is transferred by two pairs of legs with cross section of 1.00 m × 1.00 m in each case. The obliquity of the legs is 45° at the end structural beam „A" and 46° at the „D". The superstructure is also supported by two inner legs, each with a cross section of 1.20 m × 1.00 m. The two inner legs have angles of 45° to the horizontal line (*Figs. 10-11*).

The 1.86 m deep box girder has observing room capable for crawling on its whole length and reduced cross sections are above the supports.

In order to achieve a uniformly distributed load on the ground with uniform stress the flat foundation was placed obliquely with a slope of 22° and 25° according to the directions of the reaction forces from the legs of the frame. The distance between the axes of the abutments is 49.80 m and the sizes of the abutments are 6.00 m × 8.62 m.

The load bearing capacity of the structure was designed for the load of „A" type according to the Hungarian regulations (ÚT 2-3.401:2004). It was an important design aspect to take care about the stiffness of the superstructure due to the large span made of cast-in-situ concrete without post-tensioning. The longitudinal axis of the deck has a constant grade with a straight line so the aesthetic aspect for this case did not allow us any visible deflection along the span. Calculation of the structure's expected shape was based on the design assumptions set up by considering the cracked section in the modelling of the structure in two different finite elements software – AxisVM (*Figs. 12-13*) and Sofistik.

The bridge is made of cast-in-situ concrete. The concrete used for the superstructure and the V-shaped legs have the class of C40/50 and C20/25 for the supports (according to the Hungarian standards ÚT 2-3.414:2004 and ÚT 2-3.413:2005). The bridge was opened for the public in 2010 (*Fig. 14-15*).

2.7 Bridge over the Perkáta creek, on the bypass of the main road No. 62

The three-span V-leg frame concrete bridge, located at the cross-section of 15+520.70 km on the bypass of the main road No. 62 was also designed by Speciálterv in 2008. The bridge currently is in process of construction and it is to span over the Perkáta creek and an earth road (*Figs. 16-17*).

The regulation width of the bridge deck is 12.00m, and the horizontal alignment is a straight line with a skew of 70° to the axis of the obstacle. The vertical alignment is a straight line with a slope of 1.37%. The total length of the deck is 51.36 m. The position of the upper supports of the frame's legs underneath the bridge deck are at 11.50(10.81) + 25.40(23.87) + 13.50(12.69) m, and the distance between the abutments on the ground level is 37.43(35.17) m (The values in parenthesis are perpendicular measurements.).

Fig. 14-15: The M31 bridge during the construction works (left) and before opening (right)

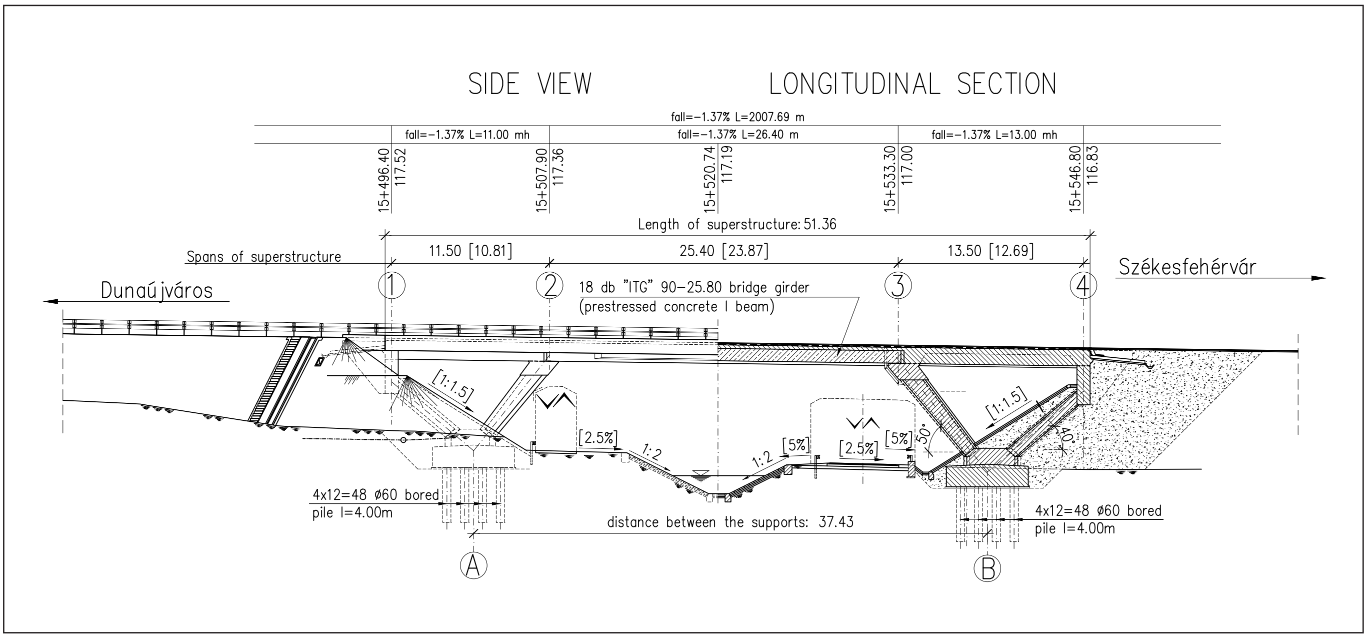


Fig. 16: *Side view and longitudinal section of the Perkáta bridge*

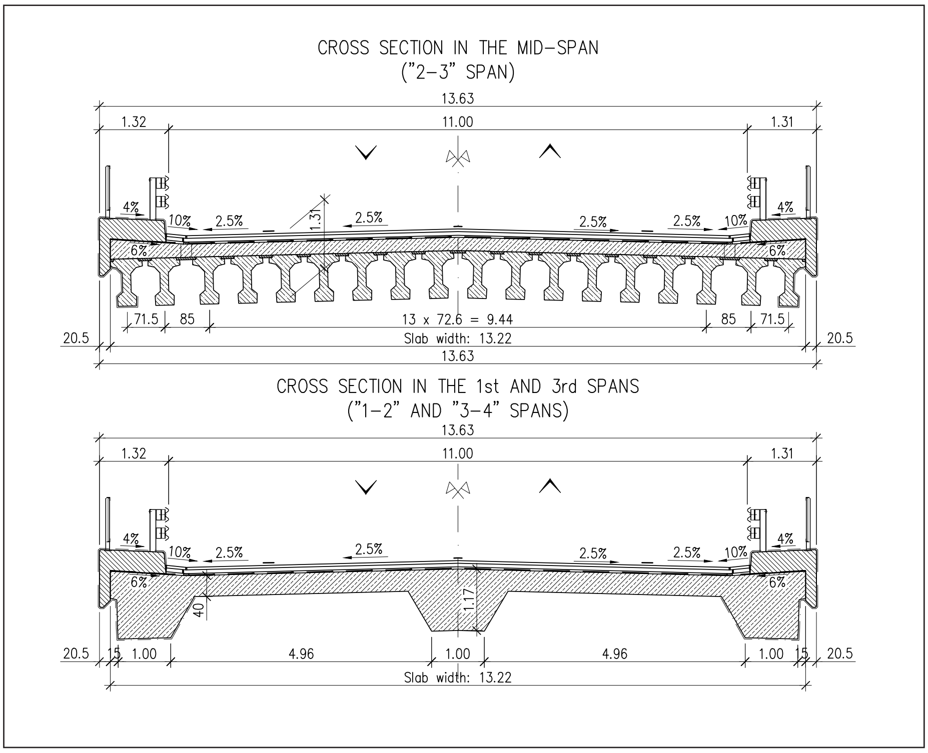


Fig. 17: *Cross-section of the Perkáta bridge*

The load bearing capacity of the structure was designed for the load type of „A" (according the Code ÚT 2-3.401:2004).

The bridge is similarly arranged to the bridge at the 8-72 junction, so at the end spans the structure is a monolithic frame and at the internal span is a composite structure with prestressed prefabricated "I" girders and monolithic slab. The monolithic legs have V-shaped arrangements. The prefabricated prestressed concrete beam marked by ITG is 25.80 m long and 90 cm deep. The beam seats are everywhere at least 40 cm. The prefabricated girders are set on technical plastic bearing plates with the sizes of 20×35×2 cm. The reinforced concrete joist head of the ITG beam is very suitable for the complex loads above the inclined frame legs.

The hidden abutments give the foundations for the pillars and columns of the frame. In this case the ascending columns are the inclined frame legs in the bank.

The bridge has a deep foundation made of CFA bored piles with a diameter of 80 cm. There are 2×5 piles at each support with a length of 4.00 m bored in a 5.00 m thick grey silty sand ground layer. The large pile-closing beams have a cross-section size of 560×150 cm. On each of these beams are standing three inclined columns to each direction, one pointing to the internal, the other to the external main girders. The No. 2 internal legs have 85×100 cm size at the bottom and 125×100 cm at the top, whilst the column No. 3 is also 85×100 cm at the bottom but 133×100 cm at the top. Both of them have a bottom surface with an angle of 50° to the horizontal axis. The sides of the abutments are parallel, the size of the outer columns (at the side of the back filling) cross-section is 90×100 cm, and the longitudinal axis has an angle of 40° to the horizontal.

The bridge is made of a combination of cast-in-situ concrete and precast concrete elements. The cast-in-situ concrete used for the bridge deck and the V-shaped legs have the class of

Fig. 18: *Perkáta bridge under construction*

Fig. 19: Perkáta bridge under construction

C35/45 (according ÚT 2-3.414:2004 and ÚT 2-3.413:2005), whilst the precast concrete structure members have the class of C40/50.The class of the concrete used for the pile-closing beams is C25/30.

The bridge is in process of construction (*Figs. 18-19*). The bridge structure has been finished and the road expectedly at the end of 2012 will be ready for opening to the public.

3. CONCLUSIONS

Our renowned ancestor was seriously interested in the difference which can be reached by creating spatial structures instead of plane ones (Bölcskei, 1951, 1953, 1956). So came the V-support in bridge building and branchy columns for halls (Bölcskei 1963).

During the decades bridges over V-supports were built, at the beginning only with monolithic reinforced concrete slabs, beams, box girders; now, because it is a claim, combinations of monolithic V-support and prefabricated girders. The results are giving hope of further possibilities to develop the V-shaped supports, having good conditions in calculation and in technology.

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Antónia Királyföldi (1932) MSc, civil engineer (1961), engineer specialist in reinforced concrete construction (1974). She worked 1950-1991 in UVATERV design bureau, her main task was designing bridges for M7, M1 and M0 motorways. She was designer of more than 100 different bridges. From 1972 she had been teaching as assistant at the ancestor of the today Department of Structural Engineering of Budapest University of Technology and Economics; from 1990 to 2005 she took part in the engineering education in foreign languages. She was given the Honorary Associate Professor title in 2003. She is active at Civilplan Ltd. Member of Hungarian Group of ***fib***.

Gábor Pál (1970) MSC civil engineer (Budapest Technical University, 1994), 1994-1999 designer (FŐMTERV Co.), 1999 onwards executive of SpeciálTerv Ltd. His scope is to manage the 30 plus organisation of designers, and to control and expertly lead the civil engineering design work of the company. Main designer of the Metro station of Kelenföld railway station and more than 100 different bridges. Member of Hungarian Group of ***fib***.

PREFABRICATED BRIDGE GIRDERS – FROM DESIGN TO IMPLEMENTATION

Kálmán Koris – István Bódi – György Dévényi

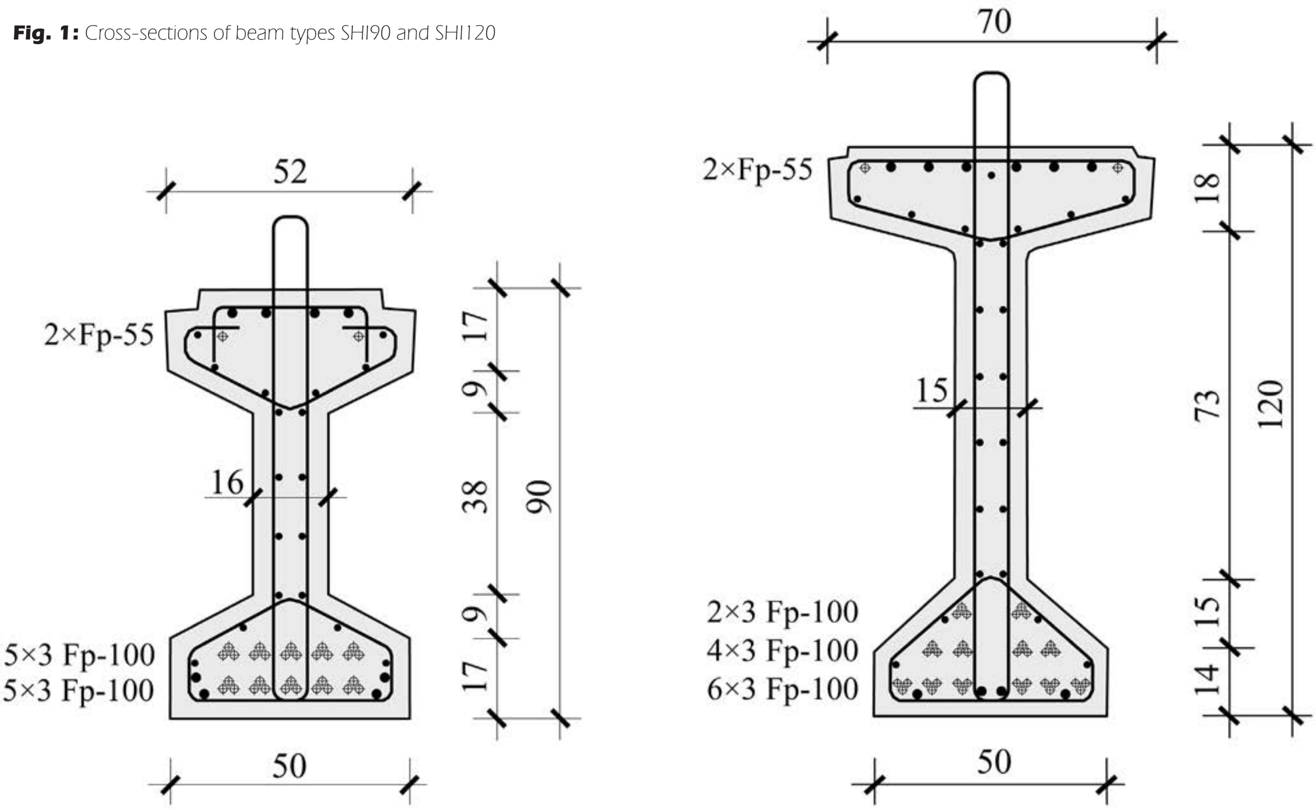
To facilitate the practical application of European Standards (EN) in Hungary the Department of Structural Engineering at the Budapest University of Technology and Economics was collaborating with the industrial partner SW Umwelttechnik Ltd. In frames of this collaboration prefabricated, prestressed concrete bridge girders were also developed among other types of prefabricated structural members. Beside the detailed numerical analysis of the girders according to EN, their durability design was also performed using a probabilistic approach. Within the scope of the durability design, the probability of failure of the prefabricated girders was analysed as a function of time, considering the effect of creep, shrinkage, relaxation, carbonation induced corrosion and the deterioration of cross-sectional sizes due to environmental effects. Beside the design of standalone girders, complete bridge superstructures consisting of the developed beams were also calculated to prove their applicability according to EN. This article also provides a brief insight into the manufacturing process of the bridge girders, and some examples to their application for highway construction projects are presented, too.

Keywords: prefabrication, prestressing, bridge girders, design, manufacturing, durability, Eurocode

1. INTRODUCTION

After the finalisation of European Standards (EN) in 2006 they were used parallel to the National Standard (MSZ) in Hungary. The deadline for the usability of Hungarian National Standard expired on 31st March 2010 and after a nine-month grace period only the European Standard can be used for the design of engineering structures. In the period before the introduction of EN – prior to the current economic recession – SW Umwelttechnik Ltd. was working on the development of a new

Fig. 1: Cross-sections of beam types SHI90 and SHI120



product line, targeting the areas of bridge construction. In order to prepare for the practical application of EN, a cooperation was established between the Department of Structural Engineering (Budapest University of Technology and Economics) and SW-Umwelttechnik Ltd. to control existing prefabricated concrete members for the purposes of building construction, as well as to develop a range of new prefabricated, prestressed concrete bridge girders by the exclusive use of EN (Bódi, Koris, 2006). The developed SHI product line consists of 90 cm and 120 cm deep bridge girders *(Fig. 1)*. Considering 40-40 cm bearing lengths at the supports, the SHI beams can be used for bridges with 10-32 m horizontal clearances. The prefabricated girders are cooperating with a cast-in-situ reinforced concrete slab, allowing the construction of simply supported or continuous highway bridge structures.

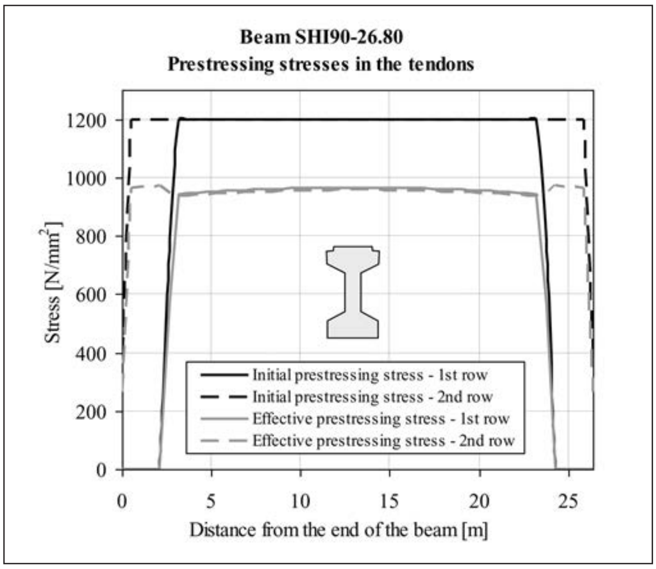


Fig. 2: Initial and effective prestressing stresses

2. DESIGN PROCESS

2.1 Design of the prefabricated girders according to EN

To be able to efficiently cover the typical span range for highway bridges in Hungary, two different cross-sections were designed *(Fig. 1)*. Beam type SHI90 is 90 cm deep and its length changes between 10.80–26.80 m, while the beam SHI120 is 120 cm deep with lengths between 16.80–32.80 m. Exact shapes of the cross-sections were determined considering the experiences of previous Hungarian road bridge girder types (Balázs, 1995; Koris, Salamak, 2007), as well as the aspects of construction, transportation, erection, economy and durability.

The analysis of the individual beams started with the detailed calculation of the effective prestressing force, considering the effect of creep, shrinkage and relaxation according to EN (Koris, 1998; Bódi, Koris, 2006). The distribution of effective prestressing force along the length of the beam was determined for each row of the cables *(Fig. 2)*. Concrete stresses in the top and bottom flange were also controlled after the release of the tendons *(Fig. 3)*. The next step was ultimate limit states (ULS) including the bending and shear resistances of the girders (Koris, Bódi, Erdődi, 2005). These types of beams are typically used for slab and beam bridges where the prefabricated girder is cooperating with a monolithic reinforced concrete slab. Therefore, the ultimate bending moment was also determined and controlled for the composite cross-section formed by the prefabricated beam and the on-site concrete slab (a 200 mm thick reinforced concrete slab was considered in case of SHI90 beams, and a 250 mm thick one in case of SHI120 beams).

Serviceability limit states (SLS) were also analysed,

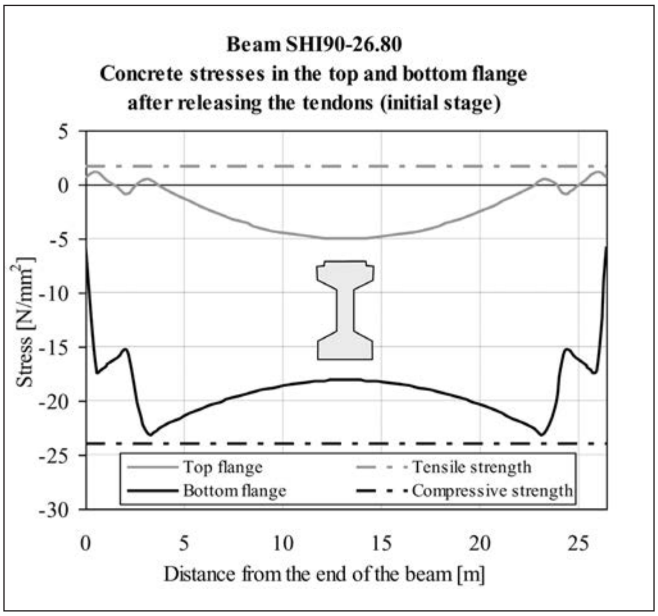
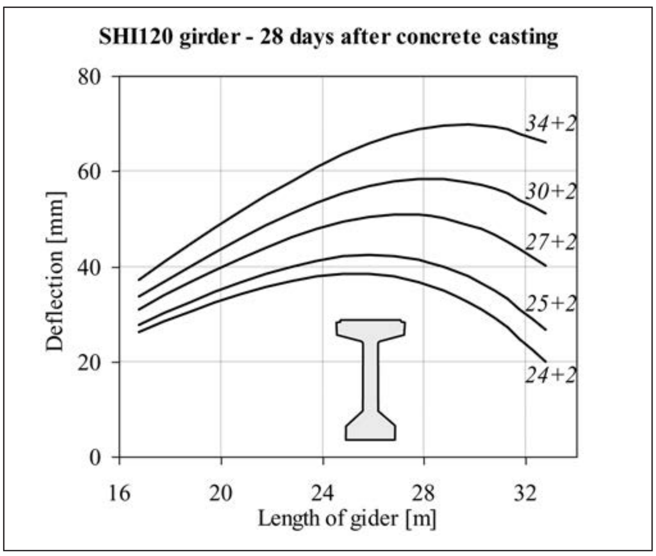


Fig. 3: Concrete stresses in initial stage

Fig. 4: Deflections of different SHI120 beams 28 days after concrete casting and in final stage



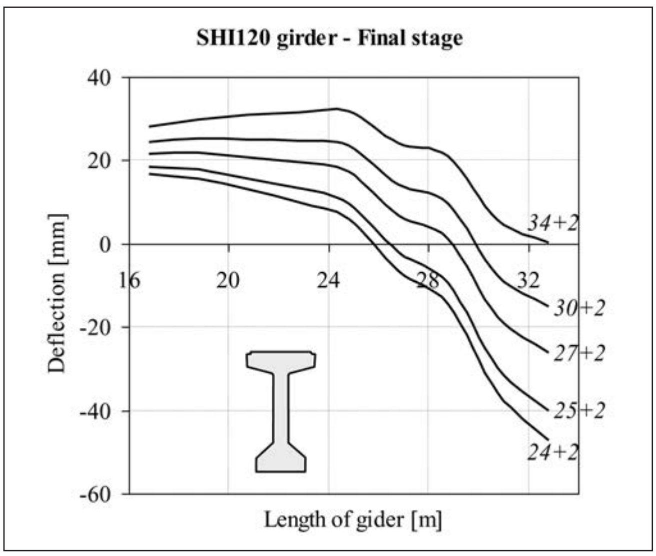


Table 1: Summary of data on the SHI90 beam type

Type of beam	Length	Area of cross-section	Concrete volume	Total weight	Prestressing tendons						Steel bars	
					Fp100			Fp55			B60.50	
		[m^2]	[m^3]	[tons]	[pcs]	[kg/pcs]	[kg/m^3]	[pcs]	[kg/pcs]	[kg/m^3]	[kg/pcs]	[kg/m^3]
SHI90-26.80	26.8	0.288	7.71	19.28	30	678.2	88.0	2	24.9	3.2	1545.8	200.5
SHI90-24.80	24.8	0.288	7.13	17.84	30	631.1	88.5	2	23.2	3.2	1443.3	202.3
SHI90-22.80	22.8	0.288	6.56	16.40	24	467.2	71.2	2	21.4	3.3	1343.4	204.8
SHI90-20.80	20.8	0.288	5.98	14.96	24	429.6	71.8	2	19.7	3.3	1240.6	207.3
SHI90-18.80	18.8	0.288	5.41	13.52	21	342.9	63.4	2	18.0	3.3	1140.7	210.9
SHI90-16.80	16.8	0.288	4.83	12.08	21	309.9	64.1	2	16.2	3.4	1038.2	214.8
SHI90-14.80	14.8	0.288	4.26	10.64	19	250.6	58.9	2	14.5	3.4	932.5	219.0
SHI90-12.80	12.8	0.288	3.68	9.21	19	220.7	59.9	2	12.8	3.5	855.3	232.3
SHI90-10.80	10.8	0.288	3.11	7.77	17	170.8	55.0	2	11.1	3.6	721.5	232.2

including the stress limitation, crack control and deflection control of the girders. Expected deflection of the mid-span was calculated in different stages of construction; namely, after the removal of the formwork, 28 days after concrete casting and in final stage when the beam already supports the weight of the monolithic concrete deck, too. *Fig. 4* illustrates the maximum deflections of the SHI120 beam of different lengths 28 days after concrete casting and in final stage (the number of the applied bottom + top tendons is indicated for each case).

Besides controlling the ultimate and serviceability limit states, we also analysed the local stresses at the ends of the girders (Bódi, Koris, 2006). The exact distribution of transversal tensile stresses at the anchorage of the tendons, as well as the necessary supplementary reinforcement were determined. Other local effects, like direct shear failure of the beam's edge at the support, and the concentrated force resistance of the concrete at the bearings were also examined. Temporary stages (lifting and transportation of the prefabricated beams) were investigated by checking the maximum concrete and steel stresses, as well as the lateral buckling resistance of the girders. Finally, the amount of reinforcement needed for the proper cooperation of prefabricated beams and the monolithic concrete deck was determined.

After performing the above mentioned calculations according to EN, the detailed formwork and reinforcement plans were prepared for each beam. The exact amount of concrete and reinforcement needed for the manufacturing of different beams were determined (Table 1 summarises the data of SHI90 beams)

2.2 Analyses of bridge superstructures

The calculation of standalone bridge girders was implemented by the analysis of complete bridge superstructures consisting of the developed beams to prove their applicability according to the EN. Altogether eight different bridge superstructures covering the total range of available beam lengths and types were controlled. Possible bridge cross-section configurations consisting of SHI90 and SHI120 beams are shown in *Fig. 5*. The internal forces of the sample bridge structures were numerically determined by AxisVM 8 civil engineering finite element software, considering the uniformly distributed (UDL) and concentrated (TS) traffic loads defined for bridges by the EN.

The design values of actions (bending moments and shear forces) obtained from the finite element calculations were compared to the appropriate resistance values determined previously. According to this comparison, the examined bridge girders fully meet the requirements of EN so they can be safely applied for the construction of highway bridges. Based on the results of finite element analyses, the maximum applicable distance between the individual beams was calculated for each beam type separately. The applicable maximum of the unit gross load (excluding the self-weight of the beam) was also determined for both ultimate and serviceability limit states. The independent control calculation of the SHI bridge girders and the sample superstructures was performed by Pont-TERV Ltd. according to the Hungarian Highway Engineering Regulations (ÚT).

2.3 Durability design of the prefabricated girders

The importance of durability became an essential aspect of the design process recently (Balázs, 2008). In order to provide proper service life estimation for the developed bridge girders, the rheological changes of material properties and the decrease of structural sizes due to environmental effects must be modelled, and these changes must be considered during the determination of structural resistance. Rheological processes in reinforced and prestressed concrete are usually affected by the environmental conditions, such as average temperature, ambient humidity and presence of aggressive agents, as well as by the initial material properties of the structure. The design procedures of EN include the consideration of durability on a certain level, but we decided to perform a more detailed durability design of the SHI girders. The implemented method is based on a probabilistic approach, that is, the probability of failure of the examined beams was expressed at different ages of the structure. To describe the long-term behaviour of the structure, we considered the creep and shrinkage of concrete, relaxation and carbonation induced corrosion of steel bars and tendons, as well as the decrease of material properties and cross-sectional dimensions during the analysis.

2.3.1 Basis of the applied probabilistic approach

The probability of failure of the SHI beams was calculated as a function of time, by means of varying load carrying capacity and external loads, considering a failure due to bending

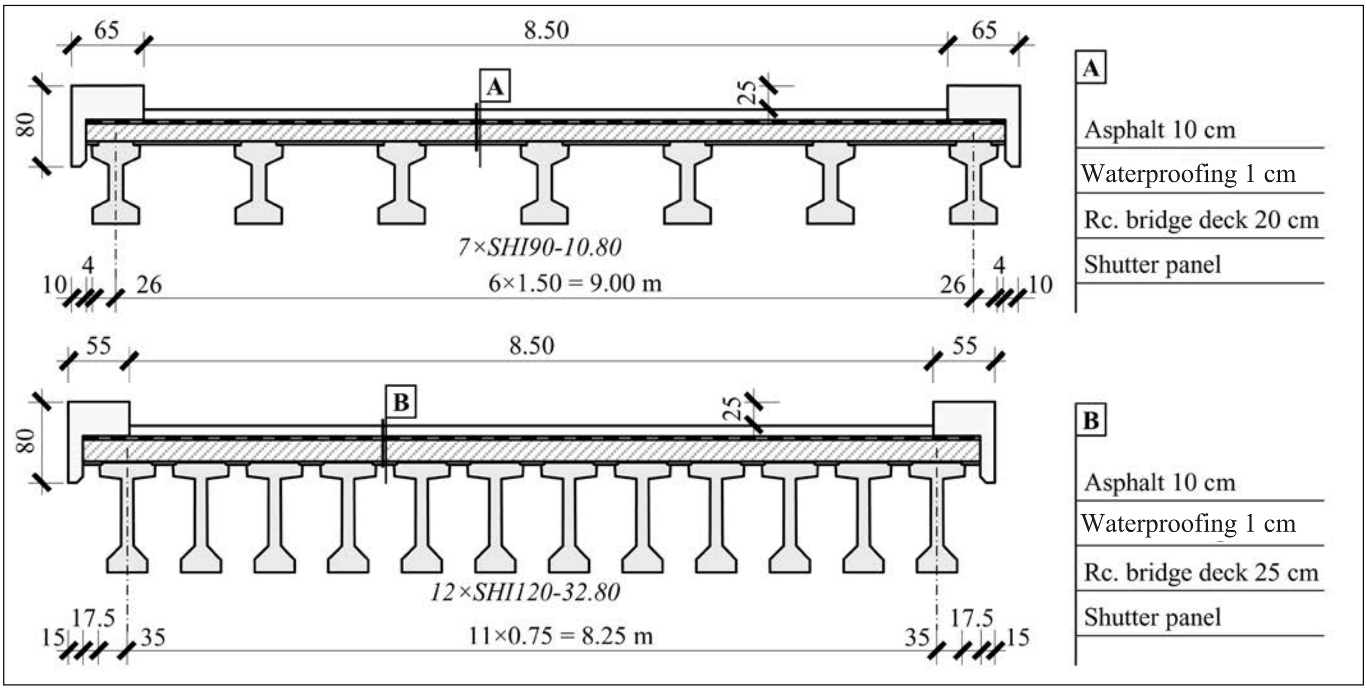


Fig. 5: Sample cross-sections of bridge superstructures consisting of SHI90 and SHI120 beams

(premature shear failure was always avoided by the application of appropriate shear reinforcement). The first two stochastic parameters (mean value, standard deviation) were used to describe the distributions of the structural resistance and the acting loads (Koris, 1996, 2007). Mean value and standard deviation of structural resistance were calculated by stochastic finite element method (Lawrence, 1989), while stochastic parameters of load effect were considered according to the referring literature (Mistéth, 2001).

Mean value of structural resistance was determined by finite element method (Bojtár, Gáspár, 2003) considering non-linear behaviour of concrete and reinforcement. Mean values of geometrical sizes and material properties were used for the analysis. The method of load increments was used and for each load increment, the chord stiffness matrix was evaluated. Failure of the structure was specified by a damage indicator, consisting of the eigenvalue of stiffness matrix in case of structural damage and the eigenvalue of current stiffness matrix at the given load level.

The scatter of structural resistance was evaluated from the scatter of the stiffness matrix ($\underline{K}$). Assuming that $\underline{K}$ is a function of an α random input variable, its variation ($\delta\underline{K}$) can be approximately expressed by the first term of its Taylor's series. This approximation is reasonable if the variation of the stiffness matrix is less than about 15%. According to the results of the calculations, this limitation does not apply to the analysed prefabricated beams. Using the above approximation, the covariance matrix of structural resistance was expressed in the following form (Eibl, Schmidt-Hurtienne, 1996; Koris, 1996, 2004):

$$\underline{\underline{C}}_{q} = \delta\underline{q}_F \cdot \delta\underline{q}_F^{T} = \underline{\underline{K}}_F^{-1} \cdot \frac{\partial \underline{\underline{K}}}{\partial \alpha} \cdot \underline{u} \cdot \delta\underline{\underline{\alpha}} \cdot \underline{\underline{C}}_{\rho} \cdot \delta\underline{\underline{\alpha}}^{\mathrm{T}} \cdot \underline{u}^{\mathrm{T}} \cdot \frac{\partial \underline{\underline{K}}^{\mathrm{T}}}{\partial \alpha} \cdot \underline{\underline{K}}_F^{-T}$$

where $\delta\underline{\alpha}$ includes the standard deviations of random input variables and $\underline{\underline{C}}_{\rho}$ is the correlation matrix. The correlation between different finite elements was described by an exponentially decaying function of the distance between the elements and the correlation length. The standard deviation of structural resistance was obtained as square root of the diagonal elements in the covariance matrix $\underline{\underline{C}}_q$. Using the above equation, the standard deviation of structural resistance was evaluated on structural level. The applied stochastic finite element formulation was described by Koris, Bódi (2009).

2.3.2 Determination of the stochastic input parameters

Initial mean values and standard deviations of material properties were determined from material tests performed during factory quality control. Concrete strength was obtained from uniaxial compression tests carried out on 150 mm cubes. Laboratory tensile tests were carried out to determine material properties of reinforcement. Modulus of elasticity, strength and ultimate strain of the steel bars and prestressing tendons were measured. The strength of concrete, steel bars and prestressing tendons are the most important material properties to affect the structural resistance in ultimate limit state. The decrease of mean values of material strengths was described by an exponentially decaying function (Koris, Bódi 2009). The loss of initial prestressing stress (σ_{p0} = 1200 N/mm^2) was determined according to Eurocode 2, considering the effect of creep, shrinkage, relaxation and the elapsed time since manufacture. The average stress loss in the tendons after 100 years was about 18% in case of the analysed beams.

Mean values and standard deviations of cross-sectional sizes (height and width of the cross-section, effective depth of steel bars and prestressing tendons) were determined from the results of measurements on manufactured SHI beams. Changes of the mean values of geometrical sizes are usually insignificant in case of concrete structures, thus, constant mean value for structural geometry was assumed during the calculation. Changes of the standard deviations of the given geometrical parameters in time were considered by the Gauss-process (Mistéth, 2001).

Decrease of steel bar and tendon diameter due to carbonation induced corrosion was also considered. The carbonation depth at a certain time after manufacture was calculated according to the *fib* bulletin 34 (2006). After the concrete cover is completely carbonated, steel bars and tendons may begin to corrode. The relation between the diameter of rusted steel bar and corrosion time under normal atmospheric conditions was considered by the following equation (Zhao, Fan, 2007):

$$\varnothing(t) = \varnothing_0 - 0.0232\int_0^t i_{corr}(t)dt$$

where t is the time of corrosion measured from the time of carbonation (t_c = the time point where the carbonation depth is equal to the concrete cover), $Ø_0$ is the diameter of steel bars before corrosion, and $i_{corr}(t)$ represents current corrosion density at time t.

2.3.3 Service life design of the SHI beams

The prefabricated SHI bridge girders were analysed by the introduced probabilistic design method. Results of the analyses are presented in case of the SHI120 beam of three different lengths (20.8 m, 26.8 m and 32.8 m). During the calculations, width and height of the cross-section, effective depth of steel bars and prestressing tendons as well as strength of concrete, steel bars and tendons were considered as random quantities. Probability of failure was calculated in different points of time. Selected times for the analysis were t=10, 25, 50, 75 and 100 years. According to our previous experiences (Koris, Bódi, 2009), the value of relative ambient humidity has a significant influence on the aging process of the structure, so we intended to use humidity values as realistic as possible for the analysis. A current place of application for the SHI beams is the main road at Csepel Island, therefore, we evaluated the humidity level at this location. The humidity values measured at a meteorological station close to the construction site were used to determine the monthly averages *(Fig. 6)*. The average relative humidity over the examined time period (4 years and 2 months) was about 70%, so this value was used for the durability design of the SHI girders. Current values of input parameters were calculated by the computer software *PFEM2008* which was developed using the *Matlab® 7* software package. The bending capacity was evaluated for each examined beam, considering the changes of input parameters due to the mentioned rheological processes (Koris, Bódi, 2009). The effect of elapsed time on the flexural behaviour of the cross-section is demonstrated by the corresponding bending moment – curvature (M–κ) diagrams in *Fig. 7*.

Fig. 6: Change of the monthly average of relative humidity over the examined time period

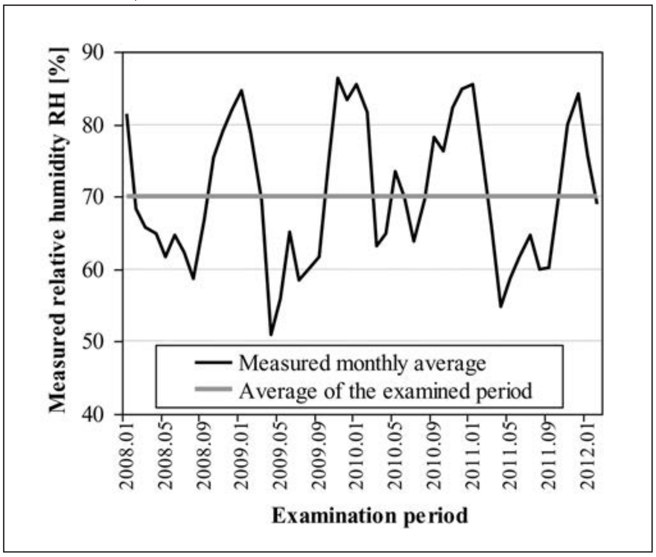
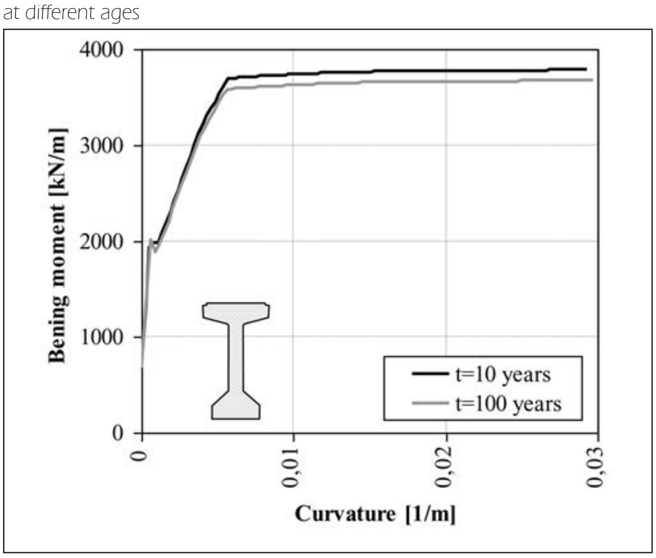


Fig. 7: Bending moment – curvature diagrams of the SHI120-32.80 beam at different ages



The load carrying capacity of the girders was expressed by the maximum applicable load belonging to the appropriate bending capacity. *Fig. 8* illustrates the change of the mean value and standard deviation of the maximum applicable load in case of different SHI120 beams. It can be stated that mean value of structural resistance slightly decreases, while its standard deviation increases over time. The mean value of maximum applicable load is of course higher in case of shorter beams; however, the relative standard deviation is larger in case of larger spans.

Using the stochastic parameters of the structural resistance and the load effect (Koris, Bódi, 2009), the probability of failure was evaluated in the selected points of time. Results of these calculations are summarised in *Fig. 9*. The different curves in the diagram refer to SHI120 girders of different length. As we have expected, the probability of failure is increasing as time is passing by. The effect of the beam length can also be observed by the comparison of the different SHI120 girders. Generally, the probability of failure is higher in case of longer beams, but over time the difference between the examined members decreases. According to EN the probability of failure cannot be larger than p=10^{-4} during the service life of the structure. The design service life of SHI girders was 100 years, and the probability of failure at this time is lower than 10^{-4} for each analysed beam (see *Fig. 9*), so the previous criterion is satisfied. The expected service life (t_s) of the different SHI beams was approximately calculated by linear extrapolation of the given curves (*Fig. 9*): $t_{s,32.80} \approx 101.5$ years, $t_{s,26.80} \approx 108.5$ years, $t_{s,20.80} \approx 117$ years.

Summarising the results of the performed durability design, the analysed SHI girders slightly outperform the requirements of the EN standard, even in case of the longest beams, so they can be safely used for the construction of highway bridge superstructures.

3. MANUFACTURING OF THE PREFABRICATED GIRDERS

The bridge girders are manufactured using the ISO 9001:2000 Quality Management System, at the Alsózsolca (near Miskolc) or Majosháza (near Budapest) sites of SW-Umwelttechnik Ltd. The geographical distance between these sites enables to reduce the costs of transportation (which is relatively high due to the large size and weight of the girders) into different parts of Hungary. Personal, technical and quality conditions for the appropriate prefabrication are given at both sites. There are 3×50 m long prestressing beds available for the manufacturing

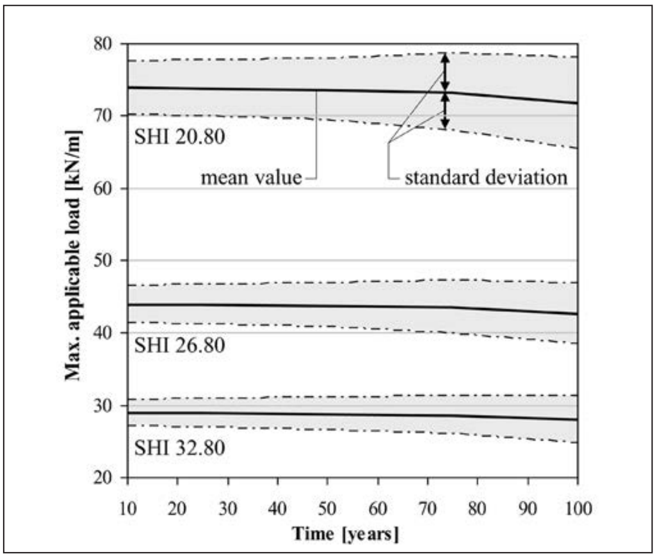


Fig. 8: *Change of the mean value and standard deviation of load carrying capacity in case of different SHI120 beams*

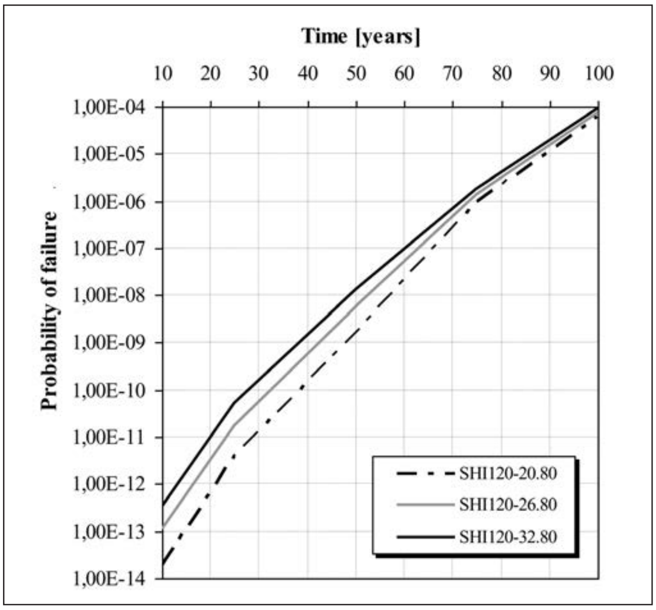


Fig. 9: *Change of the probability of failure in case of different SHI120 beams*

SHI girders at Alsózsolca, and 3×100 m beds can be used at the Majosháza site (*Fig. 10* and *Fig. 12*). The utilised materials are subjected to strict quality control during prefabrication, and they meet the requirements of the Hungarian Highway Engineering Regulations (ÚT). The concrete grade of the beams is C50/60 after 28 days. A picture of the manufactured SHI90 beams at Majosháza site is presented in *Fig. 11*.

The prefabricated bridge girders are transported from the factory to the construction site on public road, using appropriate accompaniment. For the delivery of the girders, vehicles consisting of 13–44.5 m long, three or four-axle, hydraulically steered semi-trailers and coupled tractor units are used (Fig. 13). Transportation of the bridge girders is organised and coordinated by SW-Umwelttechnik Ltd.

4. APPLICATION OF THE PREFABRICATED GIRDERS

Heavy-duty mobile crane can be used at the construction site for the lifting and positioning of the bridge girders. Beams must be supported on rubber bearings, and they must be temporarily fixed until the abutments and the concrete deck

Fig. 10: *Reinforcement of the SHI90 beams*

Fig. 11: *Manufactured SHI90 beams at Majosháza site*

Fig. 12: *Reinforcement detail of the SHI90 beams*

Fig. 13: *Transportation of an SHI bridge girder*

are constructed. These works, completed with other activities connected to the application of the bridge girders (e.g. layout of bridge members, construction of roadways and workspace

Fig. 14: *Construction of the Komárom bypass road: lifting and positioning of the bridge girders*

needed for the transport and lifting of the beams) are performed by SW-Umwelttechnik Ltd if required.

The SHI prefabricated bridge girders were used for the construction of the bypass road around the city of Komárom (*Fig. 14*). The bypass road crosses the highway number 1, as well as an agricultural road and the Győr-Komárom railway line. Both SHI90 and SHI120 girders were used for the construction, the total number of utilized beams was 73 (23 pieces of SHI120-32.80 girders, 20 pieces of SHI90-16.80 girders and 30 pieces of SHI90-18.80 girders). Prefabrication of the beams was done in the summer of 2009 for all three bridges, while on-site construction started at the beginning of 2010. The bridge above the railway line is a continuous structure, the beams for the span above the railway tracks were positioned during the night to avoid the disturbance of the railway traffic. The bridges above the highway number 1 and the agricultural road are simply supported structures, the lifting and positioning of their bridge girders were done in the middle of 2010.

5. CONCLUSIONS

The Department of Structural Engineering at the Budapest University of Technology and Economics was collaborating with the industrial partner SW-Umwelttechnik Ltd. to develop a new range of prefabricated concrete bridge girders using the EN standard. Standalone prefabricated beams, as well as sample bridge superstructures consisting of the developed beams were designed and controlled, and they proved to be safe against the appropriate traffic actions defined by EN.

A probabilistic approach was also used for the more accurate durability design of the bridge girders. The introduced method is considering the creep and shrinkage of concrete, relaxation and carbonation induced corrosion of steel bars and tendons, as well as the decrease of material properties and cross-sectional dimensions in time. Initial material properties and geometrical sizes were determined by laboratory material tests and measurements on manufactured beams. The effect of standard deviations of different input parameters (height and width of cross-section, effective depth of tendons, strength of concrete and tendons) on the standard deviation of load carrying capacity were examined in case of different prefabricated SHI bridge girders. The probability of failure of the analysed girders was calculated in selected time points, and the results of this calculation were presented graphically. It was demonstrated that the probability of failure is increasing over time, and longer girders will fail with higher probability, however the difference between girders of shorter and longer spans is decreasing as time is passing by. Service life estimation of the presented girders was also performed and it turned out that they outperform the requirements of the EN standard even in case of the longest beam. It means that they are fully suitable for the construction of highway bridges.

Besides introducing the design of the bridge girders, their manufacturing, transportation and application were also presented. Experience proves that the SHI bridge girder family represents a product line meeting the latest requirements and expectations.

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FRC CLADDING AT TWO STATIONS OF THE UNDERGROUND LINE 4 IN BUDAPEST, HUNGARY

János Kozák – Béla Magyari

The final construction works of the M4's station at the Tétényi-street had been carried out in 2011 and they contained both the claddings made with fibre reinforcement and the artistic, acoustical, and plain claddings, too.

Currently the fire resistance tests and the trial assembly of the elements of cladding are going on, that have been planned at the preparation of the station at the Fővám-square.

Keywords: underground station, concrete reinforced by polypropylene fibre, cladding, fire resistance, selfcompacting concrete

1. PRELIMINARIES

The structural plans have been prepared by the Főmterv Ltd. – the grade of the building materials have been defined by it, too. As for the coverings of the stations of M4 an outstanding importance was given to the structural elements, such as the walls and the floors, because the covering elements have to be fixed to them. The prescribed grade of the reinforced concrete walls and floors is C 35/45-16/K while that of the reinforcing bars is B 60.50. Thus the reinforcement is ø20/15 in the secondary direction and ø32/15 in the primary one completed with further bars locally if necessary. A technical speciality of the structural works of the stations are moment carrying joints between the wall piles and the foundation plates, that have been carried out by the conic threaded splice of the reinforcing bars according to the LENTON type. The designers *(Pál, 2010), (Schulek, 2008)* reported in detail of the preliminaries, the tendering and of the building, too. The chief contractor of the final building works of both stations was the Swietelsky Magyarország Ltd. As for the cladding it has a many decades long tradition in Hungary *(Magyari, 2005),(Magyari, Tassi, 2007)*. Nevertheless a technical development is in progress even now *(Kozák, Magyari, Tassi, 2011)*.

2. THE CLADDING OF THE STATION AT THE TÉTÉNYI-STREET

2.1 Manufacturing of glued plates

The related drawing had been prepared by the A Plusz Stúdió Ltd. The total area of the cladding is 819 m^2. The elements have been manufactured with the aid of the stand system. The formwork was made from timber of furniture panel type, while the framing was from form steel profiles of L shape. The largest size of the elements was 1200×710 mm at a unified thickness of 10 mm. Nevertheless, other sizes were manufactured, too. The concrete mixture was made with a high strength polypropylene fibre applying white CEM I 52.5 pc at a dosing of 400 kg/m^3. There are coloured cladding plates, too. The type of the colouring agent is Bayferrox.

2.2 The laying of the glued plates

According to the cladding plans the plates were laid in a cross-ruled pattern by the GEO-BAU Ltd. The type of the glue was VIGO C2TE manufactured by the Üveg Ásvány Ltd. typically to tiles. The next step is the jointing to be made with the material of the type MAPESIL AC. The thickness of the glue is to be employed according to the accuracy of the laying.

Fig. 1: Manufacturing of artistic panels

Fig. 2: Manufacturing of acoustic panels

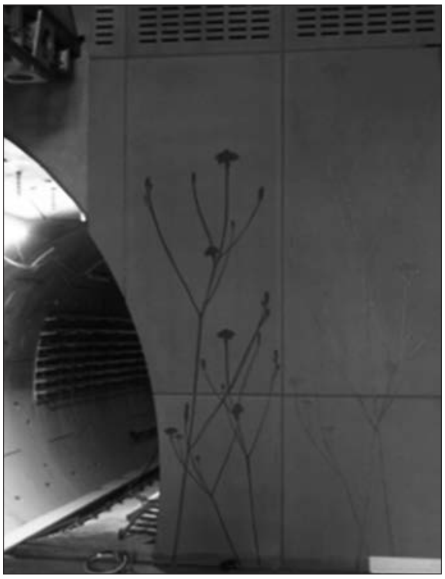

Fig. 3: Artistic cladding

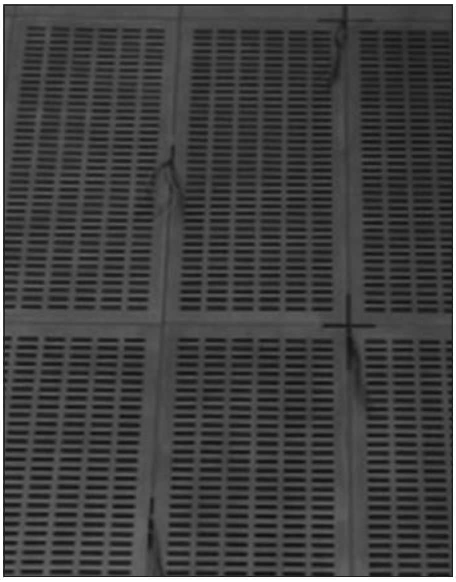

Fig. 4: Acoustic cladding

2.3 The manufacturing and fixation of the cladding plates

The relating plans had been prepared by the ARGOMEX Ltd. The cladding panels were manufactured generally with 30 mm thickness. Only the lower panels had to be 50 mm thick. For the manufacturing a C 15 type ø6 mm mesh of 150 mm quadratic arrangement was laid in the plates. A special attention must be ascribed to the cladding of the end walls because they are furnished with an ornament. Besides, the latter panels have a speciality concerning the manufacturing for the ornament was converted to a PC by the Városi Tájkép Csoport Ltd. Then the basis was furnished with retarding agent of the strengthening process of the concrete having consisted the first layer in the formwork. Next day after removing the formwork with the aid of a water-jet the ornament became visible *(Fig. 1)*. Above the first two lines of the panels another two lines of acoustic panels were fixed. The difficulties of the manufacturing of these panels were manifested by the density of the perforation. Beside the special components for the sake of the selfcompacting of the concrete reinforced by polypropylene fibres a fluidizing agent called Sika ViscoCrete-20 HE was applied in order to obtain a high initial strength *(Fig 2)*. The material of these panels coloured, too. The fixing parts were of PROFIX type having enabled an exact positioning in space *(Fig. 3), (Fig. 4)*. As for the reinforced concrete wall HILTI-type plugs of ø10 mm were applied.

3. CLADDING OF THE STATION OF FŐVÁM-SQUARE

3.1 Design of the cladding panels

The manufacturing plans of the crust panels applied for the platform level of the station Fővám-square had been designed by the PALATIUM M4 PROJEKT Ltd following the order of the ARGOMEX Ltd *(Fig.5)*. Samples had been manufactured partly for the fire resistance test, partly for the installation tests. The elements are in complete compliance with the plans, so in the cast, in fittings and fastenings, as well. In the reinforced concrete walls HALFEN rails (HTA 40/22 HEA) had been placed before casting. Apart from the hammer-head bolts the connection between the HALFEN rails is provided by hot rolled steel profile of L 65×65×5 mm shape and size. The length of the latter is 2600 mm. Threaded bolts serve the exact positioning. The bolts applied are hot-dip galvanized, the steel profiles are coated first with a primary paint then with a fire protection coat. The complete surface of the wall cladding is 1216 m^2.

3.2 Manufacturing of the cladding panels

The concrete is reinforced with polypropylene fibre, the elements are ribbed and reinforced and of selfcompacting method. The panels are manufactured with white cement of CEM I 52.5 pc grade at a dosage of 400 kg/m^3, the colouring agent is of the type Bayferrox, the maximal grain size of the aggregate is 8 mm. Among the contents of the concrete the polypropylene fibres have a special importance, which is a product of the Brugg Contec AG. Its product name is Fibrofor Multi 127, the diameter of the fibres is 0.034 mm, while the length of them is 12.7 mm. For the sake of the fire protection a dosage of 2 kg/m^3 is applied. As for the dosage and the diameter of the fibre applied the results of the material tests were of outstanding usefulness *(Lublóy, Balázs, 2007)*. The formwork of the curved panels is a concrete core, the surface is fashioned with a 3 mm thick rubber plate of MOP 3 circulated by the Taurus Techno Magyarország Ltd. The size of the panel is 490×4780 mm at a curving radius of 3250 mm. The thickness of the plate is 40 mm, the depth of the ribs is 80 mm. The HALFEN rail of HTA 40/22 HEA profile will be placed before casting the concrete and fastened to the reinforcement by welding. Also a test mounting was carried out in site, i. e. at the station of the Fővám-square *(Fig. 6)*, *(Fig. 7)*. The sealing was carried out with the special material called "Polylack Elastic" and produced by the Dunamenti Tűzvédelem Ltd.

4. TESTS FOR THE FIRE RESISTANCE

The tests for the fire resistance were carried out at the plant of the ÉMI nonprofit Ltd at Szentendre/Hungary. With the aid of oil burners a temperature of 950 °C was produced in the test chamber, while the external temperature of the test specimen was checked at several places. The test was carried out in two parts and four test pieces were manufactured for each. In the

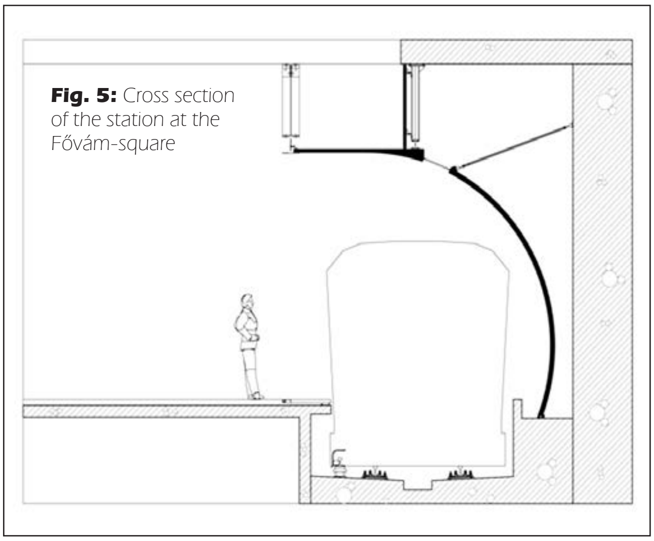

Fig. 5: Cross section of the station at the Fővám-square

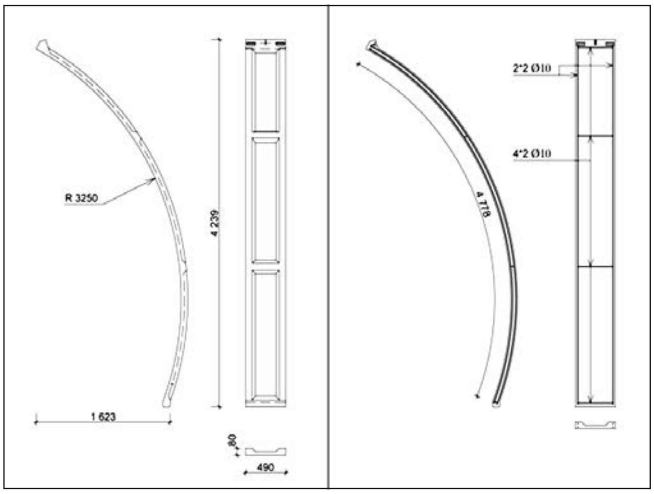

Fig. 6: Plan of the formwork and of the reinforcement for the station of the Fővám-square

first part the panels under the floor were tested, while in the second part the curved ones *(Fig. 8)*, *(Fig. 9)*. Beside the fire resistance of the panels that of the connections were tested, too. Both fire resistance tests were successful *(ÉMI nonprofit Ltd 2012.)* Both the panels and the connections had to produce a one hour's resistance, while for the sealing materials the requirement was half hour's smokelessness. A detailed report is due about the tests and there is no obstacle to pass the serviceability.

5. CONCLUSIONS

At two stations of the underground line No. 4. the high strength concrete panels of polypropylene reinforcement and other elements proved to be suitable both technically and economically and their application is very favourable. It is hoped, that this material will be used frequently in the final phase of the construction. A special attention is due to the successful tests of the fire resistance.

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János Kozák (1953) graduated 1977 as a civil engineer on the Budapest University of Technology and Economics, being now a leading designer both for load-carrying structures and architecture. Since 1991 he is a manager at the ARGOMEX Ltd. Beside architectural commissions and the manufacturing of cladding plates of façades the ARGOMEX Ltd produces cladding panels from a selfcompacting fine quality concrete core of high strength and of fibre reinforcement.

Béla Magyari, Dr. (1942) graduated 1969 as a civil engineer on the Budapest University of Technology and Economics. 1978 he was awarded a dr.technical degree and 1982 that of the Doctor of Philosophy. From 1960 until 1969 he was active as a technical assistant of the ÉTI in Budapest. From 1969 until 1976 he was a department leader at the Co. BÁCSÉPSZER and from 1976 till 1990 he served the Co. DUTÉP as a chief quality engineer. Currently, though he is retired but active as a consulting engineer at the ARGOMEX Ltd. His professional interest and activity include the concrete, reinforced concrete, the splice of reinforcing steel bars and concretes with fibre reinforcement.

Fig. 7: Test mounting at the station

Fig. 8: Preparation of the fire resistance test

Fig. 9: Picture of the element exposed to one hour's fire

INTEGRATION OF DECK AND PAVEMENT FOR SUPERSTRUCURES OF CONCRETE BRIDGES TO REDUCE LIFE-CYCLE COST


Tamás Kovács – Lili Laczák

This paper focuses on the applicability of high-performance concrete to load carrying superstructure of concrete highway bridges in order to improve their cost effectiveness and environmental compatibility. The research suggests that the maintenance cost of a bridge depends first of all on the durability of the pavement system. Consequently, the life-cycle cost-effectiveness of concrete bridges is based mainly on the proper combination of the load carrying superstructure and the pavement system. Three existing concrete decks of different cross-sections in combination with three alternative pavement systems were structurally analyzed and then compared by an approximate life-cycle cost analysis. The structural design was based on the strict consideration of durability requirements related to different pavement systems. It was concluded that for 100 years of design working life as usual for bridges the integrated concrete deck-pavement superstructure has a clear economic advantage against the widely used, separated deck-pavement solutions.

Keywords: deck, pavement, life-cycle cost, life-cycle design, high performance concrete

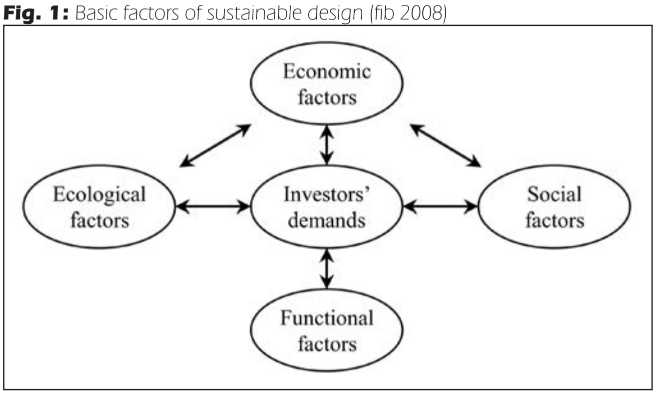
1. INTRODUCTION

1.1 Sustainability in design of civil engineering structures

Engineering structures are structures of high importance and significant cost. They have well defined function, fulfil actual demands and are required to have long service life. In designing these structures for long-term, global tendencies have to be taken into account. The engineers of the 21st century have to aware of a new way of thinking that focuses on sustainability. This comes from the investors' intention that includes not only functional and economic, but ecological and socio-cultural demands *(Fig. 1)*.

These criteria are enforced by the global ideas of sustainable development and environmental compatibility that are accepted by the whole society. At the same time ecological and social requirements often cause higher costs consequently they are contradictory to the economical interests. The designer should reduce this contradictory effect by finding the optimal structural system as well as making an effort to obtain an economic as well as socially and ecologically satisfactory solution.

Fig. 1: Basic factors of sustainable design (fib 2008)



1.2 Life-cycle design

The idea of life-cycle design was the consequence of these new demands. During this process the structural (and also architectural) design, construction, maintenance, operation and demolition of the structure are observed at the same time. Consequently, future economic, ecological, and social effects are simultaneously analyzed.

In recent years, rapid development can be observed in two areas of life-cycle design of civil engineering structures. One direction is the *service life design*, which focuses mainly on durability issues and related design approaches to extend the design working life of structures up to 100 years or even longer. The other direction is the *life-cycle cost analysis* (LCCA) that analyses the cost effectiveness of structures and finds design strategies, by which the total life-cycle cost of a structure can be minimized during its service life or a defined service period.

1.3 Application of life-cycle design for bridges

Majority of structures, to which the application of life-cycle design is desirable, are parts of the traffic infrastructure. These structures are required to work on acceptable technical level during a relatively long service life and need relatively high costs during this period.

On the bridge construction field the service life design led to the application ultra-high performance concrete (UHPC, strength class: f_{ck}>150 N/mm^2) in superstructures of bridges in Northern and Western Europe as well as in North-America (Stengel & Schießl 2009). In Norway the design service life of bridges was extended up to 120 years. The application of UHPC is one of the hottest research topics in the structural engineering field.

In the past decades, significant effort was given to provide life-cycle cost basis behind the design of roadways. By the use of economic valuation processes, the objective comparability of asphalt- and concrete-based pavement systems became possible. However, small attention was paid to include bridges into these analyses as well as to carry out life-cycle cost analysis of bridges in itself. This was the basic idea behind the bridge life-cycle cost analysis (BLCCA) research program (Hawk 2003), from which time this field has begun to develop dynamically in the USA, Canada and in Europe (Strauss et al. 2008).

1.4 Focus of the research

The research deals with the applicability of high-performance concrete (HPC) to the superstructure of concrete highway bridges in order to improve their cost effectiveness and environmental compatibility. This paper focuses on two areas:

a) How the selection of pavement system for concrete road bridges influences the structural design of the load carrying superstructure.
b) Economic comparison of different concrete deck-pavement combinations for road bridges.

The applicability of bridge decks made partly or entirely of HPC without water insulation and any pavement layer at the top as an alternative to either asphalt-based or concrete-based traditional pavement systems built on water-insulated decks is the subject of section a). The suitable structural design strategy should reflect on the different environmental exposure and the related durability requirements for different pavement systems as well as on the structural differences (concrete strength, geometrical sizes, amount of reinforcement and prestressing) of different deck-pavement solutions. A comparative study of many deck-pavement solutions as combinations of three existing concrete decks of different, widely used cross section and three pavement systems was carried out.

In order to get an overall picture of not only the static, but the economic advantages of HPC in bridges, an appropriate cost analysing method is necessary. To base the cost analysis on realistic data, structural re-design of the above deck-pavement superstructures was carried out. Their cost analysis was extended to the full service life of structures.

2. APPLICATION OF HPC IN INTEGRATED DECK-PAVEMENT SUPERSTRUCTURES

2.1 Use of HPC to structural elements to improve durability of bridges

HPC is considered as concrete that provides higher resistance against certain effects than traditional concrete (i.e. normal performance concrete, NPC). The relatively higher performance is the consequence of the improvements made by technological measures in one or more of the concrete properties such as workability and compactibility, initial or long-term strength, permeability, proneness to early-age cracking due to hydration heat, shrinkage etc. In majority of cases the primary motivation behind high performance is the improvement of durability of the structure. Nevertheless, according to experience, a durable HPC designed against severe environmental attacks produces with ease a minimum strength class of C50/60 without any special concrete technological measure. This combination of high strength and high performance against environmental attacks makes HPC suitable for several bridge applications.

Road (and especially motorway) bridges are operated under high traffic and extreme environmental conditions but maintained with possible minimum cost during their service life. Structural elements, which are exposed to aggressive deterioration effects (abrasion due to traffic, direct contact with water containing high concentration of de-icing salt), are able to fulfil the relevant durability requirements only if they are made of HPC. Otherwise, their maintenance cost became unacceptably high for the owner.

By the use of the latest achievements in field of concrete technology, the factory production of HPC is technically solvable. However, the difficulties of site work with HPC regarding transportability, workability, compactibility, curing generally make its application (much) more expensive than that of NPC. Therefore, the selection of structural elements in a road bridge, for which the application of HPC may be economic allowing for its relatively high initial and relatively low maintenance cost, requires close attention. This process generally requires the economic comparison of the alternative structural solutions. The use of LCCA offers an objective platform for this comparison.

2.2 Use of HPC as road pavement

In continental climatic regions the deck slabs of road bridges are traditionally waterproofed and then an asphalt- or concrete-based pavement system is built at the top. The total thickness of the pavement system is about 150 mm (corresponding to ~3.6 kN/m^2 superimposed dead load) for usual asphalt-based systems and between 250 and 300 mm (~6.0-7.5 kN/m^2) for concrete-based systems.

The combination of relatively high strength and durability makes HPC suitable for a special application in bridges where the top surface of the superstructure is used as an integrated concrete pavement in direct contact with the wheels of crossing vehicles. In this situation the deck slab is in one hand part of the load carrying structure and on the other hand functions as road pavement. To fulfil structural, traffic-safety as well as durability requirements simultaneously in long-term, high concrete strength (both compressive and tensile) is needed to provide sufficient structural and abrasion resistance as well as no-crack condition, furthermore improved watertightness and low permeability of the top surface is required against soluted de-icing salt and freeze-thaw attacks. If these conditions are met then both the waterproofing of the deck and the whole separated pavement system may be eliminated. Even allowing for the expensive site work with HPC, this may result in significant cost saving in both the construction (initial) and in the maintenance cost which makes the integrated HPC deck-pavement superstructures very competitive against the traditional, structurally separated deck-pavement solutions.

Earlier in the USA many integrated HPC deck-pavement superstructures were built, however majority of them is not affected by freeze-thaw attack and, consequently, by de-icing salting in wintertime. In Hungary the first integrated HPC deck-pavement superstructure (S65) was built for

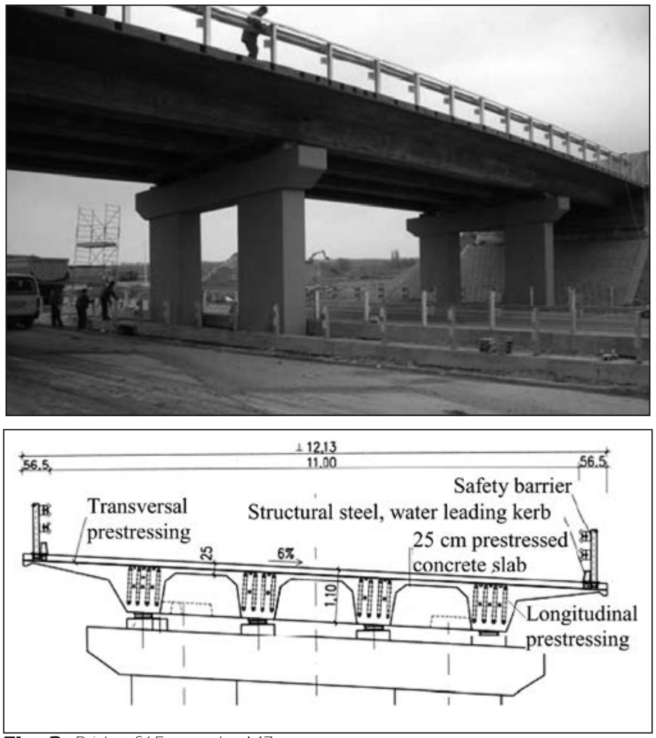


Fig. 2: Bridge S65 over the M7 motorway

experimental purposes over the M7 motorway in 2006 *(Fig. 2)*. The superstructure lies along a curved horizontal layout in varying (2.43-0.72%) longitudinal slope and constant 6% superelevation with 11.0+17.5+18.5+13.0 m spans. The 250 mm thick deck slab was monolithically built on a system of transversal and longitudinal beams (gridwork). The longitudinal beams as well as the deck slab in the transverse direction were internally post-tensioned by unbonded tendons.

2.3 Design requirements for integrated HPC deck-pavement superstructures

Based on the experiences of the S65 project, a new technical specification regarding the design and construction of integrated HPC deck-pavement road bridge superstructures has been elaborated (Design Guide 2010) which also adopted the relevant design rules of the Eurocode (EN 1992-2 2005). The basic point of design is the accordance of the necessary durability requirements with the specified structural requirements.

Most of the *durability* requirements are aimed to be fulfilled by concrete technological measures. The design task is to provide a sufficiently dense, uncracked top surface for the deck which is capable of preventing aggressive agents to penetrate into the concrete as well as hard enough to resist against abrasion effects of road traffic. For this purpose Design Guide (2010) sets the maximum diameter of aggregate to 20 mm and specifies XD3 (chloride resistance), XF4 (freeze-thaw resistance), XV3 (watertightness) and XK2 (abrasion resistance) exposure classes for concrete according to EN 206-1. The minimum strength class shall be at least C50/60; the concrete cover is set to 60 mm on the basis of structural class 4. To avoid technological cracks, the application of shrinkage compensating admixture and the thermal insulation of formwork during hardening of concrete to prevent thermal shock is recommended.

From *structural* point of view the design task is the avoidance of the formation of structural cracks. The ULS requirements are not affected by crack limitation and can easily be fulfilled by the specified minimum concrete strength. However, to ensure uncracked top surface of the deck under service conditions the following strict tensile stress limitation in both longitudinal and transverse direction is specified at the top surface:

- tensile stress is limited to tensile strength of concrete (f_{ctm}) under the characteristic combination of actions (no crack requirement),
- no tensile stress is permitted ($\sigma_c \leq 0$) under the frequent combination of actions (decompression requirement).

Because decks of usual road bridges are subjected primarily to flexure or combination of flexure and normal force under traffic loads and permanent gravity actions (dead load), the above stress limitations make necessary the application of prestressing for majority of concrete decks. The longitudinal prestressing is a necessity for ordinary monolithic, slender continuous decks to control longitudinal tensile stresses above internal supports (see *Fig. 2, 5a, 5c)* but may be unnecessary for short, simply-supported superstructures (see *Fig. 5b*) where the top surface remains under compression and, consequently, uncracked under the relevant combinations of actions. The intensity of transversal tensile stresses is considerably influenced by the cross sectional shape. For thin (<300 mm) deck slabs built on horizontal beams (see *Fig. 2, 5a, 5c*) the limitation of transversal tensile stresses generally requires transverse prestressing. However, for solid (see Fig. 5b) or voided slabs with nearly constant, >300 mm thickness may have sufficient resistance to remain uncracked due to transversal forces.

If applying appropriate water drainage system at the top of the deck the environmental exposure of the bottom faces of the superstructure is independent of the fact whether the top surface is waterproofed. Therefore, there is no need to apply HPC to structural parts, for which the relevant durability requirements can be fulfilled with NPC. Consequently, the significant cost of HPC may considerably be reduced for integrated deck-pavement superstructures where the construction process (casting) of structural parts made of HPC (deck slab) can be separated (by e.g. concreting in two steps) from that made of NPC (supporting gridwork system). In practical cases, the construction technology of usual box girders normally follows this process while that for superstructures similar to that in *Fig. 2* can be adjusted accordingly without problem. For solid or voided slabs, for which the two-step casting is impractical or technologically unrealistic, the application of HPC in full thickness may be the most favourable option.

3. STRUCTURE-ORIENTED LIFE-CYCLE COST ANALYSIS

In carrying out LCCA for an *existing* bridge, the remaining service life or a certain period of it is observed. Potential measures related to maintenance, which, in this interpretation, also includes operational measures and repair interventions, are systematically planned during the observed period; their cost are estimated and then the estimated costs are discounted to net present value. The main focus is on the minimization of maintenance cost during the observed service life. This is carried out by keeping the above system of maintenance measures up to date (by e.g. implementing new examination techniques, applying new materials and protection systems etc.) and by optimizing the key parameters (e.g. unit costs of applied measures, frequency of regular checks and structural interventions etc.) of the total cost function. As a result, the related maintenance measures are adjusted or re-planned accordingly but the existing structural system (including both the load carrying structure and the pavement system), as an initial data that has pre-determinative effect on its maintenance, cannot be modified (in general cases).

3.1 Life-cycle cost analysis to support design

However, when applying LCCA to a *new* bridge, which is under (preferably conceptual) design and in full before construction, the maintenance cost of service life, as an addition to construction cost and dismantlement cost, is only one of the three main components of the life-cycle cost *(Fig. 3).*

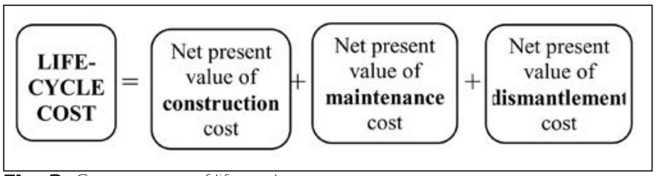


Fig. 3: Components of life-cycle cost

Aspects explained in Sec. 2.1 and 2.2 emphasized the significance of the selection of the structural and the pavement system on the foreseeable life-cycle cost of bridges. Assuming a usual concrete girder bridge to be designed with given width and span arrangement, subsequent cost analyses concluded that

- the effect of the selection of the load carrying structure (focusing mainly on cross-sectional form and construction technology) is extremely high on the construction cost but relatively low on the maintenance cost;
- the effect of type of the pavement system on the construction cost is not negligible but also not significant, however, that on the maintenance cost is considerable.

Both the structural and the pavement system is set by decisions made early in the (conceptual) design stage. Therefore, structural and pavement design should be considered as a basis, from where life-cycle cost minimization can be originated *(Fig. 4).*

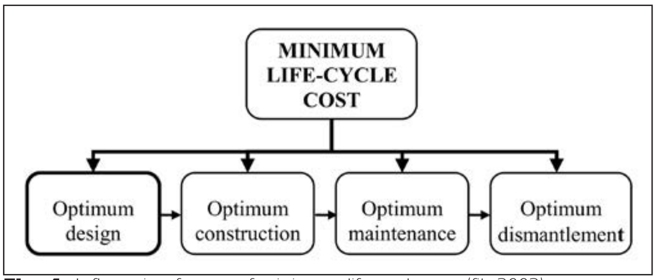


Fig. 4: Influencing factors of minimum life-cycle cost (fib 2002)

Up to present, majority of researches in this field have hardly dealt with the consequences of structural design on life-cycle cost. In contrast to that, in this research LCCA is definitely used as a mean that provides platform for the economic comparison of different structural systems in order to minimize life-cycle cost of bridges.

3.2 Partial life-cycle cost analysis

In calculating life-cycle cost of a bridge according to Figure 3 during the design stage, the net present values of all costs calculated to the time of construction are generally summarized. However, many costs related to maintenance and especially to dismantlement are difficult to be estimated because extrapolation of many economic factors (e.g. the inflation) to the end of the intended design working life is needed.

It is also important to make clear that according to Sec. 1.4 the primary aim of this paper is to point out the potential economic advantages of integrated HPC deck-pavement superstructures in comparison with traditional separated deck-pavement solutions, and not to determine the total life-cycle cost of integrated bridges. For this reason, Sec. 4 demonstrates the results of a partial LCCA carried out for different deck-pavement superstructures which is based on approximated costs and, with reference to Sec. 3.1, allows for

- the construction costs of the whole superstructure (including both the load carrying structure and the pavement system) (referred as "*construction cost*" in the following) and
- the maintenance cost of the pavement system (referred as "*maintenance cost*" in the following),

but neglects

- the maintenance cost of the load carrying structure,
- the dismantlement cost of the whole superstructure.

Owing to the omitted costs this procedure does not provide life-cycle cost but can efficiently be used for decision making in structural and pavement design of concrete bridges.

4. CASE STUDIES FOR PARTIAL LIFE-CYCLE COST ANALYSIS

Partial life-cycle cost analyses have been carried out on 9 fictitious concrete road bridge superstructures, which were combinations of three different load carrying decks with three different pavement systems.

4.1 Prototype bridges

The 9 investigated deck-pavement combinations were originated from three existing bridges. Their main characteristics are summarized in Table 1 and the belonging cross-sections are shown in *Fig. 5a-5c.*

Table 1: Characteristics of the "prototype" bridges

	Bridge type		
	A	B	C
Span lengths	5×36 m continuous	10 m simply-supp.	24+3×30+24 m continuous
Cross-section	twin box with diaphragms	solid slab	deck slab on gridwork
Span/depth ratio (max.)	14.6	14.3	23.5
Total width	20.7 m	10.9 m	11.4 m
Concrete	C40/50	C30/37	C40/50
Prestressing	longitudinal (and transversal): internal bonded post-tensioning	no	external (long.) and internal (transv.) unbonded post-tensioning
Construction technique	incremental launching	in-situ scaffolding	in-situ scaffolding
Original pavement	260 mm jointed plain concrete	150 mm asphalt	150 mm asphalt

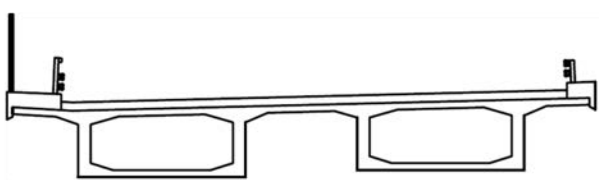


Figure 5a: Scheme of cross-section for Type A

Figure 5b: Scheme of cross-section for Type B

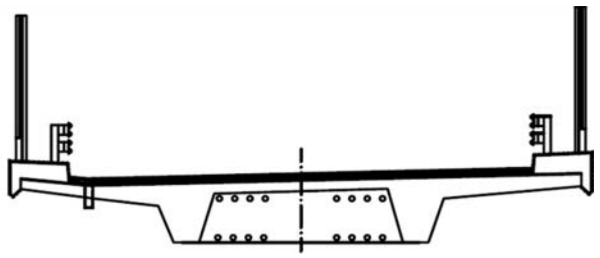


Fig. 5c: Scheme of cross-section for Type C

The fundamental difference between the "prototype" bridges is the form of their cross-sections. The selected forms are widely used in practice and represent the majority of girder bridge cross-sections. The ratio of area of the deck slab to that of the whole cross-section is 0.56 for Type A, 1.0 for Type B and 0.64 for Type C.

4.2 Description of the investigated deck-pavement superstructures

Each of the investigated nine (=3×3) fictitious deck-pavement systems was composed of the load carrying structure of one of the "prototype" bridges and one of the following pavement systems:

I. separated 150 mm thick asphalt-based pavement system built on waterproofed deck slab,
II. separated 295 mm concrete-based pavement system (260 mm pavement + 35 mm protecting layer) built on waterproofed deck slab,
III. HPC pavement integrated with deck slab without waterproofing (no separated pavement).

The application of HPC in load carrying structures took place on the basis of *durability* considerations according to Sec. 2.3. For superstructures with separated pavement (Type I and II) the load carrying structure was assumed to be made fully of NPC. In case of integrated HPC pavement (Type III) HPC was assumed only to the deck slab for the Type A and C decks and to the full thickness slab for the Type B deck. The structural parts supporting the deck slab for the Type A and C decks (webs for Type A and C and bottom slab for Type A) were assumed to be made of NPC.

4.3 Strategy of structural re-design

Using the static calculations of "prototype" bridges as baseline, the structural analyses for the nine investigated deck-pavement superstructures were carried out according to the Eurocode (EC) taking into account the differences in self-weight of pavement and in the considered structural requirements between each "prototype" and the belonging three investigated deck-pavement systems accordingly.

The self-weight of Type I, II and III pavement was taken as 3.6, 7.3 and 0 kN/m^2, respectively and summarized with other superimposed dead loads (kerb, safety and noise barriers etc.).

As usual for ordinary concrete road bridges, the structural resistance of all investigated decks was determined by the crack-related serviceability (SLS) criteria instead of the ULS requirements. In terms of crack limitation two main cases were distinguished depending on the assumed pavement type. For decks with integrated Type III pavement, tensile stress limitation according to Sec. 2.3 was applied to prevent the formation of structural cracks at the top surface. The same crack limitation was specified to other deck surfaces as for the application of separated Type I or II pavements. For decks with separated Type I or II pavements, the specified crack limitation differed depending on the protection level of the applied prestressing. For the Type A deck equipped with bonded prestressing, decompression requirement was applied under the frequent combination of actions. For the Type C deck, which was equipped with unbonded prestressing, and for the non-prestressed Type B deck, crack width was limited to 0.3 mm under the frequent combination of actions.

This crack limitation required the application of prestressing in transverse direction for deck slabs of all deck types in combination with Type III pavement. Internal post-tensioning was assumed for this purpose with the same bond characteristics as for the corresponding longitudinal post-tensioning (i.e. bonded for Type A and B and unbonded for Type C). For the Type B deck in combination with Type III pavement transverse post-tensioning positioned close to the top surface of the deck was necessary only in the 1.5 m vicinity of supports because it was verifiable that no cracks arise in ULS in the inner deck region. Concerning the longitudinal post-tensioning, the necessary amount of tendons was calculated according to the relevant crack limitation criteria without changing the tendon layout of the corresponding "prototype".

The key structural parameter, which governed the above crack limitation, was selected on the basis of structural, economic and practical considerations. For the Type A and C decks, crack limitation was governed only by the amount of longitudinal prestressing, therefore no change in the geometrical sizes of the cross-sections and in the amount of reinforcing steel compared to the "prototypes" was made. However, for the Type B deck, the fulfilment of crack width limit by the adjustment of only the amount of reinforcing steel would have resulted in extreme and unrealistic reinforcement meshes. In order to keep the amount of reinforcement between structurally adequate and practically reasonable limits, the amount of tensile reinforcement was set identical to that in the "prototype" and the slab thickness was changed in such a way that the calculated crack width was equal to the given limit.

4.4 Calculation of approximate life-cycle costs

Approximate life-cycle cost as sum of *construction cost* and *maintenance cost* according to Sec. 3.2 for all investigated deck-pavement systems was calculated assuming 100 years of design working life.

Construction cost included all related costs of both deck and pavement. Generally, it was determined as product of the required amount of structural components (concrete, steel, asphalt, waterproofing etc.) and the belonging unit price. The required amounts were calculated on the basis of either the results of structural analysis (e.g. volume of concrete, mass of steel) or usual technological solutions (e.g. thickness of pavement layers). Unit prices included all expenses of the related structural component incurred until it was built into its final position (e.g. manufacturing, transportation, placing and curing costs for concrete). The unit price of HPC was assumed to be double of the unit price of NPC.

Maintenance cost included costs of all usual, scheduled and non-scheduled technological measures (including also repair, renovation and reconstruction) that were required to keep the technical level of pavement above an acceptable minimum limit for 100 years. For simplification purposes, the full maintenance cost of decks and that of traffic control and traffic dislocation during maintenance interventions were excluded according to Sec. 3.2. Based on operating experiences, the length of service life was assumed as 15 years for the Type I (asphalt-based) pavement and as 35 years for the Type II (concrete-based) pavement. After these periods complete reconstruction was assumed, in frame of which the complete change of the whole pavement system including waterproofing, its protection and all pavement layers as well as of the adjacent bridge equipments (kerbs, barriers, lightening, etc.) was required. The 100 years long service life of Type III pavement was divided into three periods of equal length. During the first period (30-35 years) no scheduled maintenance measure was assumed. However, abrasion of top surface due to traffic may result in insufficient friction therefore mechanical roughening of the top surface was scheduled at the end of the first period. After the next 30-35 years instead of re-roughening, which would cause excessive decrease of the top concrete cover, surfacing the top by a 50 mm thick asphalt wearing layer was assumed. This asphalt wearing layer was then assumed to be changed in every 15 years, whose costs were uniformly distributed within the last period, until the end of service life. Minimum additional maintenance work was considered in the second and third periods.

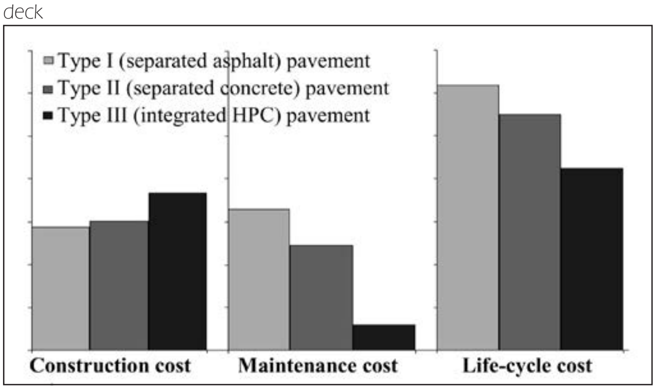
Unit *construction* and *maintenance* prices were obtained from relevant industrial partners.

4.5 Comparison of life-cycle costs

Figure 6 shows the ratios of calculated *construction* and *maintenance* costs as well as that of *approximate life-cycle cost* for three out of the nine investigated deck-pavement superstructures; each of the three consists of identical (Type A) deck and one out of Type I, Type II and Type III pavement. *Fig. 6* is used to comparison of typical costs for superstructures with different pavement systems and identical deck.

In terms of separated pavements: in comparison with Type I (asphalt-based) pavement, higher cost save can be achieved during the maintenance of Type II (concrete-based) pavement than its extra construction cost which makes the application of Type II pavement on bridges more economic than Type I pavement over 100 years of design working life. This was concluded also from the comparative cost analysis of asphalt and concrete pavement systems applied on roads allowing for 35 years service period (Keleti 2010) which underlines the practical and economical advantage of leading the concrete pavement of connecting roads through bridges. In comparison of separated (Type I and II) pavements with integrated (Type III) pavement, the economic advantage of integrated pavement is obvious due to the significantly less maintenance cost of Type III pavement.

Fig. 6: Calculated costs of deck-pavement superstructures with Type A deck



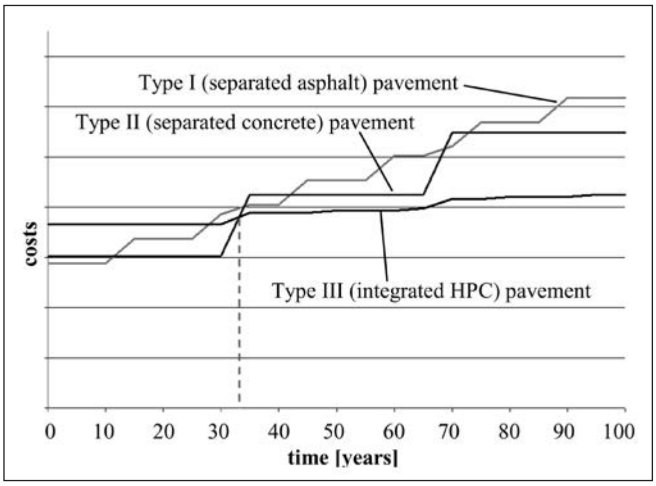


Fig. 7: Calculated life-cycle costs as function of service life for deck-pavement superstructures with Type A deck

Figure 7 shows life-cycle cost as function of time after construction for the same deck-pavement systems as for Figure 6. Values of columns in the first and last group in Figure 6 correspond to the initial (belonging to time zero) and the final (belonging to 100 years) cost values, respectively, in *Fig. 7*. This plot helps in estimating the pay-out periods of each pavement type in comparison with the others for the Type A deck. Sudden steps in lines indicate major, scheduled interventions (reconstructions). As shown, both the frequency and the measure of each reconstruction is significantly less for the integrated Type III pavement in comparison with separated Type II and especially Type I pavements. The Type II pavement requires major interventions of the highest cost. In terms of separated pavements: The pay-out period of Type II (concrete-based) pavement against Type I (asphalt-based) can be estimated between 30 and 40 years to the time of the first major upgrade of Type II pavement. Later life-cycle cost of Type II pavement gradually goes less than that of Type I. By the end of design working life about 10% save in life-cycle cost may be achieved by the application of Type II pavement instead of Type I. However, this saving increases up to about 30-35% against Type I and to 20-25% against Type II pavement if integrated HPC pavement is applied from the beginning. The pay-out period of Type III pavement (indicated by the dashed line in Figure 7) has obtained as about 33 years against Type II and about 28 years against Type I pavement, which are not more than one-third of the 100 years intended design working life for bridges. The longer the intended service life the greater life-cycle cost saving derives from the integration of HPC pavement with the deck.

Similar tendencies have obtained for deck-pavement systems with Type B or Type C deck.

5. CONCLUSIONS

Structural re-design with the consideration of three alternative pavement systems and following partial life-cycle cost analysis have been carried out on three existing, "prototype" concrete bridge decks. The considered pavements were: one asphalt-based and one concrete-based system; both structurally separated from the deck as well as one, integrated HPC deck-pavement system. The main goals of the study were

- to analyze the applicability of the integrated HPC deck-pavement system as bridge superstructure from structural and economic points of view in comparison with traditional, separated asphalt- and concrete-based pavement systems;
- to perform an economic comparison between integrated and separated (asphalt-based and concrete-based) deck-pavement systems by estimating their life-cycle costs.

The applied "partial" life-cycle cost analysis was able to demonstrate the long-term economic advantage of the investment strategy for concrete bridge decks, which focuses on improved durability design in order to significantly reduce the necessary maintenance effort despite the higher construction cost of the structure. In comparing the integrated HPC deck-pavement system (HPC system) with traditional NPC decks combined with structurally separated, pavement systems (NPC system) the following conclusions can be made:

- The application of HPC system helps in reducing the dead load of the superstructure but makes necessary the consideration of more sever durability and, consequently, structural requirements in design in comparison with NPC systems.
- The improved durability and structural requirements can be fulfilled by special concrete technological measures and structural solutions that results in definitely higher construction cost for the HPC system compared to the NPC systems.
- However, due to the significantly less maintenance cost for the HPC system, the saving, which can be achieved in the life-cycle cost during 100 years of service life, is on average 25-35% compared to NPC decks with asphalt-based pavement and 10-20% compared to NPC decks with concrete-based pavement.
- The pay-out period of the HPC system has obtained as about 33 years against NPC decks with concrete-based pavement and about 28 years against NPC decks with asphalt-based pavement.

The above results underline the exploitable structural benefits of the application of the integrated HPC deck-pavement systems.

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THE ELECTRICAL RESISTIVITY OF CONCRETE


Tamás K. Simon PhD – Viktória Vass

The determination of the corrosion risk of embedded steel in a non-destructive way is a common necessity. The corrosion risk significantly depends on the different harmful materials and their accession into the concrete. The higher the concrete porosity is, the easier for these corrosive materials to get near to the embedded steel, consequently the durability of the structure becomes limited in time. The apparent porosity measurements of concrete can be carried out by analyzing the conductivity in a function of moisture content, or it's opposite: the electrical resistivity.

Keywords: concrete durability, apparent porosity, electrical resistivity of concrete

1. INTRODUCTION

During the continuous expansion of Budapest, the development of unused lands, the rearrangement of public places and the construction of deep garages called the necessity of exploring underground pipelines and their present state diagnosis.

In the passing decades a significant amount of cement bounded potable water pressure pipes were used in the city, a certain amount of which was made out of prestressed concrete. To estimate their present state and expectable future, the possible application of the non-destructive concrete measurement methods presented themselves. The condition of these pipes is currently unknown, their failure, because of the 6-8 bar internal water pressure, occurs with a sudden phenomenon which is actually like an explosion.

The necessity of estimating the condition of the pipes brought the demand to draw conclusions regarding a section of the pipeline, which may be considered as one lot, based on the survey of a relatively small (1-2 meters long) piece of pipe. For this estimation the measurement of electrical resistivity of concrete has a great potential, which is a rarely used non-destructive method. During the measurement only a few meters long pipe section should be excavated, which is very important, because the measurement can be done under operation. The measurement of resistivity of concrete happens with a method adapted from soil mechanics.

The purpose of this paper is to introduce the measurement of the electrical resistivity of concrete, the applicability and limits of this method and the theory behind.

2. MEASURING THE ELECTRICAL RESISTIVITY OF CONCRETE

2.1 The soil-mechanical background

At a certain degree, every material, thus soils and rocks transmit electric current. So first the soil mechanical engineers developed a method for measuring electrical resistivity and utilized the obtained results. Conductivity or its reciprocal the electrical resistance strongly depend on the structure of soil, the size and distribution of pores in it, the incidental water content and the amount of dissolved salts in it.

With conductivity measurements in soil the location of inhomogenities can be predected by the evaluation of measured anomalies, like former trenches or cellars, if the backfilled materials have different structure than the original soil.

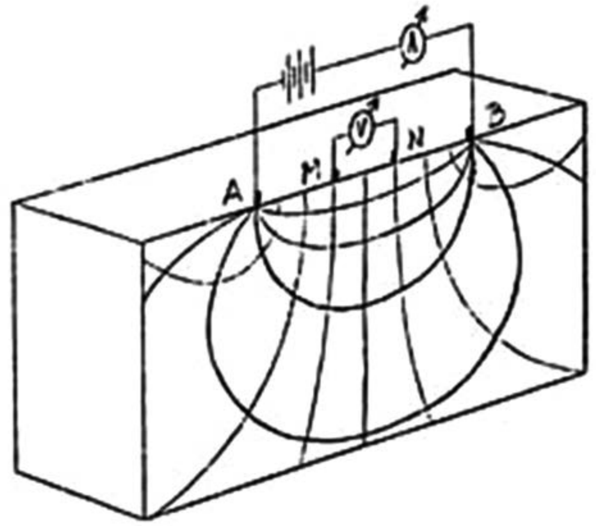


Fig. 1: Theoretical arrangement of direct current measurement

The direct current conductivity measurement is performed according to *Fig. 1:* with the current electrodes A and B a direct current signal is emitted into the soil, and the evolving potential is measured with the potential electrodes M and N. Using Ohm's law the specific resistance can be determined:

$$\rho = 2\pi a \frac{U}{I},$$

where: ρ electrical resistance of soil,
a distance of electrodes,
U voltage between the potential electrodes,
I magnitude of direct current,
π Ludolph's constant (3,14).

It can well be seen that the current flows in a large volume, so from the intensity current and the potential only a specific resistance which is featuring a larger volume can be calculated.

When the distance between the current-entering electrodes is small, the current flows only in the upper layers essentially and the resulting apparent specific resistance roughly equals to the resistance of the upper one. When this distance is big, the current flows in the deeper layers as well, and such resistance derived from the measurements can be compared to the one of lower layer.

2.2 Adaptation to the field of concrete measurements

Based on the above mentioned principles of soil mechanics the methods used for examining the surface resistance of concrete were developed. For in-situ measurement of concrete a four-point resistivity meter is used, which is a smaller, adopted for concrete version of the resistivity meter as the one used in soil mechanics.

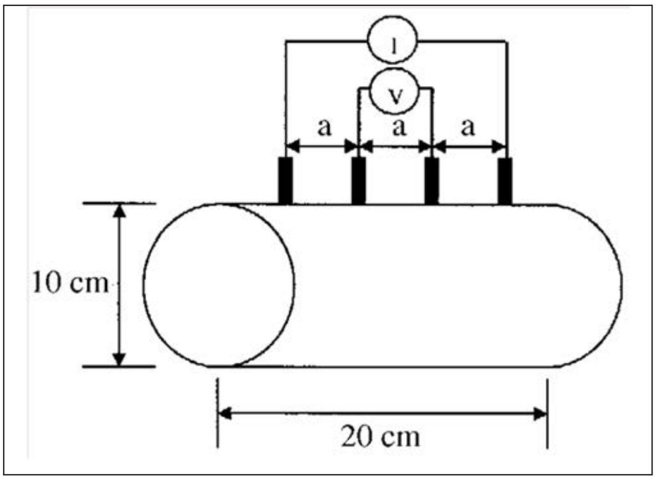


Fig. 2: The four-point Wenner probe resistivity meter

In practice it is obvious to use a commonly available four-point Wenner probe resistivity meter. These meters use a probe with four terminals set up in a linear array with a distance "*a*" between the probes *(Fig. 2)*. The two outer probes are for the introduction of the current, whereas the two centre electrodes are the voltage measurement points. When the probe touches the concrete surface, the electronic control unit circulates the test current and measures the potential between the inner points. The electronic contact is made of foam pads, which are to be saturated with water preliminarily for electrical conductivity.

Nowadays several manufacturers offer instruments for this purpose, for us the RESI type resistivity meter made by Proceq was at service *(Fig. 3)*. The resistivity meter gives the values of resistance (ρ_c) in units of kΩcm. Furthermore it estimates the reliability of the results in %.

Fig. 3: The RESI resistivity meter

3. ELECTRICAL RESISTIVITY OF CONCRETE

3.1 As the function of porosity

The portland cement based concrete is a multicomponent, microporous, microstructure sensitive construction material. Porous materials absorb water from the air. The equilibrium of the water content of the porous material with the moisture content of the air (relative humidity) is described by the sorption isotherm. Up to a relative humidity (RH) value of 40% the uptake of water is a pure sorption process. This water is not mobile, not free and is strongly bonded to the inner surface of the cement paste. At higher relative humidity values (RH greater than 40%) additional water is absorbed by capillary condensation. This part of the porosity is called apparent porosity.

The pores in concrete are randomly distributed, differently sized and irregularly connected to each other. The movement of water and different ions in these curvy channels is governed by permeability, absorption and several diffusion mechanisms. The cement-based materials consist of air-filled voids, micro-cracks and internal surface pores between the C-S-H gel. Several types of pores are known. Formation of air pores is unavoidable during the mixing and compaction of fresh concrete. The mixing-water filled channels connected to each other and the surface of concrete are called capillary pores. The gel pores are too small to become saturated during the hydration, their presence is the function of the environmental humidity, the porosity and the moisture content of concrete.

Some parts of the pores are not in connection with the concrete surface, these are called closed pores. The other parts of pores are connected to the surface through the capillaries, so these are open. The porosity derived from the amount of open pores is the apparent porosity.

The resistivity of mortar and concrete is a function of the microstructure of the cement paste (pore volume, distribution of the pore radii), the moisture and soluble salt content, and the temperature. The microstructure is affected by various factors such as water to binder ratio, the cement content, the degree of hydration, and the type and amount of admixtures. During an extensive investigation the resistivity of several conventional and ready mixed mortars for cathodic protection has been measured for two years (Hunkeler, 1993). During the evaluation of the results it could be found that the conductivity of the materials decreases dramatically with the decrease of relative humidity. In the background of this was assumed that in case of high relative humidity the carbonation practically stops. During the carbonation at lower relative humidity the OH^- concentration decreases, which increases the conductivity of pore water.

Therefore the electrical resistivity of concrete is strongly influenced by the porosity, the relative humidity and the resistivity of the pore liquid (water in pores containing dissolved salts).

3.2 In function of the type and concentration of dissolved salts in pore water solution

The quality of pore water solution is strongly affected by the age and type of concrete, the origin and type of cement, and the water/binder ratio. The major components of the pore liquid are K^+, Na^+ and OH^- and in smaller extent the Ca^{2+} and the SO^{2-} ions. Till now the effect of the presence of chloride ions on the resistivity of concrete has been investigated only by very few authors. The increase of the conductivity of pore water due to the increasing amount of chloride is balanced by a simultaneous decrease in the OH^- concentration.

4. THE COHERENCE BETWEEN THE RESISTIVITY OF CONCRETE AND THE CORROSION OF REBARS

Resistivity represents the relative current conductivity of an intermediating carrier device. The pores of concrete could contain water with dissolved salts in it, therefore concrete becomes electrically conductive. By knowing the electrical resistivity of concrete we can draw important conclusions regarding the corrosion risk of embedded rebars. The speed of corrosion process *(Fig. 4)* is a function of several parameters, out of which the most important ones are the porosity and thepore structure of the concrete.

The specific resistivity of concrete is the most important factor in hindering the propagation of corrosion *(Fig. 5a and b).*

At lower relative humidity, i.e. at lower moisture content of concrete, the corrosion current density is approximately inversely proportional to the electrolyte resistance and the resistivity respectively or directly proportional to the conductivity according to the following equation:

$$I_{corr} \approx \frac{1}{R_E} \sim \frac{1}{\rho_C} = \sigma_C.$$

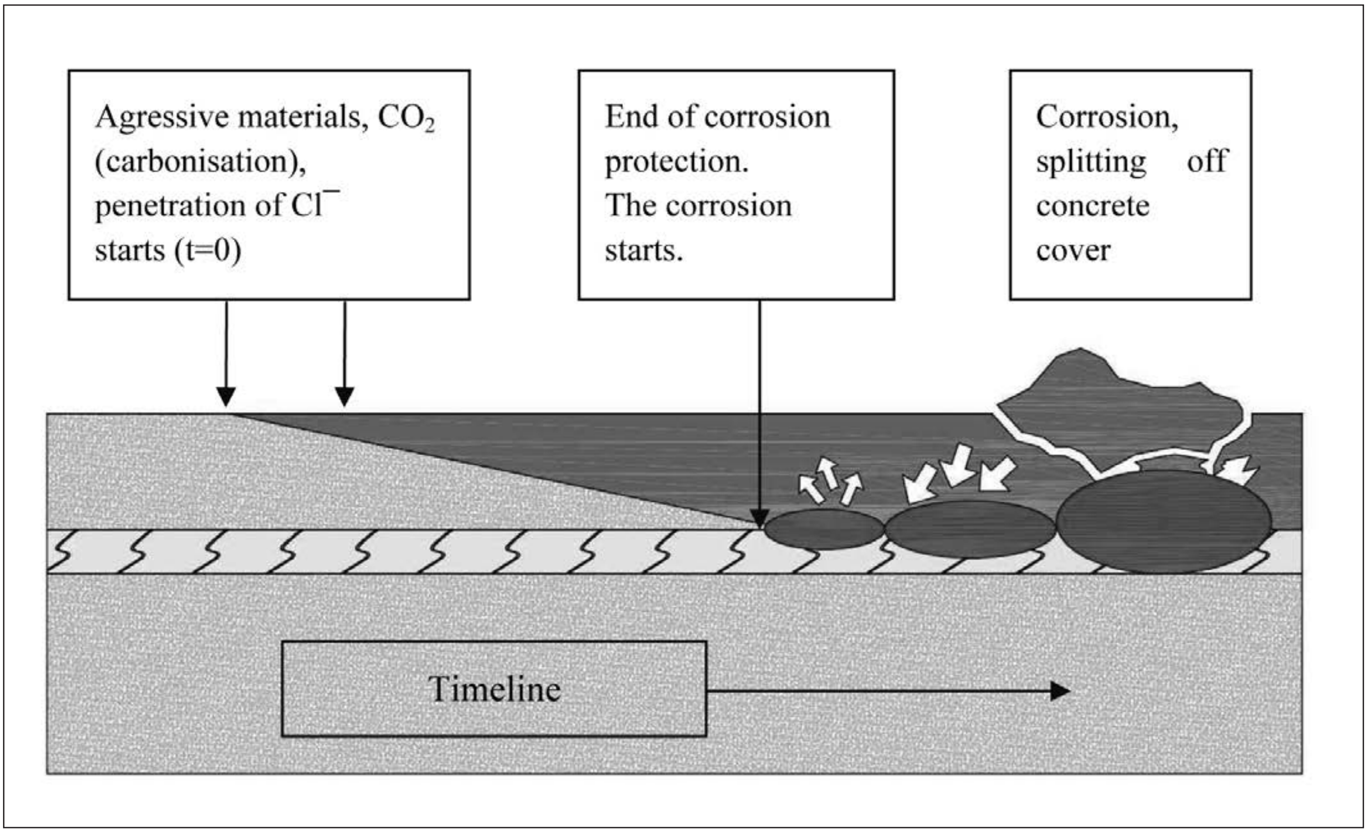
where: I_{corr} corrosion current density,
R_E electrolyte resistivity,
ρ_c resistivity,
σ_c conductivity.

The results of different studies are shown in *Fig. 6*, they are in good agreement with this expression. Although the scatter is large, the same tendency can be seen: with increasing conductivity the rate of corrosion increases. This information has a great importance in practice, when we have to determine the corrosion risk as well as to be able to make judgement about the efficiency of repair methods. The importance of the determination of electrical resistivity is accepted, but still regular, long-term investigations are very rare.

5. EVALUATION OF MEASUREMENTS

The resistivity meter RESI was used to measure the resistivity of reinforced concrete components. This makes it possible to estimate the risk of corrosion of the embedded steel rebars. The corrosion of steel in concrete is an electrochemical process, in the course of that local electrical cells are forming. Their electrical power is very low and they generate very little current, but since working continuously they can cause quite a big harm (Balázs, 2002). The lower the electrical resistance of concrete, the greater is the probability of corrosion. The metal loss as a function of time, i.e. the rate of corrosion also increases. In the manual of the RESI resistivity meter the following limits of possible corrosion risks can be found as a

Fig. 4: The corrosion process



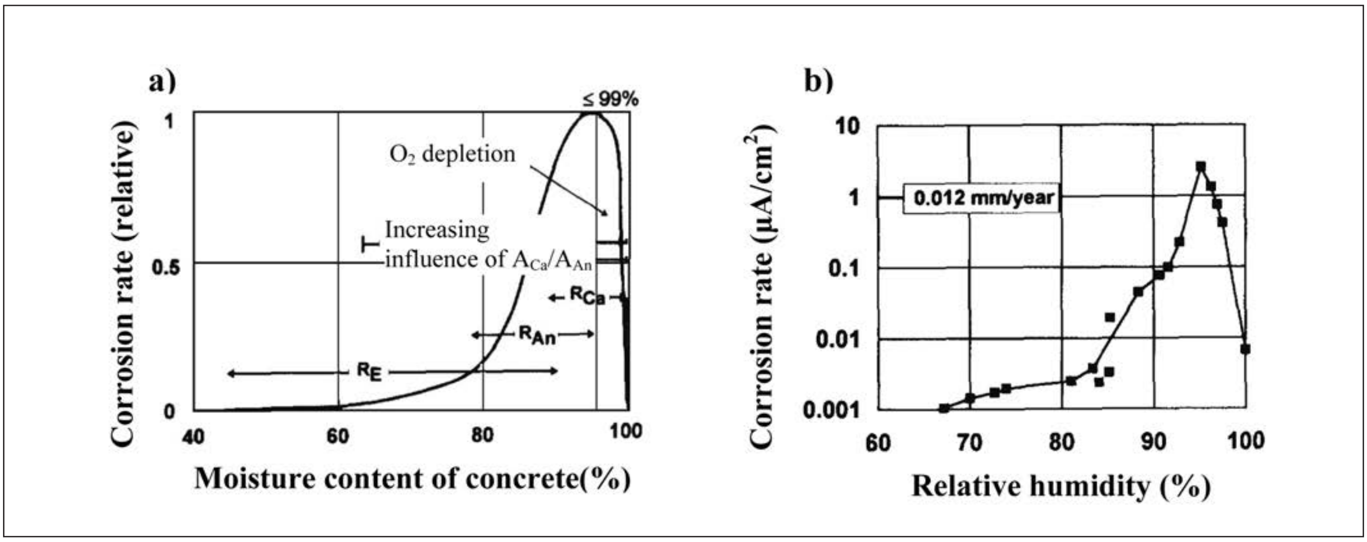


Fig. 5: a) Dependence of the corrosion rate on the water saturation and the influence of the different resistances (RAn: electrochemical resistance of the anode, RCa: electrochemical resistance of the cathode, RE: electrolyte resistance of the concrete/mortar, AAn: area of the anode, ACa: area of the cathode)
b) Corrosion of steel in carbonated mortar (Parrott, 1990)

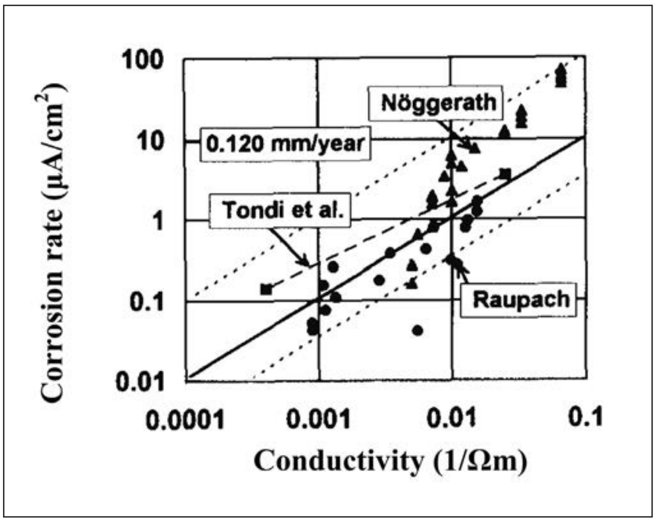


Fig 6: Corrosion rate as a function of the conductivity of concrete or mortar (Hunkeler, 1993)

function of the specific resistance (ρ_c):

$\rho_c \geq 12$ kΩcm	no danger of corrosion,
ρ_c = 8-12 kΩcm	danger of corrosion,
$\rho_c \leq$ 8 kΩcm	high danger of corrosion.

According to the experimental results of Dr. J. P. Broomfield however:

ρ_c > 20 kΩcm	low risk of corrosion rate,
ρ_c = 10-20 kΩcm	low to moderate corrosion risk rate,
ρ_c = 5-10 kΩcm	high corrosion risk rate,
ρ_c < 5 kΩcm	very high corrosion risk rate.

It can be predicted in a logical way too, that the electrical resistivity of concrete strongly depends on the moisture content, and the results are also strongly influenced by the environmental effects. Therefore it is not advisable to accept uncritically on their own the above mentioned limiting values. It is essential to mention, that the acceptability of the measured results is valid only in case of water saturated concrete (100 % moisture content). This circumstance was given during our tests on SENTAB pressure pipes, which were under operation, since the concrete was subjected to water pressure from the inside of the concrete tubes.

6. CONCLUSIONS

The damages of reinforced concrete structures due to corrosion are considerable worldwide and influence their endurance and durability.

The process is directed by several parameters, but mainly is a function of the porosity and pore structure of concrete. The pores of concrete could contain water with diluted salts in it, therefore concrete becomes electrically conductive. The resistivity of concrete is measured with a locally very rarely used non-destructive method, which makes it possible to investigate the corrosion risk of embedded rebars of different reinforced concrete structures or elements under operation.

By the evaluation of the measured results we should however pay special attention to:

- the moisture content of concrete,
- the environmental circumstances,
- the conductivity of the saturating solution,
- the presence of chloride or other corrosive materials.

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CONSOLIDATING AND STRENGHTENING EFFECT OF BIOMINERALIZATION ON POROUS MATERIALS

Péter Juhász - Katalin Kopecskó

Porous materials are widely used for construction purposes. While they are often exposed to environmental impacts, their deterioration and decay is unavoidable in a longer period of time. Therefore protection of porous materials, such as concrete, brick and stone, is necessary. Bacteria induced calcium carbonate precipitation nowadays is a widely examined process, being a possible alternative for traditional conservation methods. Biomineralization could not only be used for surface treatments, but also for in-depth post-consolidation of porous materials. There are also investigations on the development of compound materials where bacteria formed crystals act as a binder agent. While biomineralization has already been used for constructional purposes, application connected in situ experiments are recommended by researchers engaged in this field. This paper gives a summary about the possible applications of bacteria induced precipitation on porous materials, and shows the result of an experiment carried out on Hungarian porous limestone specimens. In our experiment, method and curing material developed by a French research group were applied in situ. Cured specimens gave better results for all the examined properties, comparing to the non-cured ones. While effective penetration depth of the curing compound plays an important role in conservation, examination of the penetration depth was also subject of our experiment.

Keywords: biomineralization, bioconsolidation, strengthening, porous materials

1. INTRODUCTION

Many of important and widely used construction materials are porous in their structure. Even if concrete, mortar, stone and brick surfaces are usually covered in the buildings, they are often exposed to environmental impacts, for example on façades. Threats like salt crystallization, freez-thaw, ion-exchange reactions due to carbon-dioxide, or acids in rainwater decrease the initial strength of the materials. This leads to deterioration of the material. After some decades, these sorts of porous materials need supplementary protection of surface, consolidation or - in the worst case - complete replacement of the material. The first two methods consist of applying different chemical compounds onto the exposed surfaces, depending on the characteristics of material (such as components, porosity, bounding material), level and type of deterioration. The desired results of these treatments are partial blocking of the pores, and rebinding detached sediments. Hence, lower water-absorption and delayed deterioration of the material is expected.

Compounds used for surface treatments and reconsolidation are usually of inorganic origin, therefore they can be harmful to their environment. Moreover, inorganic compounds used for the consolidation of stone materials are often not compatible with the characteristics of the stone.

For more than twenty years a natural phenomenon, called biomineralization has been developed for utilization in consolidating treatments. Biomineralization is a phenomenon, that different bacteria strains are capable of producing calcium-carbonate crystals in adequate environment. In connection with construction materials, bacterially induced carbonate precipitation was first developed for protection and consolidation of ornamental stones.

Nowadays there are two main trends in biomineralization: biodeposition and biocementation (De Muynck et al. 2010). Biodeposition is an organic originated and highly compatible method for restoration and conservation of porous stone materials, especially porous limestone or lime-bound sandstone. It results in a deposition of a carbonate layer on the surface, and in a depth of a few centimetres under the surface of porous materials. Crystals produced during the precipitation integrate themselves into the matrix of the stone material in a high extent. This method was first used for conservation purposes by a French research group (Le Metayer-Levrel et al. 1999).

The other form of application of microbially induced carbonate precipitation (MICP) is biocementation, used for the generation of binder-based materials such as mortars and concrete. Investigations related to biocementation have more sub-branches than of biodeposition. The four main fields are: development of biological mortar, remediation of cracks in concrete, bacterial concrete and self-healing concrete (De Muynck et al., 2010).

Biological mortar refers to a mixture of bacteria, finely ground limestone and nutritional medium, which contains calcium salt. In this mortar the binder has microbial origin, in form of carbonate crystals, which cements together the aggregates. The technique has already been successfully tested on a small scale on sculptures of the Amiens Cathedral and on a portal of the church of Argenton-Château (France). Visual observations two years after the treatment indicated a satisfactory appearance of the repaired zones (Le Metayer-Levrel et al., 1999; Orial et al., 2002).

The objective of microbially induced crack remediation is sealing the cracks and fissures, thus avoiding the ageing process of concrete structures upon exposure to weather changes. Specimens with cracks filled with bacteria, nutrients and sand, demonstrated a significant increase in compressive strength and stiffness values when compared with those without cells. Recent investigations focus on the improvement of strength as a result of the cementation of powdering particles (Ramachandran et al., 2001). Besides external application of bacteria in case of remediation of cracks, microorganisms have also been applied in the concrete mixture. With the aim of increasing the durability of the concrete, bacteria and nutrition were added directly to the concrete mixture, producing bacterial concrete. The investigations revealed that calcium carbonate precipitation could not occur due to the inhibition of bacteria by alkaline pH and the lack of oxygen inside the mixture. The first experiments did not lead to the strengthening of the mortar specimens, thus further research is running in this field (Ramachandran et al., 2001). Investigations on the self-healing concrete based on the idea of activating the precipitation process, when cracks or fissures occur in the material. Water penetrating through the openings could activate bacteria spores, which would produce calcium-carbonate crystals. Similar to the bacterial concrete, bacteria and different organic compounds were added to the concrete mixture (Jonkers, 2007). In this field of application, investigations – for similar reasons to bacterial concrete – have not yet confirmed the competences of bacteria precipitated calcium-carbonate as effective bounding agent.

From the above mentioned applications of biomineralization, only biodeposition and crack remediation have shown potential to strengthen and consolidate porous materials. In our investigation consolidating and strengthening effect of the biodeposition method was tested on limestone specimens. While changes in strength correlate with the changes of mass-properties and water-absorption, results of our experiment were evaluated by comparing the parameters affecting each other.

2. MATERIALS AND PREPARATION OF THE SPECIMENS

2.1 Stone material

The stone material used in our experiment is porous soft limestone of Sóskút (Hungary) of two different types. The first one has ooidic texture with finer grains (a), the second one has an ooidic-bioclastic texture with coarse grains, bioclasts and larger pores (b). Two blocks of the finer, as well as five blocks of the coarse limestone, altogether seven blocks of limestone (marked with capital letters A-G) were drilled and cut in order to prepare the specimens.

2.2 Bacterial isolates and culture media

In our experiment curing materials developed by the French company Calcite Bioconcept were applied on the porous limestone specimens. With the desired aim of obtaining homogenously, in-debt cured specimens, the samples were immersed into the liquid compound instead of spraying the curing solution onto their surface. The bacterium strain applied in the compound is *Bacillus cereus*, which is a large, 1 x 3-4 µm, Gram-positive, rod-shaped, endospore forming, facultative aerobic, non-pathogen soil-bacterium. *B. cereus* is mesophilic, growing optimally at temperatures between 20°C and 40°C, and is capable of adapting to a wide range of environmental conditions (Vilain et al., 2006). Thus it is ideal for outdoor treatments in calm weather. According to its spore-forming ability, this bacterium is capable for survival and revitalization depending on the accessibility to nutrition.

2.3 Preparation and grouping of the specimens

The porous limestone specimens were prepared according to the requirements of the standard MSZ EN 1926 (April, 2007). Thus diameters and heights of the cylindrical specimens are 50±5 mm. The blocks were classified according to their textures (two types) and their bulk density values (measured on the specimens). This grouping resulted in the establishment of four groups: I. - blocks A and D; II. - blocks B, C and E; III. - block F; IV. - block G (*Tab. 1*). In each group half of the specimens were cured, and the other half was used as controls. In the uniaxial compressive strength test specimens from five blocks were involved (B and F were not presented herein), and specimens from block F were entirely left out of the evaluation.

Table 1: Groups of the specimens

Mark of block	Texture	Number of specimens		Average bulk density [g/cm^3]	Group	Comment
		cured	control			
A	ooidic (a)	5	5	1.6376	I.	-
B	o-bioclastic (b)	10	0	1.6472	II.	-
C	o-bioclastic (b)	5	5	1.6954	II.	-
D	ooidic (a)	5	5	1.6280	I.	-
E	o-bioclastic (b)	3	3	1.6640	II.	-
F	o-bioclastic (b)	1	9	**1.5250**	-	minimum
G	o-bioclastic (b)	5	5	**1.7389**	III.	maximum
∑	-	**34**	**32**			
Average bulk density of blocks (g/cm^3):				1.6480		
Standard deviation of density (g/cm^3):				0.0663		
Lower border:				1.5818		
Upper border:				1.7143		

2.4 Environmental circumstances

Upon previous recommendations of the researchers engaged in biomineralization (De Muynck et al., 2010), our experiment was carried out in situ. With some differences, we applied a one-week-long treatment on the limestone specimens following the instructions of the French Calcite Bioconcept company. Different from the original method, the samples

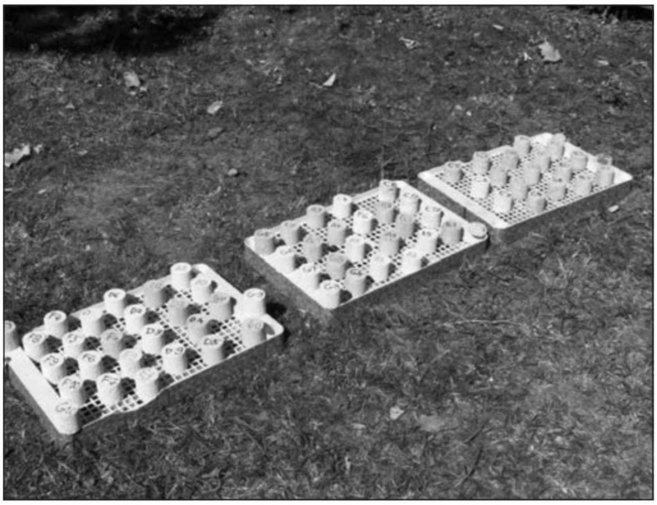


Fig. 1: Display of the specimens

were immersed into the liquid compound, instead of spraying the curing solution onto their surface. The treatment was carried out between the 23th and the 29th of August, 2011. Specimens were exposed to intensive sunlight during half of the day (*Fig. 1*). Daily temperature fluctuation is shown in *Tab. 2*. Surface temperatures of the specimens were also measured. Temperature of sunny sides of the dry (before treatment) specimens vary between 36.6-39.7°C (in the morning at 11:04), and in wet (treated) condition between 25.0-42.5°C (at 11:50) and 22.9-26.8°C (at 19:30). Thus bacterial activity was not inhibited by the temperature.

3. EXPERIMENTS

3.1 Uniaxial compressive strength

Generally a material with high compressive strength is more durable, than other ones with lower values. Consolidating effect of biomineralization has yet been examined only by indirect methods, giving relatively comparable results for the cured and non-cured surfaces and specimens. One of these measurements is the peeling tape-method, where adhesive tapes remove loose or poorly cemented surface carbonate grains (Jroundi et al., 2010). Other experiments on durability are related to the weight-loss due to freezing-thawing, salt crystallization (De Muynck et al., 2008) or sonication (Rodriguez-Navarro et al. 2003). In order to give objective and easily comparable information on the durability of our stone material, we measured the uniaxial compressive strength of the cured and non-cured specimens. Measurements and evaluation were done according to the regulations of the standard MSZ EN 1926. Individual results for compressive strength of the specimens made from the tested limestone were expected to be in the range of 5-50 N/mm^2 (Kleb, 1975).

3.2 Water absorption and saturated water-content

Water absorption in time and saturated water-content highly affect the strength of porous stone materials by decreasing it in a great extent. Sedimentary stones containing expanding particles such as clay minerals are even more endangered by this impact. Water absorption was tested in time by measuring wet masses of the specimens placed on a grill and immersed into 5 mm deep water in time. Water supplement was continuous. Absorption was evaluated by calculating the water-content at every step. Saturated water-content was measured by comparing the dry and the saturated masses of the specimens in atmospheric conditions. Saturation was reached by immersion of the specimens before and after the treatment until reaching constant weight, respectively. Due to the treatment we expected decrease of water absorption and maximum water-content.

3.3 Mass properties

Precipitation during the biodeposition process results in the formation of calcium-carbonate capsules, which are strongly bound to the surface of the pores. Therefore they have a capability to strengthen the inner structure of the material. During the measurement of mass properties we examined the changes of dry weight and apparent porosity. By combining these results, we could examine the correlation between apparent porosity and relative changes of dry weight due to the precipitation. Measurements were carried out before and after the treatment, respectively. Subjecting the specimens to saturation with water and drying until constant weight could not affect the results. After the treatment grains and leftovers inside the specimens were washed out, and the specimens were dried. As result of the biomineralization we expected increase of dry mass and decrease of the apparent porosity.

3.4 Scanning electron microscopy and indirect methods

These measurements were carried out with the aim of estimating the migration depth of the bacteria, thus the expectable effective depth of the treatment. While some of the consolidating compounds are not able to seep deeply into the porous matrix of the material, they result in the formation of a solid layer only on the surface, causing deterioration of the stone. Therefore analysis of the effective depth of a consolidating treatment is highly recommended. We carried out two observations to estimate the migration of the bacteria: scanning electron microscopy (SEM) and an indirect method.

For observations with SEM 3-3 samples were taken out from three depths (surface, radius/2, middle) of one cured and one non-cured specimens (controls). We were looking for patterns of newly precipitated crystals on the samples.

The indirect method is based on the assumption, that bacteria are only transported

Table 2: Air temperature and relative humidity data between the 23th and 29th of August, 2011 (OMSZ, 2012)

Date		23. 8.	24. 8.	25. 8.	26. 8.	27. 8.	28. 8.	29. 8.
Air temperature [°C]	min.	22	22	23	23.5	22.5	16	17
	max.	34	34	36	35	31	24.5	27
	6:00	22	22.5	24	23	22.5	16	17
	14:00	33	33	34	33	20	23	26
	22:00	27	27.5	28	28	22	21	23
Relative humidity [%]	min.	30	32	28	32	34	46	37
	max.	83	75	75	73	60	73	74
	6:00	80	72	70	70	58	72	70
	14:00	33	36	35	42	38	47	42
	22:00	50	55	44	46	47	52	44

by the migration of the curing liquid inside the porous matrix. Thus, by splitting the specimens in two pieces, and printing the surfaces onto agar plates, bacterial colonies growing on the imprint will show the migration depth of the bacteria. Curing of the specimens consisted of inoculating two pieces of stone with bacteria solution by immersion for 60 seconds. After 24 hours of incubation at 30^{o}C, specimens were split, and the inner surfaces were printed on *B. cereus* – selective substrate, consolidated with agar (MERCK Cereus-Selective Mannitol-Egg-yolk-Polymyxine-Agar). Distribution of the germination inside the contour of the prints would indicate penetration depth of the bacteria.

By immersing the specimens into the curing liquid, instead of spraying it onto the surface, we expected increase of the migration depth, thus total saturation, which would result in evenly cured specimens.

4. RESULTS AND DISCUSSION

4.1 Uniaxial compressive strength

As a result of the treatment, the average values of compressive strengths increased for each groups in our experiment (*Tab. 3*). However, standard deviations of the values are higher, than the changes, which is likely to occur during measurements on heterogeneous porous stone materials (Jroundi et al., 2010). Therefore tests designed to evaluate the efficacy of conservation treatments applied on heterogeneous stones, have to be considered with caution. In order to have a better understanding on the durability of such materials, further measurements on more homogenous materials with similar apparent porosity and mechanical characteristics are necessary.

4.2 Water absorption and saturated water content

Both water absorption and maximum water content were decreased by the treatment. Speed of saturation and water content in time (*Fig. 2*) were also decreased. Water content was calculated with the following equation:
$Wc = ((m_{wet} - m_{dry})/m_{dry})*100$ [m/m %]. As result of the treatment, maximum water content of the cured specimens was also decreased with an average of 8.5 %, compared to the non-cured ones. (*Fig. 3*). These results indicate, that subjecting porous stone materials to biomineralization provides a better resistance against water-related problems, thus decrease of strength, salt-crystallization and freezing-thawing.

Table 3: Changes of compressive strengths

Mark of block or group	Compressive strength [N/mm^2]		Relative difference [%]	Standard deviation [%]	
	Non-cured	Cured		Non-cured	Cured
A	6.42	7.99	+24.59	7.44	30.75
C	3.84	4.54	+18.38	24.97	30.68
D	6.35	6.53	+2.78	22.50	10.50
E	3.40	3.55	+4.40	14.34	18.85
G	4.21	4.54	+7.99	18.44	26.74
Mean	**4.84**	**5.43**	**+11.63**	**17.54**	**23.51**
I. (A and D)	6.39	7.26	+13.74	14.97	20.63
II.b(C and E-B)	3.62	4.05	+11.81	19.66	24.77
IV. (G)	4.21	4.54	+7.99	18.44	26.74

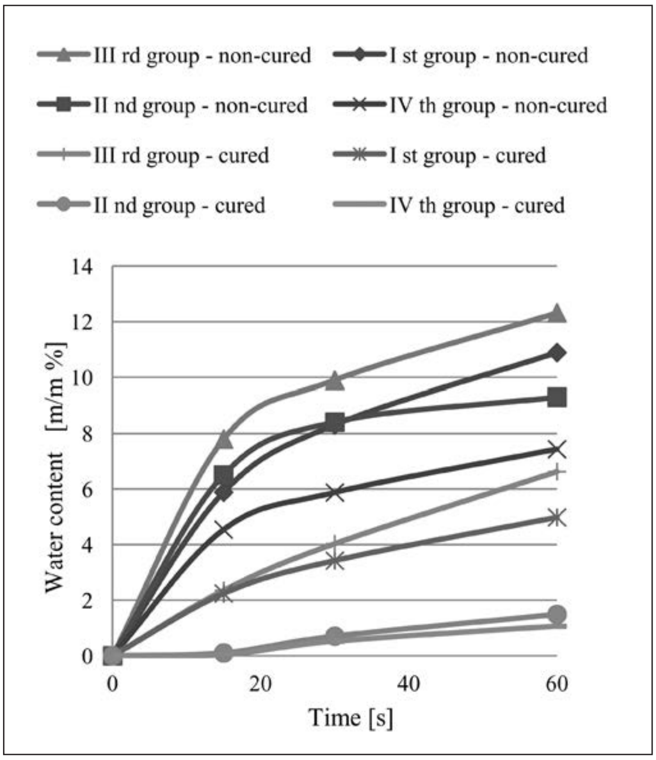


Fig. 2: Water-content of non-cured, and cured specimens in time

4.3 Changes of the mass properties

After the treatment, leftovers were washed out from the specimens by three days of immersion in water. Then specimens were dried till constant weight. Dry masses of the cured specimens increased, while control ones did not change. This is an evidence for the successful calcium-carbonate precipitation, which also indicates, that the newly formed crystals bound strongly to the inner pore surfaces. With a relative dry-mass difference of 0.26 and 0.28 m/m %, block A and D showed the highest rate of change (*Tab. 4*). This result can be contributed to their texture, while both of them have ooidic texture with finer grains. Furthermore, their initial apparent porosity values were the highest (paralelly their bulk density values the lowest), thus the efficiency of the biodeposition treatment was shown to be higher on more porous material. Increase of the dry masses also resulted in the decrease of apparent porosity (*Tab. 5*). Average decrease of the apparent porosity was 8.27 %. Hence water-absorption was decreased, too.

4.4 Scanning electron microscopy (SEM) and indirect test method

Fig. 4 shows bacterial bodies encapsulated in a layer of calcium-carbonate, as result of induced biodeposition with the method of Calcite Bioconcept. Calcium-carbonate precipitation occurs in membrane-level, thus bacterial bodies play active role in decreasing pore-size. Our SEM-observations did not reveal formations shown on *Fig. 4*. Only some sort of precipitation shown in *Fig. 5.a* and *Fig. 5.b* was found on the surface, but those are not necessarily result of the bacterial activity. Absence of the capsules could be the

Table 4: *Average dry masses of the groups*

Mark of block or group	Means of dry masses [g]		
	Non-cured	Cured	Relative difference [%]
A	194.90	195.41	**+0.26**
B	197.91	198.34	**+0.22**
C	205.24	205.71	**+0.23**
D	196.44	196.99	**+0.28**
E	200.43	200.87	**+0.22**
G	210.37	210.78	**+0.20**
I.	195.67	196.20	**+0.27**
II.	201.19	201.64	**+0.22**
III.	188.28	188.55	**+0.14**
IV.	210.37	210.78	**+0.20**

result of the preparation of samples for electron microscopy observation. Coating of the samples with high pressure might have broken the capsules. Another possible explanation for the absence of the capsules is that the precipitation might have occurred in a different way, and resulted in the formation of diffuse groups of crystals instead of regular capsules. Furthermore, identification of newly formed calcium-carbonate crystals on a stone-surface or in the pores of an identical material is really difficult.

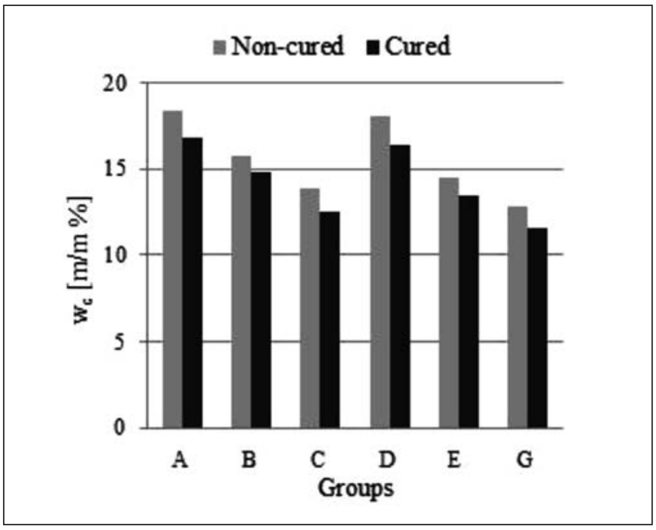
In this situation we could not use scanning electron microscopy for analyzing the migration depth. For the better evaluation of migration depth and morphology of the precipitated calcium-carbonate, it is necessary to study the effect of biodeposition on materials, which are not calcium-carbonate based. Observation of broken surfaces of biomineralized mortar and concrete specimens, as well as observation of crystal morphology on glass plates, could provide more accurate results.

Indirect test aimed to determine the effective penetration depth of the curing compound. Results of this test indicated even distribution of bacteria inside the specimens. Surfaces of the sections printed onto the agar plates shown similarly even distribution of growing colonies, as imprints of the top surfaces, which were directly exposed to the curing compound. The only difference is the density of the colonies, which can be contributed to the hampered migration of the compound inside the specimen, hence smaller amount of absorbed bacteria in volume units.

Table 5: *Average apparent porosities of the blocks and the groups*

Mark of blocks and groups	Apparent porosity		
	Non-cured	Cured	Relative difference [%]
A	0.301	0.276	-8.37
B	0.260	0.245	-5.72
C	0.235	0.212	-10.04
D	0.294	0.267	-9.15
E	0.241	0.224	-7.00
G	0.223	0.202	-9.36
I.	0.297	0.271	-8.76
II.	0.245	0.227	-7.59
III.	0.223	0.202	-9.36

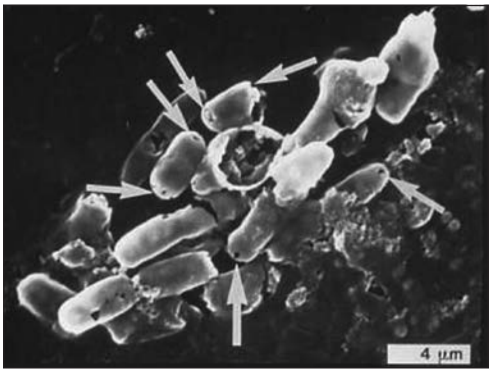
Fig. 3: *Changes of the maximum water-content of the different blocks*



5. CONCLUSION

In our experiment biodeposition treatment was applied on Hungarian porous Sóskút limestone, in situ. Materials and method developed by the French company Calcite Bioconcept were used. Our measurements confirmed that the precipitation occurred, and resulted in favorable changes of the measured properties. Compressive strength increased with an average of 11.43%, however, standard deviations were higher than the rate of changes. As result of the treatment, saturated water content of the cured specimens at atmospheric conditions decreased with an average of 8.5 %, and also the speed of absorption decreased comparing with the non-cured ones. Dry masses of the specimens increased, thus apparent porosity was reduced. Besides, efficiency of the biodeposition treatment turned out to be higher on more porous material. For determination of the migration depth scanning electron microscopy was not appropriate, however the indirect method indicated, that biodeposition occurred inside the specimens, too.

Fig. 4: Bacteria encapsulated in $CaCO_3$ at membrane-level (Castanier et al. 1999)



The results of our experiment suggested, that biodeposition treatment is appropriate for consolidation of porous materials, too. For better understanding of the results, further experiments of bioconsolidation and comparison of the method with other bioconsolidating compounds and techniques is necessary.

6. ACKNOWLEDGEMENT

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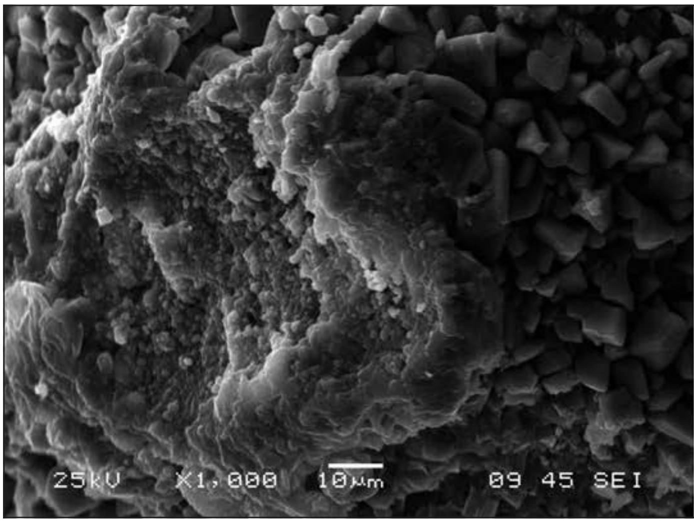

Fig. 5.a: Surface of the sample: original surface on the right, precipitation on the left at 1000 X magnification.

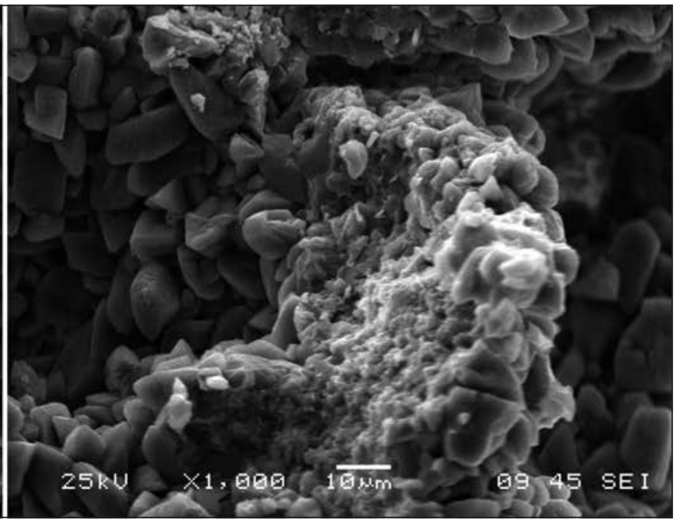

Fig. 5.b: Surface of the sample with small, spot-like groups of crystals at 1000X magnification

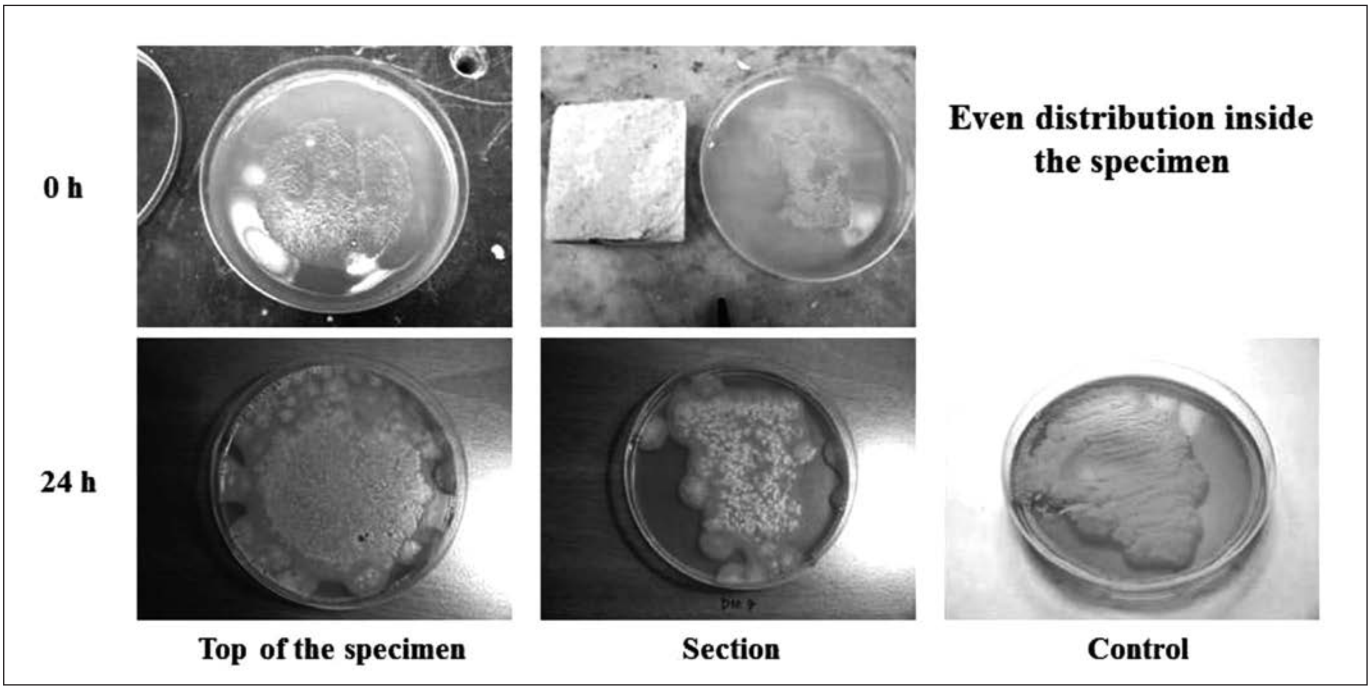

Fig. 6: Indirect evaluation of the effective penetration depth. Prints at 0 h and after 24 hours of incubation, and the control.

material, the method and for consultation. We would also thank Willem de Muynck (Ghent University) and Dr. Ágnes Suhajda (BUTE) for their help and advices before and during the experiment, and Dr. Béla Koczka for his help with the scanning electron microscopy.

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REINFORCED CONCRETE STRUCTURES IN AND AFTER FIRE


György L. Balázs – Éva Lublóy

Construction materials suffer in fire. Deterioration of material characteristics and structural performance highly depend on constituents and on the temperature history. Design for high temperatures requires additional aspects of material composition and material characteristics compared to design for ULS (ultimate limit states) and for SLS (serviceability limit states).

A detailed experimental analysis is given to modification of various characteristics such as surface cracking (spalling), strength (compressive strength, flexural strength) and bond between concrete and reinforcement. Present experimental study included variable tests: cements with different slag contents as well as different aggregates (quartz gravel and expanded clay), different fibres (polypropylene and steel both with various geometries).

Present test results provide information on possible optimization of concrete composition for high temperatures including selection of appropriate cement, aggregate and fibres.

keywords: fire, fire test, spalling, fibres, load, residual strength, concrete composition

1. INTRODUCTION

Concrete has excellent properties in regards of fire resistance compared with other materials and can be used to shield other structural materials such as steel (Khoury, Grainger, Sulivan, 1985).

Effects of high temperatures on the mechanical properties of concrete have been investigated as early as the 1920s (Schneider, Lebeda, 2000). In the 1960s and 1970s fire research was mainly directed to study the behaviour of concrete structural elements (Kordina, 1997). There was relatively little information on the concrete properties during and after fire (Waubke, 1973).

During fire the mechanical characteristics of the concrete are changing. During the cooling process concrete is not able to recover its original characteristics. Deterioration of concrete at high temperatures has two forms: local damage in the material itself (*Fig. 1*) and global damage resulting the failure of the elements (*Fig. 2*). Recent results by Horiguchi (2011) indicated that in some situations a part of the strength can be recovered.

As polypropylene fibers melted at its fusion point of 160-170 °C, polypropylene fibre reinforced concrete (PFRC) showed more reduction in its residual properties compared to steel fiber reinforced concrete (SFRC). More inclusion of

Fig. 1: Damage of concrete

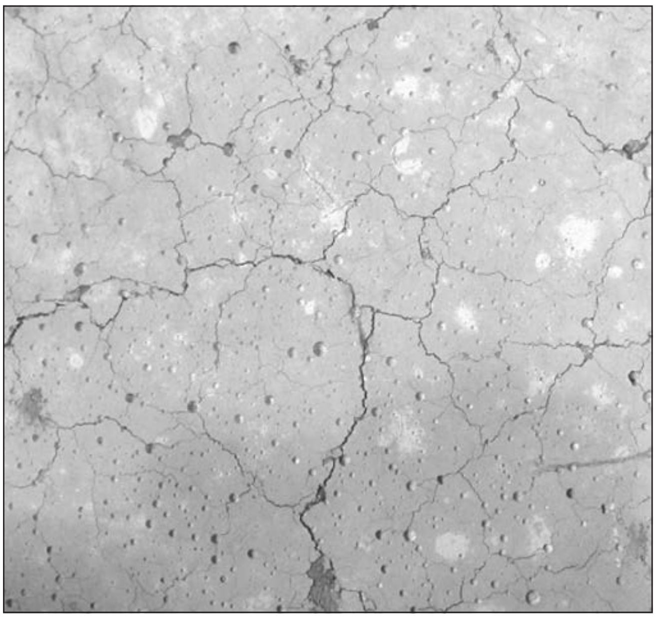
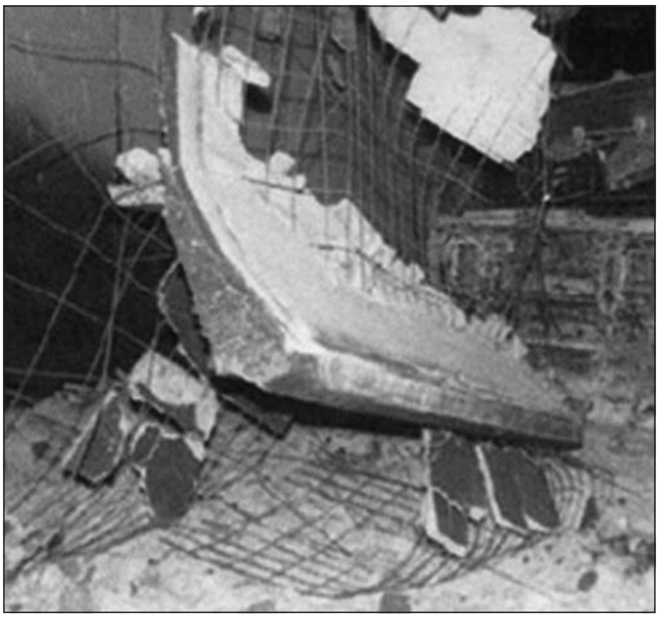


Fig. 2: Damage of structure (http://www.polizia.ti.ch)



polypropylene fibers will tend to reduce most of PFRC and HFRC (hybrid fibre reinforced concrete) residual properties, especially its residual permeability performance. On the other hand, more inclusion of steel fibers may improve the splitting tensile strength of SFRC and HFRC.

In residual permeability performance, more inclusion of steel fibers was found to be quite effective in the series also consisting 0.25% of polypropylene fibers in the HFRC, at both 200 °C and 400 °C. For PFRC, fiber length significantly affected the residual water permeability coefficient. The longer the fiber, the higher the residual water permeability coefficient.

Significant recovery of properties was observed on heated concrete specimens cured under saturated condition compared to the ones cured under ambient temperature (Horiguchi, 2011). The recovery rate showed a rapid increase in the first two months on heated concrete being cured under saturated condition and slowed down after that

2. BEHAVIOUR OF CONSTRUCTION MATERIALS

2.1 Concrete

Concrete is a composite material, that consists mainly of mineral aggregates bound by a matrix of hydrated cement paste. The matrix is highly porous and contains a relatively large amount of free water unless artificially dried.

When exposed it to high temperatures, concrete undergoes changes in its chemical composition, physical structure and water content. These changes occur primarily in the hardened cement paste in unsealed conditions. Such changes are reflected by changes in the physical and mechanical properties of concrete that are associated with temperature increase.

Chemical changes can be studied with thermogravimetrical analyses (TG/DTG/DTA). The following chemical transformations can be observed by increase of temperature: Around 100 °C the weight loss is caused by water evaporating from the micropores. The decomposition of ettringite ($3CaOAl_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$) occurs between 50 °C and 110 °C. At 200°C there is further dehydration which causes small weight loss. The weight loss with various moisture contents was different till all the pore water and chemically bound water were gone. Further weight loss was not perceptible around 250-300 °C (Khoury, Grainger, Sullivan, 1985, Schneider, Weiss, 1997).

During heating the endothermic dehydration of $Ca(OH)_2$ occurs between the temperatures of 450 °C and 550 °C ($Ca(OH)_2 \rightarrow CaO + H_2O\uparrow$) (Schneider, Weiss, 1977). In case of concretes with quartz gravel aggregate an other influencing factor is the change of crystal structure of quartz α formation→ β formation at the temperature of 573 °C (Waubke, 1997). This transformation is followed by 5.7% volumetric increase.

Dehydration of calcium-silicate-hydrates were found at the temperature of 700 °C (Hinrichsmeyer, 19897).

Modification of stress-strain diagram of concrete for increasing temperatures is shown in *Fig. 3.* One can observe how the

- strength decreases and the corresponding strain increases as well as the
- initial modulus of elasticity decreases.

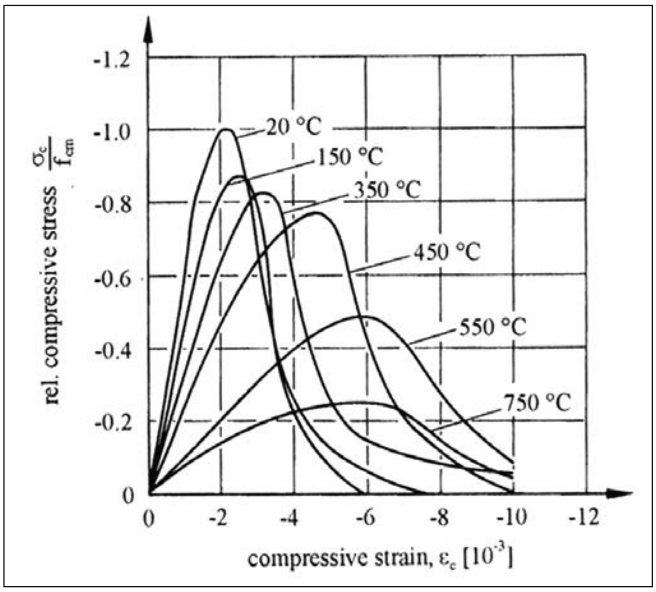


Fig. 3: Stress-strain relationship for concrete with quartz gravel aggregate as a function of temperature (Schneider, Lebeda, 2000)

An experimental study was carried out on concrete specimens with the following parameters:

- temperature (maximal temperatures were: 20 °C, 50 °C, 150 °C, 200 °C, 300 °C, 400 °C, 500 °C, 600 °C or 800 °C)
- cement types (Portland cement or slag Portland cement)
- aggregate (quartz gravel, expanded clay (types 1 and 2), expanded glass, *Table 1*)
- fibres (plastic, steel, *Table 2*).

Table 1: Characteristics of the applied aggregate types

Type of aggregate	particle density (kg/m^3)	bulk density (kg/m^3)	material density (kg/m^3)
expanded clay 1	1047	597	2427
expanded clay 2	1053	771	2469

Table 2: Characteristics of the applied fibre types

Fibre	macro fibre*	mono fibre**	steel fibre
Material	polypropylene	polypropylene	steel
Length (mm)	40	18	40
Diameter (mm)	1.1	0.032	1.1
Melting point (° C)	171	160	
Decomposition temperature (° C)	360	365	

* POLITON V40, Kaposplast Ltd.

** FIBRIN 1832, Kaposplast Ltd.

For all mixtures the water to cement ratio (w/c=0.43) and water to aggregate ratio were constant.

In our study various concrete mixes were tested after temperature loading as a function of the cement type, aggregate type and fibre type.

During our tests, all specimens were kept for two hours at various maximum temperatures. Our heating curve was similar to the standard fire curve up to 800 °C (EN 1991-1-2).

After heating up the specimens to the given temperatures, the specimens were let to cool down and the tests were carried out at room temperature.

2.1.1 Influence of cement type

According to our observations, type of cement has a considerable influence on the residual compressive strength of concrete (on cube specimens) subjected to high temperatures.

Reduction of compressive strength measured after temperature loading (300 °C, 600 °C or 800 °C) decreased with increasing the slag content (*Fig. 4)*. The highest reduction of the compressive strength was recorded by using Portland cement (CEM I 42,5 N with no slag), the lowest reduction was observed using CEM III/B 32,5 N (with slag content of 66 m%). In case of CEM III/B 32 N cement the residual compressive strength was 40% lower than in case of the Portland cement after temperature loading of 800 °C. By increasing the slag content the size and number of surface cracks were decreased (*Fig. 5*).

These differences both in surface cracking and in compressive strengths can be explained by the different structure of the concrete using Portland cement and slag Portland cement. Different structures of concretes using Portland or slag Portland cements are shown in *Figures 6 and 7* obtained by scanning electron microscopy.

2.1.2 Influence of type of aggregate

Whenever the quartz aggregate was substituted by expanded clay aggregate, the compressive strength was significantly improved after heating up to 600 °C and leaving to cool down. When lightweight aggregates were also used in the concrete, the specimens were deteriorated during the heating up to 800 °C. When the specimens deteriorated it has been observed that the aggregate particles were cracked through.

Fig. 4: Compressive strength of hardened cement paste as a function of maximal temperature and cement type

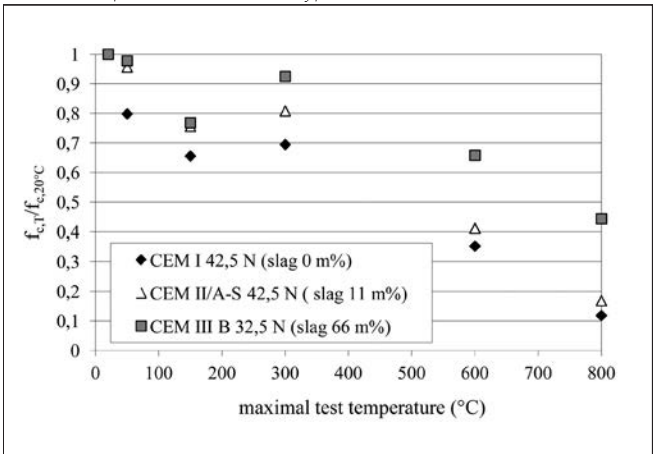
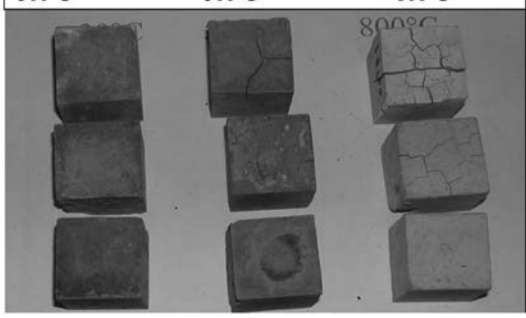


Fig. 5: Development of cracks in hardened cement stone due to high temperatures

Maximum temperature			Cement type
300°C	500°C	800°C	
			CEM I 52,5 N CEM II/A-S 42,5 N CEM III/B 32,5 N-S

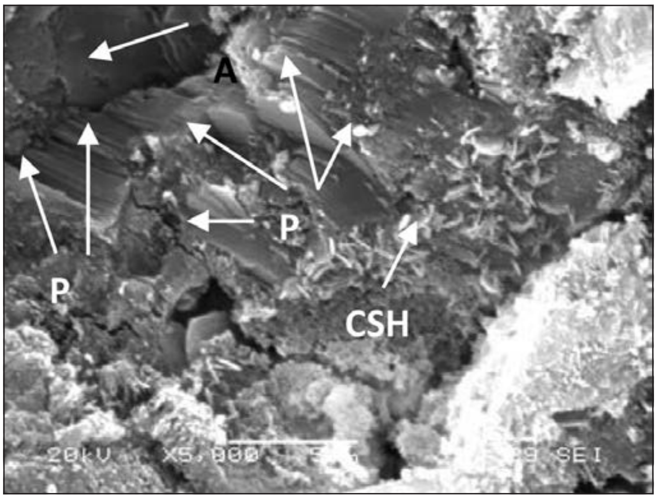


Fig. 6: N 5000 (SEM) Concrete using Portland cement
Notation: A aggregate quartz gravel, P portlandit, CSH calcium-silikat-hidrat

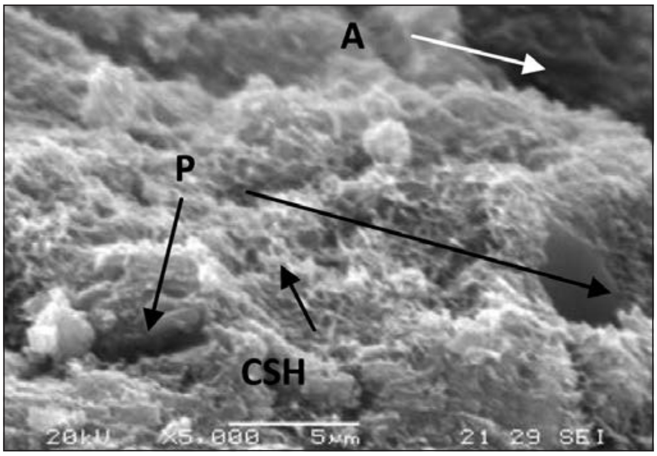


Fig. 7: N 5000 (SEM) Concrete using slag Portland cement
Notation: A aggregate quartz gravel, P portlandit, CSH calcium-silikat-hidrat

We also prepared a concrete mix with expanded clay and 1% by volume PP fibres. In this case the test specimens did not exhibit surface cracks.

Compressive strength measurements with quartz gravel or expanded clay aggregates are presented on *Fig. 8* as a function of the maximal temperature. In order to obtain comparable results the compressive strength of the concrete exposed to high temperature was divided by the strength value measured at room temperature. The following conclusions can be drawn from *Fig. 8*:

– In the case of lightweight aggregate concrete and concrete with quartz gravel significant decrease in the residual compressive strength was observed when the specimens were heated above 500 °C and then cooled down to room temperature. This can be explained by the decomposition of the portlandit around 400 °C and the CSH-s around 700 °C.

– Residual compressive strength of the lightweight aggregate concrete with expanded clay was higher after heating the specimens up to 400 °C and then cooled down to room temperature as in the case of concrete with quartz gravel aggregate. The residual compressive strengths of the quartz gravel aggregate samples were by 20% lower. The difference in the modification of the compressive

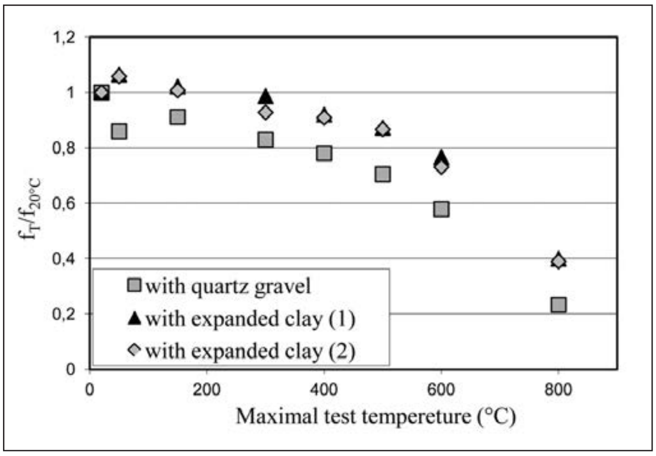


Fig. 8: Change of compressive strength after exposing the concrete to high temperature, measured at room temperature (20°C)

strength can be explained by the different modes of load dispersions and the different contact zones. (Balázs, Lublóy, 2010)

– Application of synthetic fibres, both with quartz gravel and expanded clay aggregates, the residual compressive strength slightly decreased at room temperature and also due to high temperatures.

Fig. 9: Normal weight concrete at temperature of 20°C

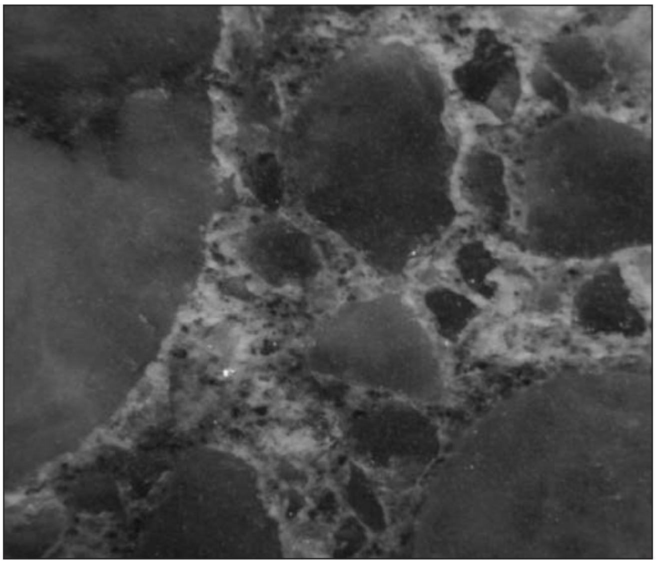
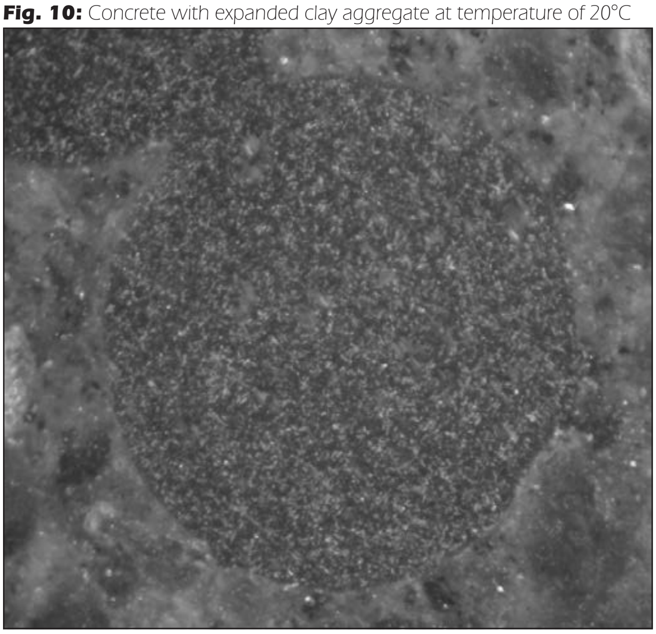


Fig. 10: Concrete with expanded clay aggregate at temperature of 20°C



– In the case of concrete with expanded clay with no polypropylene fibre, the specimens were deteriorated during the heating up to 800 °C in the furnace.

The different behaviours of the various concrete mixes can be explained by the differences in the contact zones of aggregate and cement stone *(Figs. 9 and 10)*. It can be observed in the optical microscope photos that a layer of crystals can be found beside the quartz gravel (see *Fig. 8*). The contact zone of the expanded clay is quite different *(Fig. 10)*. The cement paste penetrates into the external pores of the aggregate of their porous structure.

2.1.3 Influence of types of fibers

Spalling of concrete cover can be decreased by the application of synthetic fibres (Horiguchi, 2004, Horiguchi, 2005). We have tested two types of polypropylen fibers (mono fibres with d=0.032 mm; macro fibres with d=1.1 mm).

In case of concrete with small diameter mono fibres considerable surface deformation was not observed up to 800 °C (*Fig. 11*). In case of the reference concrete without

Fig. 11: Mono fibre reinforced concrete (800°C temperature load)

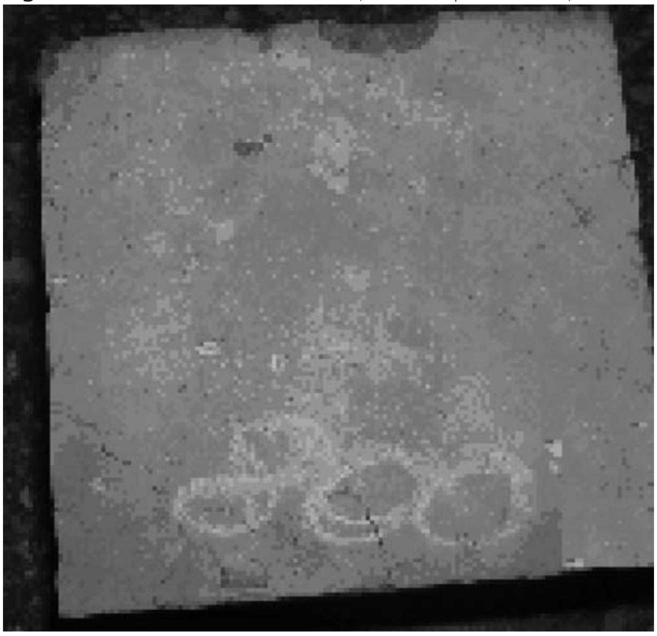
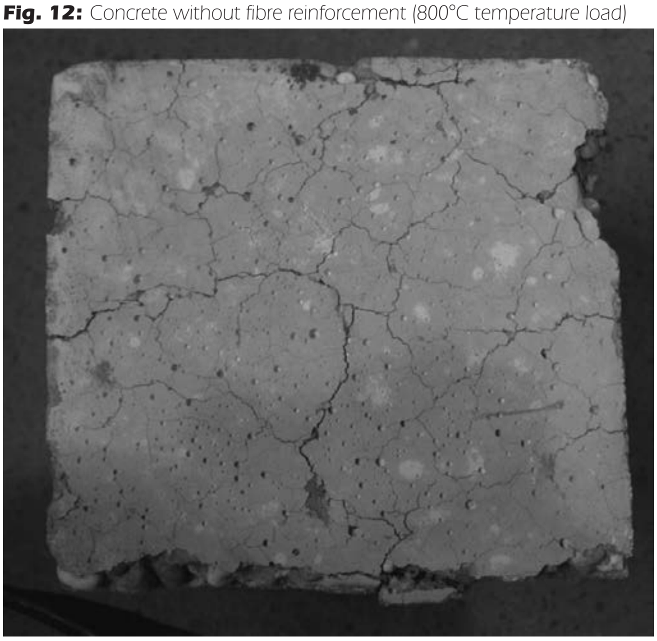


Fig. 12: Concrete without fibre reinforcement (800°C temperature load)



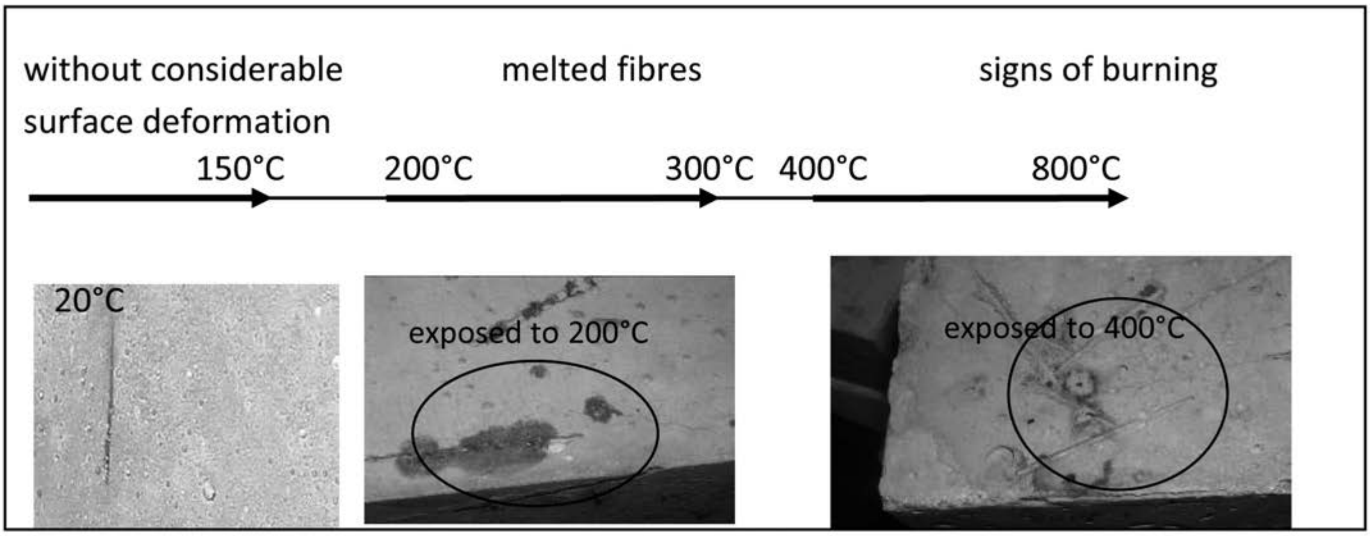


Fig. 13: Concrete specimens with macro fiber reinforced concrete

fibres we have observed surface cracks by heating up to 800 °C (*Fig. 12*).

We observed that macro fibres close to the surface flowed (at 200 °C and at 300 °C) to the surface and (at 400 °C) by leaving colour on the surface (*Fig. 13*). In some places small holes could be also observed. These fibres were probably perpendicular to the concrete surface and off in this position. Signs of burning could be seen on the concrete surface. These could be avoided by using of small mono fibres instead of relatively large macro fibres (*Balázs, Lublóy, 2010*).

2.2 Steel

Modification of stress-strain diagram of reinforcing steel for high temperatures is shown in *Fig. 14* (Reinhardt, 1973). The consequences of temperature increase are:

Fig. 14: Stress-strain relationship for steel as a function of temperature (Reinhardt, 1973)

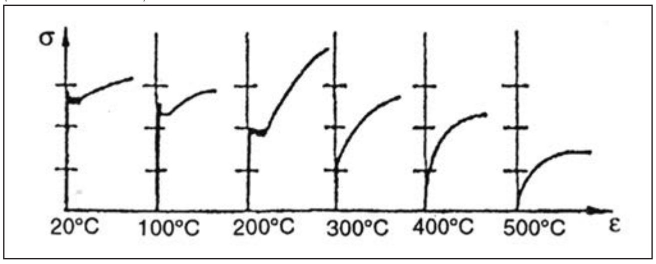
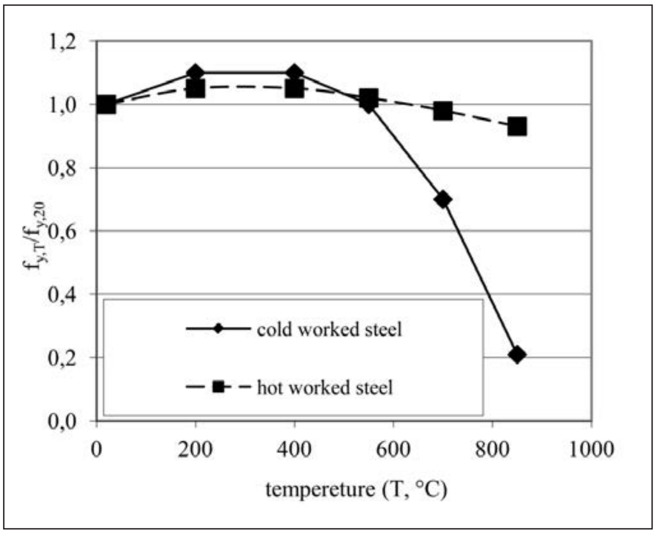


Fig. 15: Tensile strength of steel as a function of temperature (Felicetti, Meda, 2005)



– reduction of yield strength
– reduction (and finally disappearance) of the yield plateau
– reduction of tensile strength

In *Fig. 15* (Felicetti, Meda, 2005) the development of tensile strength of steel as a function of temperature demonstrated.

3. DETERIORATION OF THE STRUCTURAL PERFORMANCE

3.1 Spalling

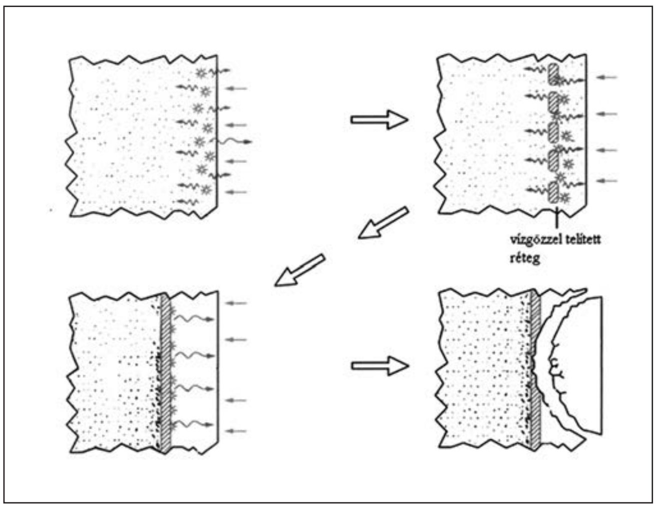
In case of fires, in addition to the reduction of load bearing capacity, spalling of concrete cover causes further difficulties. The probability of spalling of concrete cover increases by increasing the strength of concrete (Janson, Boström, 2004).

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. 16* (Høj, 2005).

Special care is needed to avoid spalling of concrete cover. A group of experiments including normal strength concrete suggested that the application of polymeric fibres considerably reduced the probability of spalling of concrete cover (Wille, Schneider, 2002; Dehn, Wille, 2004; Janson, Boström, 2004; Dehn, Werther, 2006) (*Fig. 17*).

Experiments with tunnel segments (length 11 m, height

Fig. 16: Mechanism of spalling (Høj, 2005)



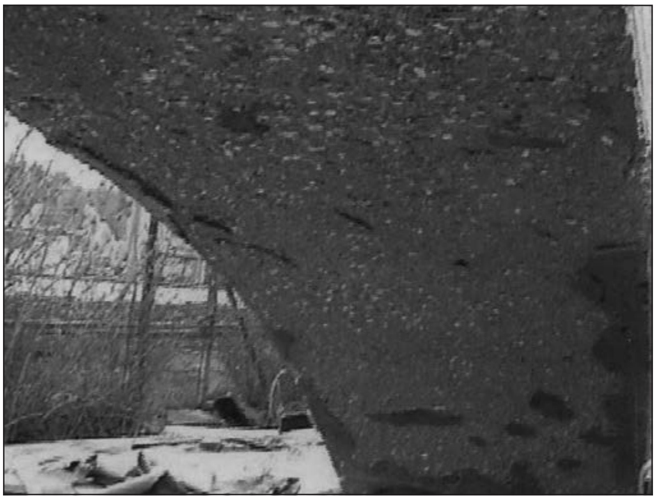


Fig. 17: *Tunnel segments after exposure to 1200 °C temperature (Mörth, Haberland, Horvath, Mayer, 2005)*
a) without fibre reinforcement *b) with 2 kg/m³ polypropylene fibre reinforcement*

2 m) carried out by Mörth, Haberland, Horvath and Mayer (2005) indicated that the cover of the polypropylene fibre reinforced concrete did not spall. In Austria a group of researchers (Walter et al., 2005) had the same finding (*Fig. 18*).

They tested reinforced concrete slabs which were loaded in their planes. Spalling of concrete cover developed in case of the conventional reinforced concrete slabs without polymeric fibres. However, no spalling was experienced in slabs made with 1 to 3 V% polypropylene fibres. Silfwerbrand (2005) suggested that application of polypropylene fibres is preferable also for high strength concrete.

Utilisation of polypropylene fibres does not only reduce the probability of spalling of concrete cover but it may reduce the residual compressive strength (Dehn, König, 2003). Horiguchi, (2004; 2005) experimentally proved on cylinders (Ø=100 mm, ℓ=200 mm) that the addition of polymeric or steel fibres has changed the value of the residual compressive strength. Specimens were heated by 10 °C/minute rate up to 200 °C or 400 °C, then kept for 1 hour at high temperature, and finally tested at room temperature *(Fig.19)*. The water to cement ratio was 0.3 (with 583 kg/m³ cement and with 175 ℓ water). Mix A: prepared without fibres, Mix B: with 0.5 V% polypropylene fibres, Mix C: with 0.5 V% steel fibres, Mix D: with 0.25 V% polypropylene and 0.25 V% steel fibres.

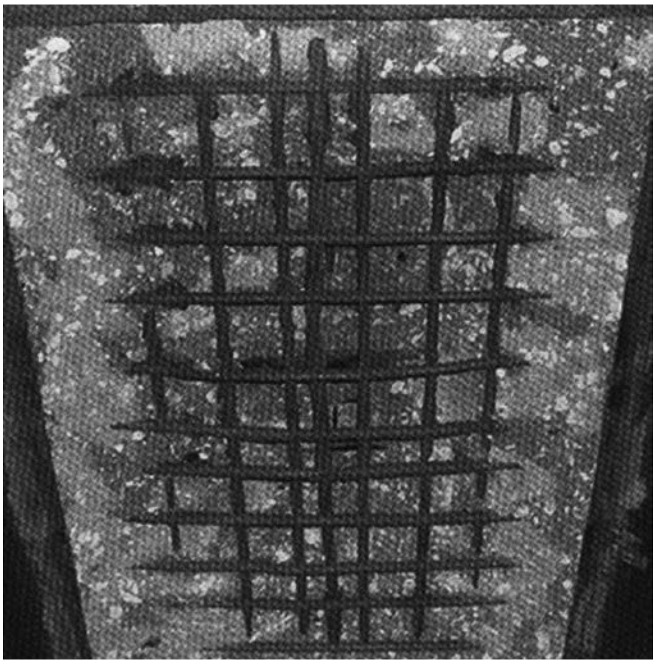
A group of researchers (Horiguchi, 2005) experimentally proved on cylinders that the addition of synthetic or steel fibres has changed the value of the residual compressive strength. The experiments were carried out at 20 °C, 200 °C and 400 °C. Cylinders (Ø=100 mm, ℓ=200 mm) were heated by 10 °C/minute rate, then kept for 1 hour at high temperature, and finally tested at room temperature (*Fig. 19*). The water-cement ratio during the experiment was 0.3 (with 583 kg/m³ cement).

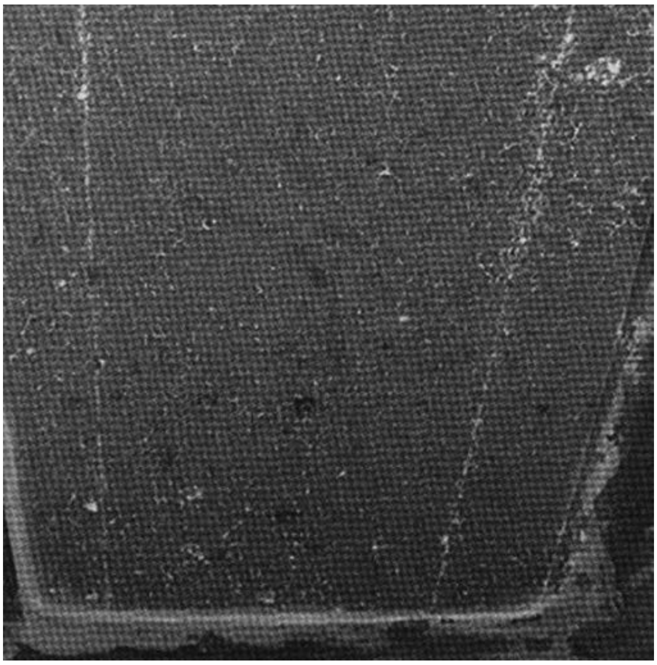
- Mix A was prepared without fibres,
- Mix B with 0.5 V% polypropylene fibre reinforcement,
- Mix C with 0.5 V% steel fibre reinforcement,
- Mix D with 0.25 V% polypropylene 0.25 V% steel fibre reinforcement.

3.2 Deformations during fire

Separation of deformations due to temperature increase may lead to considerable deformations even in parts of building without temperature increase (*Fig. 20*). For instance, columns can shear off as a result of the thermal expansion of a fire exposed floor (*Fig. 21)* (*fib*, 2007).

Fig. 18: *Surface of the slabs after 2-hour fire exposure (Walter, Kari, Kutserle, Lindlbauer ,2005)*
a) without fibre reinforcement *b) with synthetic fibre reinforcement*





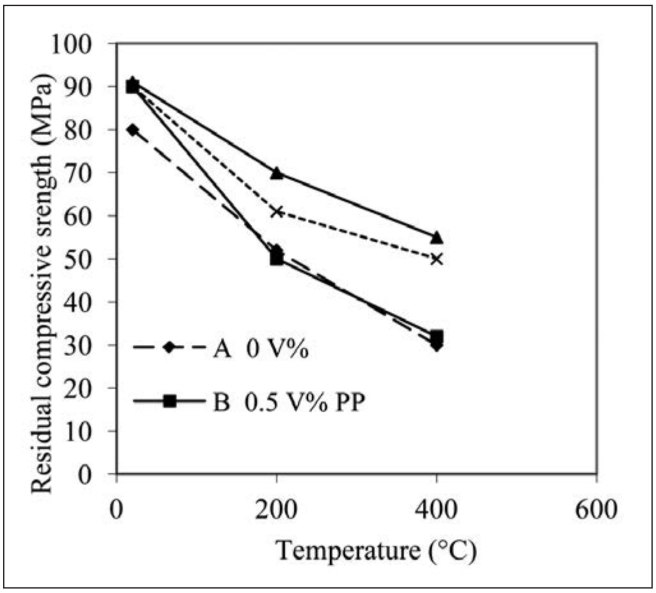


Fig. 19: *Residual compressive strength of high strength concrete with or without fibres (Horiguchi, 2005)*

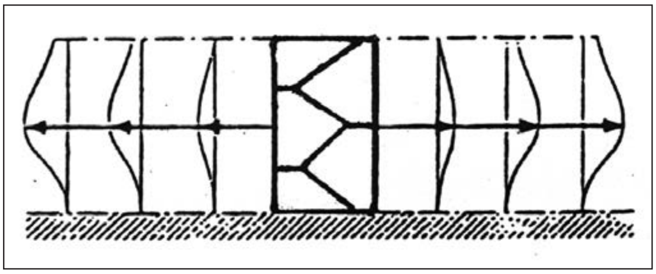


Fig. 20: *Deformations during fire (fib, 2007)*

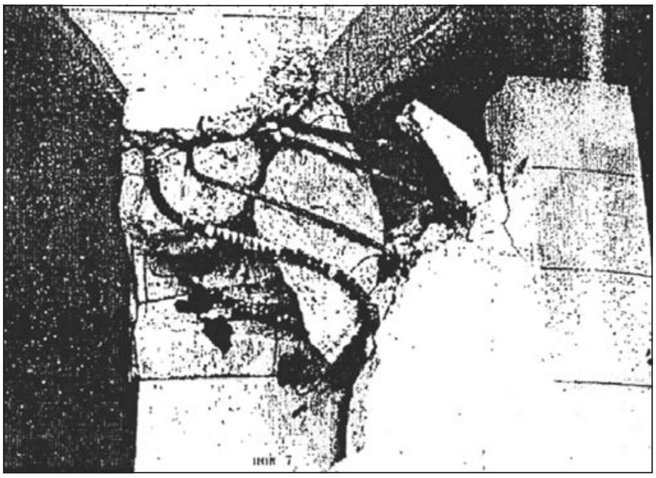


Fig. 21: *Example of shear failure in an unexposed concrete column due to the thermal expansion of the fire exposed floor connected to it (fib, 2007)*

3.3 Bond behaviour

An experimental study was carried out on pull-out specimens with following parameters:

- temperature (maximal temperatures were: 20 °C, 50 °C, 150 °C, 300 °C, 400 °C, 500 °C, 600 °C or 800 °C)
- aggregate (quartz gravel or expanded clay)
- fibers (plastic or steel).

The pull-out specimens (*Fig. 22*) – with diameter 120 mm and height 100 mm – were subjected to a 2 - hour exposure to high temperature. The pull-out tests were carried out after cooling down, at room temperature.

The compressive and the bond strength decreased significantly by increasing the temperature. *Fig. 23* presents

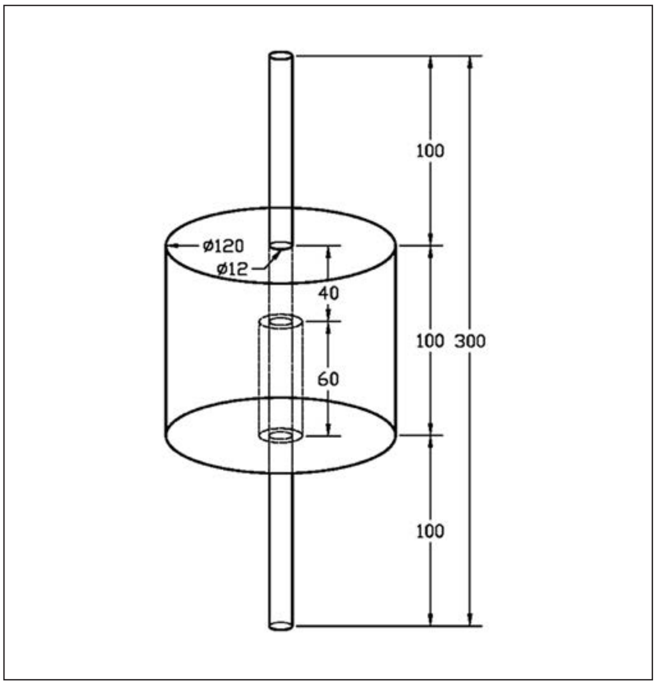


Fig. 22: *Pull-out specimen*

the compressive and average bond strengths of concretes with different composition stored at 20 °C (reinforcement S500, deformed bar, f_{yk}=500 N/mm²). According to the figure the following conclusions can be drawn:

(1) The compressive strength of concrete with expanded clay aggregate measured at 20 °C was less than the concrete with quartz gravel aggregate.
(2) The bond strength of reinforced concrete with steel fibers at 20 °C is higher than the concrete with quartz gravel aggregate without fibres.
(3) The bond strength of reinforced concrete with polypropylene fibres and lightweight concrete at 20 °C is less than the concrete with quartz gravel aggregate without fibres.

Fig. 24 indicates the ratio of the bond strength to compressive strength of concrete as a function of the maximum temperature. Between 400-500 °C a significant strength reduction took place. In the temperature ranges 20 °C to 400 °C and well as 500 °C to 800 °C the strength ratio decreased approximately linearly.

In case of concrete with quartz gravel aggregate, the crystallised structure of the portlandite layer is clear (*Fig. 25*). There is also portlandite layer on the surface of the reinforcement.

4. CONCLUSIONS

Present paper summarizes the most important influences of fire on concrete structures. Deterioration of material characteristics and structural performance highly depend on constituents and on the temperature history. Design for high temperatures requires additional aspects of material composition and material characteristics compared to design for ULS and for SLS.

Material properties

Concrete has excellent properties in regards of fire resistance compared with other materials and can be used to shield other structural materials such as steel. When exposed it to high temperatures, concrete undergoes changes in its

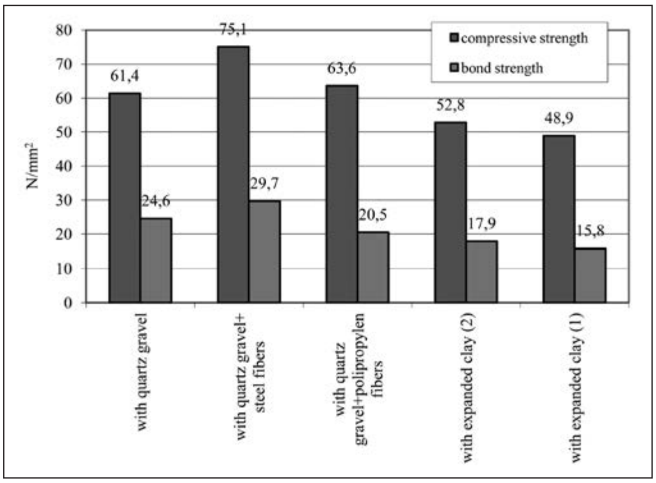


Fig. 23: Concrete compressive strength and bond strength with various mixes at 20°C

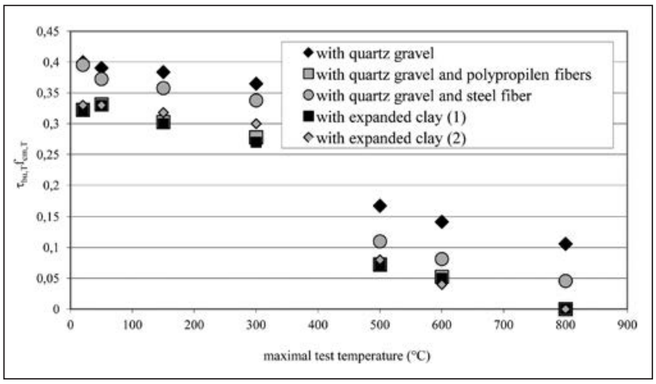


Fig. 24: $\tau_{bu,T}/f_{cm,T}$ as a function of maximal temperature

chemical composition, physical structure and water content. Such changes are reflected by changes in the physical and mechanical properties of concrete that are associated with temperature increase.

An extensive experimental study was carried out at the Budapest University of Technology and Economics Department of Construction Materials and Engineering Geology on temperature changes of concrete parameters like type of cement, type of aggregate and type of fibre.

We should distinguish between post-heating strength and hot strength of concrete. In both most considerable strength reduction takes place in both cases between 400 °C and 800 °C. Hot strength values for high temperatures can be slightly higher than post-heating strength.

A strength valley is observed for relatively low values of maximal temperatures, i.e. a small strength decrease then small increase between 20 °C to 300 °C, respectively.

Structural performance

Especially in tunnels it is important to avoid spalling of concrete cover. A great number of experiments supported that the application of synthetic fibres considerably reduced the sensitivity for spalling of concrete cover. In case of fires in tunnels, in addition to the reduction of load bearing capacity, spalling of concrete cover causes further difficulties. The probability of spalling of concrete cover increases by increasing the strength of concrete.

Separation of deformations due to temperature increase may lead to considerable deformations even in the parts of a building without temperature increase.

In the ratio of the bond strength to compressive strength of concrete as a function of the maximum temperature took place Between 400-500 °C a significant strength reduction. In the temperature ranges 20 °C to 400 °C and as well as 500 °C to 800 °C the strength ratio decreased approximately linearly.

In case of fire the behaviour of the expanded clay aggregate needs further experiments, but so far, it looks promising if synthetic fibres are also added.

5. ACKNOWLEDGEMENTS

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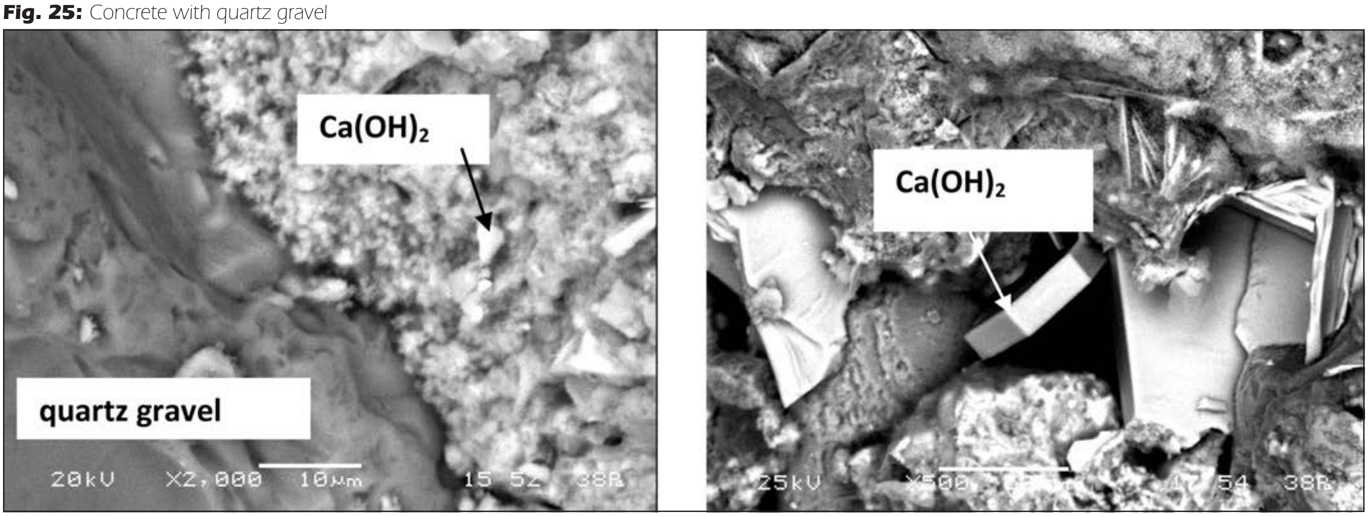
Fig. 25: Concrete with quartz gravel



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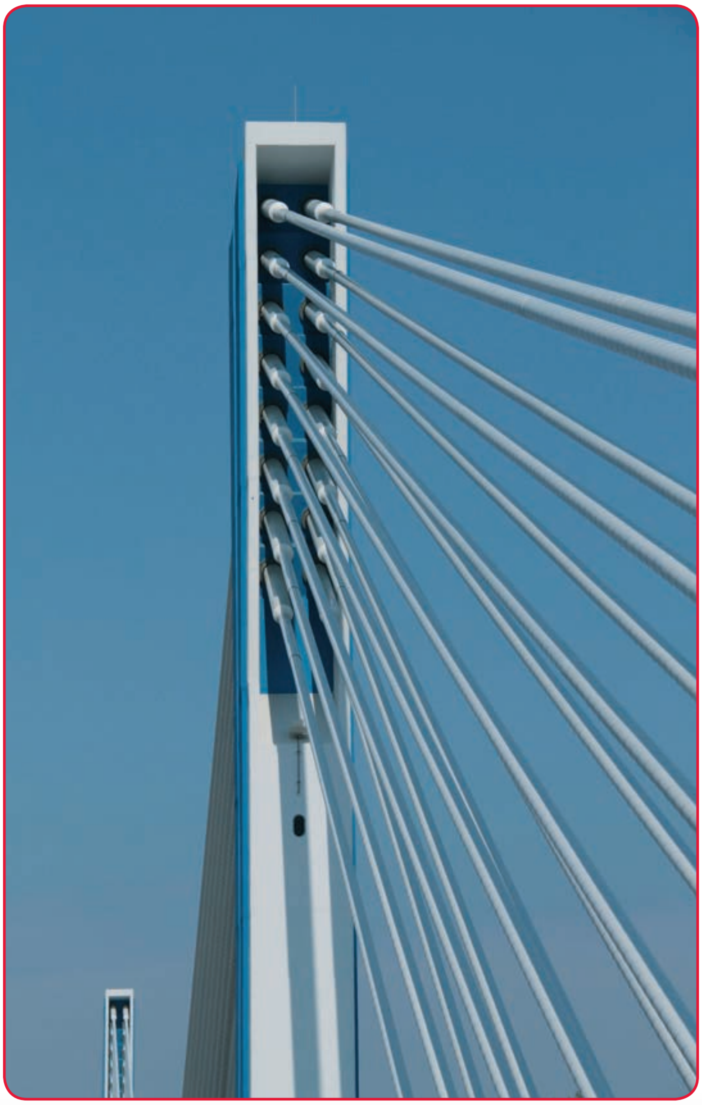

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