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Interaction of nature and materials - The photo was taken in India during the ***fib***-days 2012 in Chennai.
(Photo: György L. Balázs)

CONTENT

INDIA AND HUNGARY FROM HISTORY TO CONCRETE


Géza Tassi – György L. Balázs

This journal of the Hungarian Group of fib has the honour to survey the connections between the organizing country of the yearly fib meeting and Hungary. On the occasion of the fib Congress 2014, we take this opportunity to highlight the excellent cultural relations and cooperation in concrete technology following the establishment of the independent state of India.

1. INTRODUCTION

1.1 THE AIM OF THIS PAPER

It has become a tradition that the leading article of our journal deals with the connections between Hungary and the countries of the *fib* Member Group that organizes the yearly international meeting.

In 2014 the congress of *fib* is held in Mumbai, India and we anticipate that the congress will be highly successful. We base our very positive comment on our awareness of the advanced concrete technology of India and the experience provided by previous international meetings held in India which have served very well to further the cooperation between member groups and has contributed to advancing technology in the building industries of countries of all continents.

1.2 THE MUTUAL KNOWLEDGE OF THE PEOPLE OF OUR TWO COUNTRIES

India is a very large country in the subcontinent of Asia. Hungary is a very small country situated in Middle Europe. The population of Hungary is approximately 1% that of India. There follow other differences. India is the root of the great majority of European languages while the Hungarian people have a mother-tongue which is among the very few languages in the European continent that is not an Indo-European one.

In spite of geographical distances and geo-ethnic differences there is a similitude, mainly in our very rich traditions and culture.

We have no quantifiable information about what Indian children and young people know about such a small European country, Hungary, other than what they may learn in geography and history. A great number of Hungarian children have their first lesson in kindergarten about a land called India through learning a verse for the young penned by the Hungarian poet Lőrinc Szabó (Szabó 2013). 10-12 years olds and teenagers of the 20th century mainly became acquainted with the land of India through reading “The Jungle Book” by the British novelist R. Kipling (1865-1936) (Kipling 1894) or reading Indian Fairy Tales by the famous Hungarian orientalist Á. Vámbéry (1832-1913) (Vámbéry, 1905). In most Hungarian junior secondary schools today, geography and history curriculum includes important information about India relating both to contemporary times and the period of independence, post 1947.

Hungarian citizens travelling in India relied on the well known guidebook (Fodor, 1976).

There are many areas of significant connections between India and Hungary, for example, computer science and technology, which is highly developed in India. Specialists working in this field are most likely familiar with the pioneers of informatics: János (John) Neumann (1903-1957), born and educated in Hungary, as well as Dénes (Dennis) Gábor (1900-1979) Nobel Prize winner, inventor of holography.

The name of Ferenc Puskás (1927-2006), an excellent Hungarian footballer and coach, is famous in sporting circles of India.

No doubt the most famous Indian personage whose name is deep in the conscience of Hungarians is Mahatma Gandhi (1869-1948). His popularity is reflected in the postage stamp

Fig. 1: Mahatma Gandhi in Hungarian postal stamp

bearing his portrait, issued in 1969 on the occasion of the centenary of Gandhi's birth *(Fig. 1)*, [Del. & sc. by the famous lithograph S. Légrády (1906-1987)].

The majority of Hungarian intelligentsia is familiar with the classical Sanskrit books "Mahabharata" and "Ramayana" (Baktay, 1960), as well as "Panchatantra" (Schmidt, 2010). There are many adults in Hungary who also take in interest in the "Kama-Sutra" (Baktay, 1971).

It is generally known that Yoga is a system of inspiration, gymnastics and meditation of Indian origin and many Hungarians engage in the practise of Yoga. Most Hungarians know that the word "Maharaja" is connected with India, and its meaning is "Great King".

In Hungary, as in India, there is popular interest in sports. From India came the modern equine sport of polo, which was introduced to Europe via the British in the 19th century. The first polo club was founded in 1862 in Manipur, India.

2. HISTORY

There is a disparity in age of the two countries and in the social heritage of the inhabitants. The ancestors of the people of India came to the Indian Peninsula more than 3000 years ago whilst Hungarian ancestors arrived a little more than 1000 years ago to the Carpathian Basin. The common feature of the history of these two nations is that both were fond of liberty and struggled for it.

Following is a short description of the common events after 1947 leading to the establishment of the modern Republic of India.

2.1 DIPLOMATIC RELATIONS

Diplomatic relations were established in 1948. Both countries have commemorated both the 50th and the 60th anniversary of this.

Decades of Indo-Hungarian economic interaction further sustained the relationship as India became Hungary's biggest Asian trading partner and Hungary had 25 joint ventures in India in the 1980s.

Intense and fruitful scientific and technological interactions created further value. As the fourth largest economy and among the fastest growing markets with an emerging IT (Information Technology) power, India is an important partner for Hungary in Asia.

Over the years the following high-level visits have taken place:

State Presidents: Z. Hussain 1958, P. Losonczi 1969, V.V. Giri 1970, F.A. Ahmed 1975, P. Losonczi 1976, Á. Göncz 1991, S. Sharma 1993.

Prime Ministers: F. Münnich 1962, Gy. Kállai 1966, Indira Gandhi 1972, J. Fock 1974, R. Gandhi 1988, P. Medgyessy 2003, F. Gyurcsány 2008, V. Orbán 2013.

Speakers: B.R. Jakhar 1980, I. Sarlós 1986, R. Ray 1991, Z. Gál 1994, S. Patil 1996, J. Áder 1999, L. Kövér 2012.

From 1962 to 2013 important bilateral agreements were signed between Hungary and India relating to the following: culture; air services; science and technology; protection and promotion of investments; double taxation avoidance; health care; strategic funds; social security; sports; and most recently in 2013, microbiological and radiological protection. Memorandums of understanding (MoU) have been established on traditional systems of medicine and a cultural exchange programme for 2013-2015.

Minister of External Affairs of India, S.S. Khurshid visited Hungary in July 2013, and held discussions with Hungarian Foreign Minister, J. Martonyi, on questions of cooperation *(Fig 2)*.

Fig. 2: Ministers of external affairs of India and Hungary in Budapest, 2013 (Photo: „Véssey Endre, kormány.hu")

A number of other agreements provide the overall institutional framework for economic cooperation with Hungary.

2.2 ECONOMIC LINKS

India's trade with Hungary has been increasing steadily. Bilateral trade in 2002 was as follows (in USD Million): Indian exports 79.2, and imports 23.2. In 2012 these grew to 362.6 and 279.4, respectively.

There are numerous Indian investments in Hungary covering the following sectors: electrical equipment, pharmaceuticals, auto components, IT, electronics, food processing, textiles, logistics, etc. These companies employ approximately 8,000 people in Hungary.

In June 2011, within the Budapest Chamber of Commerce and Industry, a department for the development of trade relations with India was formed.

There exists a small Indian community resident in Hungary. These people are mostly professionals in the IT industry and in business and they include a transient community of about 80 students who are studying in various universities throughout Hungary. From time to time, there is an exchange of research scholars and scientists. In recent years, the number of Indians in Hungary is estimated to be 300-350.

In July 2013 Minister S.S. Khurshid participated in the Annual Conference of Hungarian Ambassadors held in the Ministry of Foreign Affairs in Budapest. The event was opened by Prime Minister V. Orbán. Prime Minister V. Orbán visited

Fig. 3: Manmohan Singh Prime Minister of India welcomes Viktor Orbán Prime Minister of Hungary in New Delhi 2013, (Photo: „Burger Barna, kormány.hu")

India in October 2013. This was an important event improving the good links *(Fig. 3)*.

It is most important for Hungary to have the best possible connection to India, a nation that has the second largest population of the world. The rapid growth of economic potential and significant political status indicates that India is considered to be an important global partner. We mutually enjoy positive cooperation in many fields of culture, industry commerce and political life.

3. CULTURE

There are clear differences between India and Hungary, as a result of geographical situation, ethnic and historical variances. However, there are fields of human endeavour that bring these different nations closer to each-other.

Among these the most international territory is the arts. The strongest spiritual links between India and Hungary are therefore, music, dance, fine arts, and to some extent, movie pictures and literature.

In this short paper of our technical journal it is only possible to highlight a few examples, and without any systematic order:

The flagships of our cultural links are the two cultural centres: The Indian Cultural Centre of Hungary, which operates within the Indian Embassy in Budapest, whose aim is to strengthen the cultural ties between the two countries, and The Hungarian Information and Cultural Centre in New Delhi which was established in 1978 as the first Hungarian cultural institute in Asia. Not only is the building beautiful, but over the past 34 years it has been acknowledged as being among the best and well known of foreign institutions.

The emblematic figures of this cultural relationship are Rabindranath Tagore, Sándor Kőrösi Csoma and Ervin Baktay, about whom we write later in more detail.

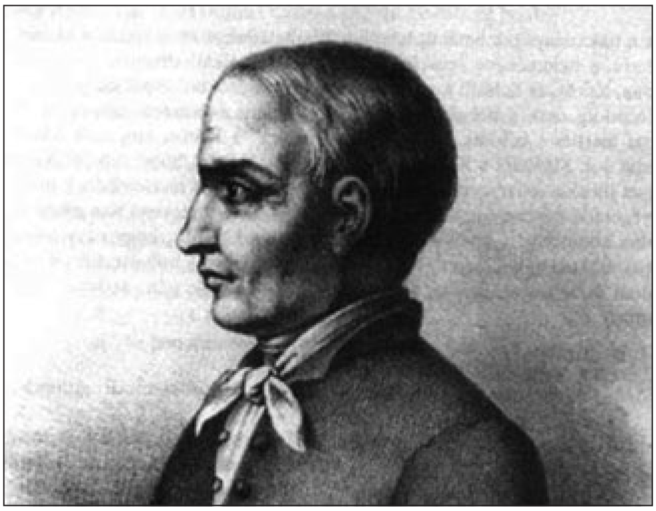
The Indian Cultural Centre hosts many events such as dance shows, yoga classes and various exhibitions. They also have a Film Club where new and older Indian films are screened.

In the Hungarian centre there are various programmes dealing with science, music and literature organized in the capital and throughout India. Following we honour the activity of Hungarians who lived and worked in India and support their life's research.

3.1 ORIENTAL SCIENCE

Sándor Kőrösi Csoma Hungarian scientist (1787-1842) *(Fig. 4)* educated in the famous school "Bethlenianum" in Nagyenyed (today Aiud). Kőrösi Csoma had a special interest in ancient history of the Hungarian people. From 1816 he continued his studies in Göttingen. There, using the rich library and help of famous professors, he was introduced to oriental studies. He learned many foreign languages – among them eastern languages – and in 1819 he set out for the East to seek the roots of the Hungarian people.

Fig. 4: Painted portrait of Sándor Kőrösi Csoma



After a long stay in the Near East Kőrösi Csoma arrived in Persia (now Iran) and from there he later travelled to Lahore, Amritsar, Jammu, Kashmir and Leh. His aim was to reach Middle Asia across Karakorum and Yarkend. This was a dangerous route for a European and he therefore turned back to Lahore, travelling from there to Leh. It was here that his interest in Tibet emerged.

Kőrösi Csoma hoped that in studying Tibetan literature he would find information about the origins of the Hungarian people. During that time the British-Ladakh connection was established. He left Kashmir in spring 1823 and arrived in Leh. From there he travelled to the community of Zangla where he remained until October 1824. Here he learned the Tibetan language and started studying Tibetan literature. He studied Tibetan Sanskrit texts from the 9th century which became the base of his later work, the first English-Tibetan dictionary containing 30,000 words. In 1825 he arrived at Sabatu colony and started to work for the British in Zanskar. In 1830 he went back to Sabatu and in 1831 arrived in Calcutta to prepare the publishing of his works and to earn his living as a librarian. In 1834 his two main works were published, the Tibetan grammar and the dictionary.

Kőrösi Csoma made a decision to remain in India and continue his studies of Sanskrit and the other languages of India. In 1831 he joined the Royal Asiatic Society of Bengal in Calcutta and in 1833 was unanimously elected as honorary member. From 1837 to 1841 he worked as a librarian of the Asiatic Society.

For the remainder of his life he was occupied in the study of Tibetology. In 1842 he set out once again for Tibet, then to North China to the land of Uyghurs and Mongols. Kőrösi Csoma did not reach Lhasa as planned and arrived in Darjeeling in April 1842 after contracting Malaria in Terai. He died in Darjeeling from the fever. His grave is in a cemetery of Darjeeling on the slope of the Himalaya Mountains with an inscription in Hungarian *(Fig. 5)*.

In the preface of his books he emphasized that he was a Hungarian researcher who gave services to India.

Ervin Baktay (1890-1963) *(Fig. 6)* Hungarian writer, art historian, orientalist, travelled throughout India. In the 1920s he was a recognized writer-educator of Indian culture in Hungary (Baktay, 1958). He offered two books to Rabindranath Tagore before he visited the Visva-Bharat University in Shantiketan in 1926. From 1946 to 1962 he was active in the Ferenc Hopp Museum of East Asian Art in Budapest. In his book on Sándor

Fig. 5: Tomb of Kőrösi Csoma and the inscription

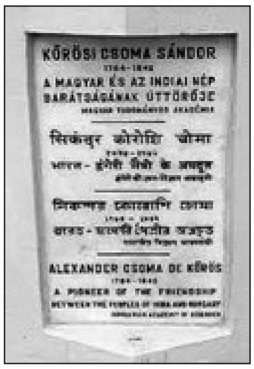


Fig. 6: Ervin Baktay

Kőrösi Csoma (Baktay, 2009) he accompanies the reader along the journey of Kőrösi and engages one to feel the spirit of the great Hungarian orientalist. Baktay also shows the soul of the Indian people in his book on traditional Indian mythology (Baktay, 2004). The wisdom of India is well detailed in his book (Baktay, 2006). Through his translations Baktay also helped Hungarians to become acquainted with the most famous of ancient Indian books (Baktay, Vekerdi, 1997), (Baktay, Apostol, 1960), (Baktay, 1971).

Gyula Germanus (1884-1979), the internationally renown Hungarian orientalist, was in 1927 invited by Rabindranath Tagore to be the Head of Department of Islamic Sciences of Visva Bharati University in Shantiketan. He also lectured at another seven Indian universities. He published two books on his experiences in India (Germanus 1934). Professor Germanus arrived with his wife Rózsa Hajnóczy (1892-1944) and they remained in India for some years. She recorded her impressions in her novel “Fire of Bengal” (G. Hajnóczy, 1943).

Oszvald Szemerényi (1913-1996) was educated in Hungary at Eötvös Loránd University of Science in Budapest, and he further studied at the universities of Heidelberg and Berlin. He was a Hungarian –Europeanist with strong interests in comparative linguistics in general. He was influenced by Hungarian linguist Gyula Laziczius (1896-1957). In 1942 he was appointed lecturer at Budapest University. In 1944 he habilitated, and in 1947 he was appointed professor of comparative Indo-European linguistics in Budapest. He lived in England from 1948, then in Germany from 1965 to 1981. He published his numerous works under the name Oswald John Louis Szemerényi.

János Harmatta (1917-2004) was an excellent Hungarian linguist. He was Professor and Head of Department of Indo-European Languages at Eötvös Loránd University of Science in Budapest. Among many other fields he was an outstanding expert in indology and he lectured on Sanskrit language. He did much to improve the good connections between scientists of India and Hungary.

The *Ferenc Hopp Museum* of East Asian Art in Budapest was founded in 1919. It is the only museum of Oriental art in Hungary. The founder, Ferenc Hopp (1833-1919), was a devout collector of pieces of art. He travelled across many countries of the globe, also visiting India. On these journeys he acquired precious stones and pieces of art. Before he died in 1919, he donated his collection and his villa in Budapest for the purpose of a museum of Oriental art. Today, beside the exhibitions there is a library and a research centre. In 1982 an exhibition of Indian art was held at which the Ambassador of India was in attendance.

The *Gandhi secondary school* in Pécs was founded in 1993 for education of the Roma (Gypsy) population living mainly in South-West Hungary.

Alongside the schoolboys and schoolgirls adults also received education including, among other subjects, knowledge in field of the Gypsy ethnography, about their ancient homeland in India and their wanderings around Europe.

A significant event on 9th February 2013 was the inauguration of the sculpture of Mahatma Gandhi in presence of the ambassador of India, Gauri Sankar Gupta. The bronze bust was donated by the Cultural Council of India on the occasion of the 20th anniversary of the foundation of the institution.

3.2 LITERATURE AND ART

Links in the field of art and literature between India and Hungary are very broad. We mention here only a selection of significant examples without any systematic order.

Fig. 7: Rabindranath Tagore

Rabindranath Tagore (1861-1941) *(Fig. 7)* an Indian poet and writer, was born in Calcutta. From 1878 he continued his studies in England and from 1890 lived in the countryside in India. From 1905 he supported the movement for the liberation of India.

In 1912 he was again in England and published his poems translated into English. For this volume he was awarded in 1913 the Nobel Prize for literature. 1924-30 Tagore travelled in several countries. He was an opponent of fascism and of the war.

In 1926, during his travels in Europe, he came to Hungary where he spent some time in the heart sanatorium of Balatonfüred. There he was treated by the famous Hungarian professor of medicine, S. Korányi (1866-1944). Today, one can find in this sanatorium a memorial hall named after Tagore.

During his sojourn in Hungary Tagore met many prominent representatives of Hungary’s literature. Between 1920 and 1925 about 30 of his works were published in Hungarian. The esplanade along the shore of Lake Balaton is named Tagore Esplanade, and close to the pier a memorial preserves his leaning towards Hungary *(Fig. 8)*.

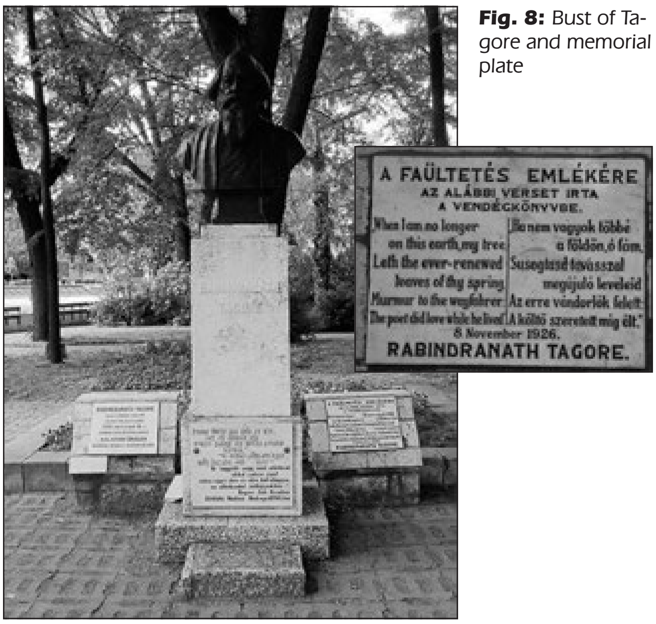


Fig. 8: Bust of Tagore and memorial plate

In 2013 when Minister of External Affairs of India, Shri Salman Khurshid, visited the Tagore monument at Balatonfüred, the community held a tree planting ceremony along the Tagore Esplanade. The neighbouring place is now named India Park. In this place in 1968 his grandfather, President of India, Zakar Hussain, planted a tree. Let us mention here that Ch. R. Alimchandani, president of *fib* Group of India visited this same place after the *fib* Symposium Budapest 2005.

Jayadeva (about 1200 AD-?) was the last great poet of Sanskrit poetry, the poet whom the Hungarian poet S. Weöres (1913-1989) translated with genius into Hungarian. It was an intimate occasion of Indian-Hungarian cultural links, when at a meeting in New Delhi a young Hungarian journalist was reciting the poem of Jayadeva as translated by Weöres (Jayadeva, 1982). The Indian journalist recognized from the sound and rhythm, that it was a Sanskrit poem in Hungarian.

Fig. 9: Self portrait of A. Sher-Gil

Amrita Sher-Gil (1913-1941) *(Fig. 9)* was an Indian painter who was born in Budapest to her Hungarian mother and Sikh father. Her first language was Hungarian and she was related to the Hungarian scientist, E. Baktay.

The family left for India in 1921. She studied art in France and came back to Hungary in 1938. In 1939 they returned to India. Amrita Sher-Gil is acknowledged in India as a national treasure. In 2010 twenty six of her pictures were exhibited at the Ernst Museum in Budapest. The permanent exhibition of the National Gallery of Modern Art in New Delhi begins with her creations. On the occasion of the centenary of her birth in 2013 UNESCO declared a Sher-Gil memorial year.

On visit in 2013, Minister of External Affairs, S. S. Khurshid, participated in the opening ceremony of the exhibition of this famous painter at Vaszary-villa in Balatonfüred.

Two Hungarian painters, *Erzsébet Sass Brunner* (1889-1950) and her daughter Erzsébet Brunner (1910-2001), moved to India in 1930 and both became acknowledged artist there. Numerous famous Indian historical persons were captured for posterity in their paintings. In 2010 an exhibition was opened in New Delhi in Indira Gandhi National Centre for the Arts. They received several high level awards, both Indian and Hungarian.

Sivasakti Kalananda is a dance theatre *(Fig. 10)* and was founded in Budapest in 1997. The aim was to popularise among the Hungarian public the Bharatanatjam classical dance form of South India. The Hungarian founder, Panni Somi, started teaching Indian dance in 1991. The theatre produced more than 400 performances in the classic style of India as well as plays of contemporary authors. The ensemble received numerous invitations to different forums abroad.

Fig. 10: Sivasakti Indian dance ensemble performance in Hungary

Fig. 11: Ravi Shankar

Fig. 12: András Kozma

Anuradha Shinde dance teacher and artist was active after 2002 in Hungary and introduced many pupils to the mysteries of the art of Indian dance.

Ravi Shankar (1920-2012) (*Fig. 11)* the best known musician of India has been three times to Hungary, last in1997. He commenced teaching A. Kozma in 1980 and Ravi Shankar founded the Ravi Shankar Institute for Music and Performing Arts and from it about 50 Indian dance and music ensembles have appeared on stages in Hungary.

András Kozma (1952-) (*Fig.12)* was the first in Hungary who from 1970 studied the classical music of India. From 1980 his teacher was the internationally renowned sitar virtuoso, Ravi Shankar. He was the only European student of his master. Kozma spent more than 10 years in India. He did much to popularize Indian music in Hungary and in many other countries.

Kozma also studied philosophical systems of India, the Sanskrit and Hindi language.

He founded the Calcutta Trio in Budapest which has weekly performances and is the only regular program in Europe dealing with Indian culture. Kozma lectured at different universities of India and was awarded three honorary doctor titles.

4. CONCRETE

As a slogan of a previous *fib* meeting announced, "concrete is a bridge between nations". If we mention a few examples of common activity between India and Hungary in this field – without striving for completeness – we can strengthen the truth of this statement.

Firstly we would reflect on the marvellous creations from antiquity to today, the classical structure built from natural stone, the Taj Mahal in Agra, to the contemporary shape of the Baha'i Temple in New Delhi with its white concrete shell structure.

The Hungarian participants of the FIP Congress 1986 in New Delhi (including Author[1]) visited the Baha'i Temple then under construction (*Fig. 13*). Author[2] was delighted to visit the Baha'i Temple 25 years later, in January 2011, during his visit for the *fib*-course in New Delhi. The Baha'i Temple is a beautiful example of concrete engineering (*Fig. 14*).

Today one can see many developed concrete structures, bridges, hydraulic works, and power stations, industrial, communal and residential buildings. We mention results of advantageous collaboration between the specialists of the friendly nations of India and Hungary.

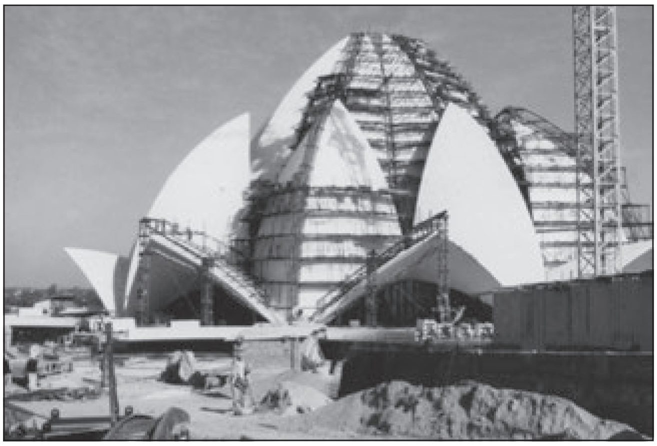


Fig. 13: The Baha'i Temple in New Delhi during construction (1986)

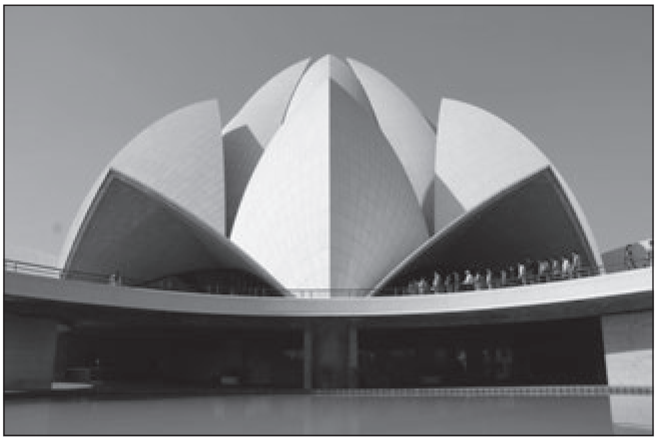


Fig. 14: The Baha'i Temple in New Delhi as fib delegates admired it in 2011

4.1 CONNECTIONS AMONG CONCRETE SPECIALISTS

István Medgyaszay (1877-1959) was a Hungarian specialist in reinforced concrete. He designed the structures of the theatres of Veszprém, Sopron and Nagykanizsa (Hungary) and was an invited professor of the Technical University of Budapest. In 1911 he travelled in India and from 1930 was associate president of Hungarian-Indian Society (Ötvös, 2011).

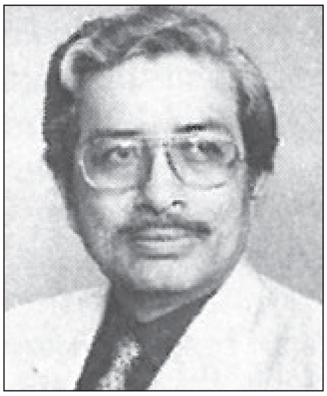


Fig. 15: Chander R. Alimchandani

Chander R. Alimchandani (1935-) (Fig. 15) is an outstanding, celebrated civil and architectural engineer. He graduated in 1957 from Poona University (India). He studied prestressed concrete design and construction under a scholarship in France. He worked with Y. Guyon and P. Xercavins. In 1963 Alimchandani joined as deputy chief engineer of STUP Consultants Ltd. (Bombay). In 1967 he became chief engineer and in 1972 the first managing director and in 1975 chairman and managing director.

Under his leadership there were constructed numerous outstanding engineering structures for various purposes winning international recognition.

He has been vice president of FIP respectively *fib* since 1978. He was the chairman of the Organizing Committee and International Scientific Committee for FIP Congress 1986 in New Delhi. For the period 1985-86 he was president of the Institution of Engineers (India). Ch. R. Alimchandani established strong connections with Hungarian delegates *(Fig. 16)*. His merits, taking into account his work in India as well as his international activity, were recognized by the FIP Medal in 1986. (Crozier, 1986).

Alimchandani hosted Hungarian specialists in India and visited Hungary several times. From these visits emerges his contribution, together with his son, to the *fib* Symposium 2005 in Budapest (Alimchandani, Ch. R., Alimchandani, A.C., 2005).

There were a number of Indian engineers who received their doctoral degree at the Budapest University of Technology and Economics. From among them the following specialists were involved with concrete structures.

The study of *Om Prakash Chhangani,* who defended his thesis in 1986, was about the deflections and load bearing capacity of one and two way reinforced concrete slabs. His scientific leader was Prof. P. Lenkei D.Sc., owner of *fib* Medal of Merit.

Radha Karta Sarkar presented his thesis on shell structures in 1987. His supervisor was Prof. I. Hegedűs D.Sc. Széchenyi, prize holder.

Fig. 16: I. Fogarasi, C.R. Alimchandani and J. Beluzsár at the Tenth Congress of FIP, New Delhi

4.2 INTERNATIONAL PROFESSIONAL MEETINGS

There were several international meetings on concrete technology in India at which Hungarian specialists contributed and vice versa:

The International Conference on Shear, Torsion and Bond in Reinforced and Prestressed Concrete, held in Coimbatore, 1969. (Tassi, Baranyay-Horváth, 1969), (Juhász, 1969).

The Tenth Congress of FIP was the first noteworthy event in the life of FIP that took place in Asia. The congress in New Delhi was the meeting at that time. It had the highest number of participants. 55 countries were represented. Without doubt the vast majority of more than 2200 delegates came from different regions of India. This provided a good opportunity for participants coming from other countries to become better acquainted with development in this huge country.

The members of the Hungarian delegation were J. Beluzsár, I. Bódi, Gy. Fogarasi, I. Fogarasi, E. Lakatos, M. Loykó, M. Márkus, T. Sigrai, B. Sztanó, G. Tassi, L. Varga, J. Vörös.

The congress in 1986 was organized by the Institution of Engineers of India. Chairman of the organizing committee was Ch. R. Alimchandani (India).

Hungary was represented at the FIP Council meeting on the occasion of the congress by L. Garay, who at that time was president of the Hungarian Group of FIP.

The Hungarian delegates performed worthy activities at the congress. At the technical sessions three papers by Hungarian authors were presented (Fogarasi, Gy.1986[2]), (Gecsényi, 1986), (Vörös, Lakatos, Fogarasi, I., 1986). There was also a poster presentation (Tassi, Bódi, Erdélyi, Ódor, 1986).

At the National Reports session Author1 presented the achievements of the Hungarian concrete construction industry in years 1982-86.

Among the booths of the exhibition were two from Hungarian firms displaying their products and services, the NIKEX export company and the State Building Enterprise No. 31.

The book of Gy. Fogarasi (1986[1]), as a gift from the Hungarian Group of FIP, was distributed among the leading delegates of the congress.

During the congress there was a meeting of the FIP Commission on prefabrication, working group on concrete sleepers. Author1 participated and a report was published on the results (Gylltoft et al. 1986).

The International Committee for Concrete Technology in Developing Countries periodically organizes scientific meetings dealing with technical problems. The Fifth International Conference of this organization was held in New Delhi in 1999. Traditionally specialists from European countries are invited. The conference in India was organized by the National Council for Cement and Building Materials, New Delhi, India. The chairman of the organizing committee was C. Rajkumar.

Two Hungarian participants took the floor at the technical sessions (Tassi, 1999) (Orbán, 1999).

The conference demonstrated the high level of achievements of India – among other countries.

The fib Symposium in New Delhi in November 2004 on "*Segmental Construction in Concrete*"was the preceding *fib* Symposium before the *fib* Symposium in Budapest in May 2005. The quality of the program in New Delhi provided a challenge for the organizers of the Symposium in Budapest. The Chairman of the Scientific Committee was Jim Forbes from Australia and the Chairman of the Organizing Committee was Tippur N. Subba Rao, Author[2] was member of the Scientific Committee.

A special series of conferences were established in India entitled *fib-days.* The series of *fib*-days was an excellent concept comprising part of the preparations for the *fib* Congress 2014 in Mumbai.

The *fib*-days had a triple agenda: (i) to make *fib* know in India, (ii) to present the most important results of the previous *fib* international Symposium or Congress to Indian colleagues, (iii) to review the most recent Indian projects in concrete engineering. The *fib-days* were held in the following sequence:

fib-days 2007 Mumbai
fib-days 2008 Bengaluru
fib-days 2009 Calcutta
fib-days 2010 New Delhi
fib-days 2011 Ahmedabad
fib-days 2012 Chennai.

These all served to draw the attention to *fib* of Indian colleagues as well as colleagues from the neighbouring countries to *fib* and especially to the 2014 *fib* Congress in Mumbai. Chander R. Alimchandani and Subhashchandra Joglekar together were the driving force behind organizing the *fib*-days. Author[2] participated in all of the *fib*-days, with exception of the first, by giving keynotes presentations and encouraging Indian colleagues to attend the Mumbai Congress. *Figures 17-20* indicate memorable moments of the *fib*-days.

Fig. 17: Opening of fib-days in New Delhi

Fig. 18: Opening of fib-days in Ahmedabad

Fig. 19: Together with colleagues of fib-days in Chennai

Fig. 20: Together with students of fib-days in Chennai

4.3 CONSTRUCTION WORKS OF HUNGARIAN ENTERPRISES IN INDIA

4.31 HUNGARIAN PARTICIPATION IN THE CONSTRUCTION OF THE FIRST METRO LINE IN INDIA

The attainment of independence of India (1947) was followed by rapid population growth in Calcutta (Kolkata). It was clear

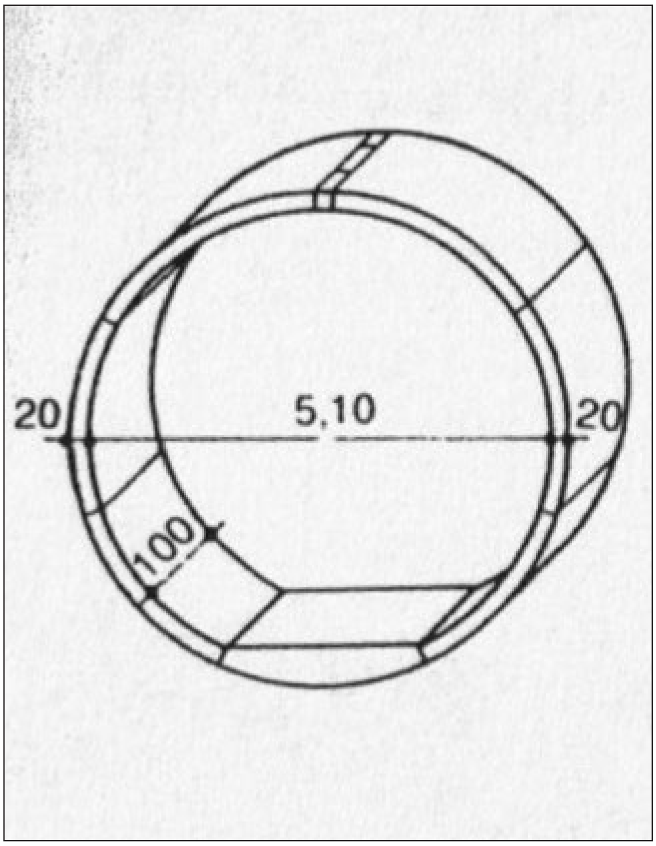


Fig. 21: Cross section of the metro tunnel of Calcutta

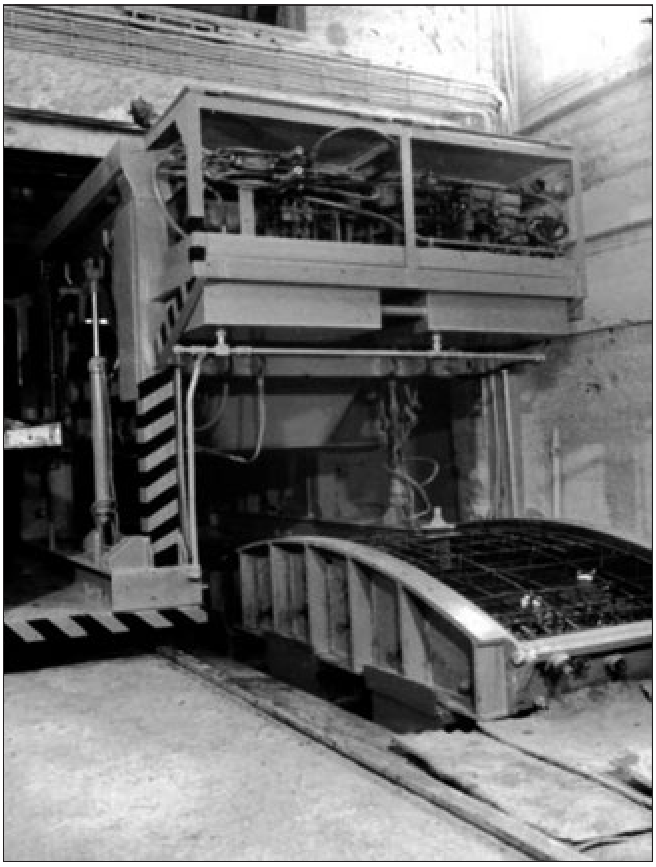


Fig. 22: Manufacturing equipment to prefabrication the tunnel lining segment elements

that the traffic problems could not be solved by surface public transport alone. The idea of the first underground railway line (metro) in India appeared in 1971. The full designed length of the North-South line of Calcutta, crossing the Circular Canal, was 22.3 km. Out of the 21 stations 15 were planned to be situated underground, the others on the surface or elevated. The gauge of tracks is 1676 mm. The planned internal diameter of running tunnels was 5.10 m *(Fig.21)* and the design speed was about 55 km/h.

Prime Minister Indira Gandhi (1917-1984) laid the foundation stone in 1972.

Construction commenced in 1977-78 and Hungary participated in the design and construction of this metro.

The soil conditions and the applicable local designs required tunnelling by shield operation under compressed air and the relevant Indian Authorities issued an international call for delivery of equipment connected to the compressed air operations.

Hindustan Construction Company (HCC), a local firm, was selected as main contractor for the construction of the first lot of tunnelling works.

Hungarian companies that have successfully worked on the implementation of the metro lines in Budapest/Hungary were UVATERV consulting engineers, KÉV-METRÓ tunnelling and civil contractor and BVM reinforced concrete works (prefabrication) under organization of NIKEX Hungarian foreign trade company of heavy industry. They delivered a complete know-how for shield tunnel driving in compressed air applying precast reinforced concrete tunnel lining segment elements with watertight joints and the additional grouting and putting those into operation and managing a long test period. The "Budapest" type shield tunnelling equipment was used and connected to the compressed air operations (personal and material locks, compressors, pumps etc).

Complete manufacturing equipment *(Fig. 22)* and formworks to prefabricate the tunnel lining segment elements were delivered. The RC segment elements in storage are seen in *Fig.23* and the interior of the tunnel in *Fig. 24.*

Coaching and training of HCC's personnel for the entire tunnelling operation was the task of Hungarian experts. A complete working team including designers, construction managers, shift engineers, foremen and tunnelling workers, designer, mechanical and construction advisors were members of the Hungarian team. The trainer of tunnelling miners was I. Ádány *(Fig. 25).*

Supporting the activity of HCC the Hungarian advisors and execution team spent 20 months in Calcutta during the introduction and test period of the works by driving and construction of two sections of running tunnels. Later an advisor team remained there for some more years to support, advise, supervise and manage HCC's works.

Specialists of UVATERV - who mostly contributed to the Calcutta underground design both in India and in Hungary - were the following:

Deputy general manager and chief engineer I. Kozáry

Fig. 23: Precast concrete tunnel lining element in storage

Fig. 24: Interior of the Calcutta metro line under construction

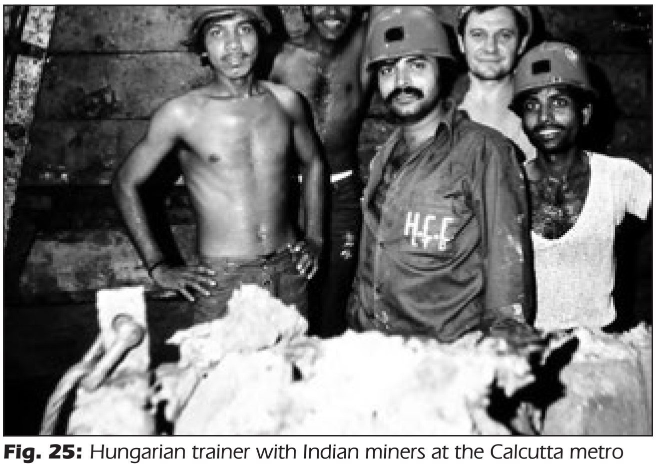


Fig. 25: Hungarian trainer with Indian miners at the Calcutta metro

participated at the International Seminar on Metro Railway Problems and Prospects, organized by G.N. Phadke by order of Ministry of Railway of Government of India.

Fig. 26: L. Rózsa (1925-1993)

The head of underground design department was L. Rózsa (1925-1993) *(Fig 26)* who supported the entire design work and visited many times the construction site in Calcutta. He also contributed to the theory of tunnels constructed by RC tubing elements (Rózsa, 1979).

Engineers of UVATERV made a considerable contribution to the Calcutta metro, among them Cs. Pethő, J. Starkbauer.

The team of KÉV-METRÓ presented significant support to the construction works under the leadership of E. Lakatos, tunnelling director, I. Janitsáry, project director, G. Klados as local representative and chief technical advisor, and their co-workers, Z. Loppert, F. Schmidt, Gy. Friedrich, T. Bohus.

The tunnelling shield used earlier in Budapest was adapted to Calcutta in the workshop of KÉV-METRÓ and then delivered to the construction site and assembled there for operation.

An important first step of Hungarian experts' task was the construction of the concrete structure of the starting pit. BVM contributed the expertise of M. Márkus, L. Tamás, A. Tápai, Z. Várnagy, I. Szabó.

The locks and other equipment for the compressed air tunnelling technology were designed and delivered by KÉV-METRÓ as well as the site construction team and trainers, and the site construction work.

The management of the RC tunnel lining elements system was introduced and directed by the Hungarian engineer G. Klados and later was continued by G. Szőllőssy. The training of the tunnelling technology for Indian specialists was the task of KÉV-METRÓ as well as tunnelling foreman J. Hadas.

There were sections where, because of soil conditions, the tunnelling was performed using cast iron segment lining elements. For other sections UVATERV designed reinforced concrete lining segment elements. BVM delivered the know-how and the design of formworks, out of which one master element was manufactured in Hungary and delivered to the prefabrication site in Calcutta.

The elements were produced using high strength concrete with very rigorous tolerance. (Bodai, 1980). The manufacturing equipment was designed by BVM and the site work manager was sent by them, too. The lining of the circular cross section tunnel was produced of these RC elements with hinged joints. (Rózsa, 1979) The joining of sections was designed by UVATERV.

The design of several sections, as well as the shield arrival pit (to receive the arriving shield), was also done by Hungarian experts based in Calcutta.

There were several other parts of the tunnel sections which were constructed by participation of Hungarian firms. Furthermore, many items of mechanical engineering equipment were constructed by Hungarian enterprises. Hungarian experts in soil mechanics also contributed to the underground works in different subjects such as chemical soil treatment and anchoring.

The mission of Hungarians in Calcutta ended in 1989. Since that time many dozens of kilometres of metro lines were established in various towns of India. Without doubt, the first steps were made by Hungarian experts. We are honoured by our Indian colleagues who further developed the technology that Hungarian engineers applied in Calcutta some decades ago.

4.32 EARLY PRECAST CONCRETE STRUCTURE IN INDIA FOR INDUSTRIAL BUILDINGS DESIGNED BY HUNGARIAN ENGINEERS

Hungarian engineering bureaus designed for India numerous buildings with RC load bearing structures. From among these we mention one important industrial building ensemble.

In 1957 an international tender was announced by an investor from India. The Hungarian IPARTERV design bureau for industrial and agricultural buildings received a contract from the INTEGRAL railway coach factory furnishing unit in Madras, Perambur, India. The responsible chief engineer of the investment was R. M. Sambamoorthi.

The design, preserving the main specifications, was open for any construction material and form. The seven halls consisted of two basic types (Garay, 1984).

The Hungarian site engineer of the designer bureau was G. Nádai (who was a member of the designing group) and also Gy. Seres.

The head of the IPARTERV structural engineering group was L. Garay (1923-2002) *(Fig. 27)* who remained in India for long time during construction.

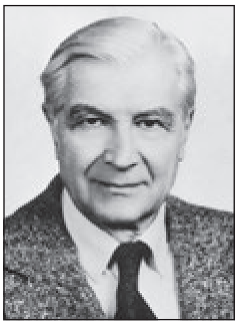


Fig. 27: Lajos Garay

L. Garay was the president of the Hungarian Group of FIP. He was awarded the FIP Medal, and among his many merits was noted his participation in the design of the wagon factory in India (Crozier, 1992).

G. Nádai reported on the complex of the seven halls and this report was published in the Indian Concrete Journal.

The Hungarian design was of precast concrete elements. It was necessary to find a contracting firm that could reliably apply standards of India. The successful contractor was the Hindustan Construction Co. (HCC). The responsible chief engineer was N. S. Gupchup and leading site engineer was K. Rama Rao.

In 1958 the representatives of HCC came to Hungary and the contract documents were signed.

Detailed description of the buildings was written by L. Arnóth. The full ground area was 51,300 m^2.

The main types of hall structures were pile foundation, with pad footings. On these Vierendeel columns were placed. On the columns truss main girders were mounted *(Fig. 28)* with 18.30 m and 15.25 m spans at different halls. The secondary girders were also trusses. Simply supported inverted L-shaped purlins were applied.

The RC windows and the RC trusses are seen *Fig. 29*. The side wall panels are fixed by the columns.

Fig. 28: The erected columns and the trusses before mounting

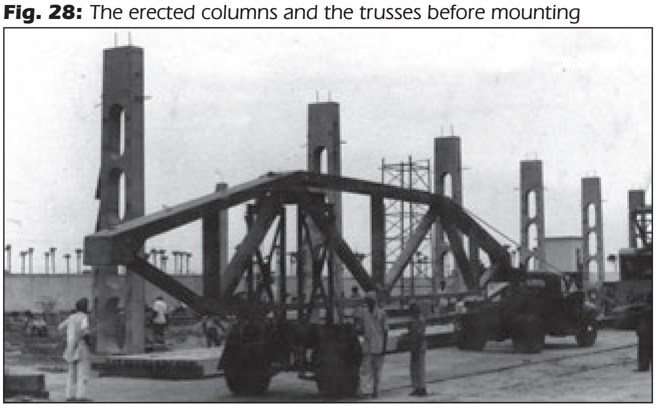


Fig. 29: Truss and side panel

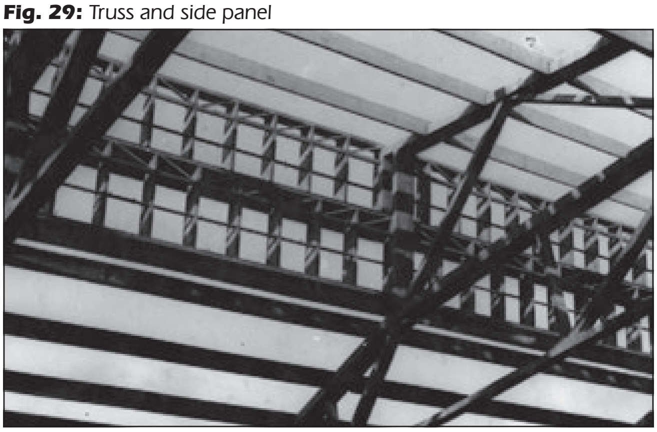


Fig. 30: The site prefabrication

Fig. 31: The erected reinforced concrete structure of a hall

There was a possibility to arrange crane girders. The structure was statically determined for vertical loads. Forming stiff joints between columns and main trusses the stiffness of the structure was partly solved for horizontal loads. The moments on the columns were reduced significantly by the side panels and the windows. Furthermore, at the hall ends under the purlins, wind ties were applied.

At that time precast concrete had already been extensively used in Hungary. In India this technique had still not yet been widely experienced.

The in situ prefabrication plant *(Fig. 30)* at the construction site of the coach furnishing factory consisted of two parts, one for the light panel elements, and the other for the columns, trusses and purlins.

The dismantling of the elements occurred after 24-36 hours and cured for two weeks before mounting.

The contracting work commenced in April 1960. This work became practical and useful experience for blue-collar specialists of India.

An important task of Hungarian site engineer was to ensure adherence to the standards and regulations of India.

The work was the first significant performance of prefabrication in India. K. Rama Rao wrote an acknowledgement of the services provided by the Hungarian trade company, Complex and Design Bureau IPARTERV. The December 1962 issue of the Indian Concrete Journal acknowledged the professional contribution of the Hungarian firms.

One of the halls prior to completion is shown in *Fig. 31.*

4.33 CABLEWAY IN THE PROXIMITY OF THE CITY OF KORBA

Many engineering objects were designed for India by Hungarian experts, following details one of them.

A complex *cableway* system was designed and constructed by Hungarian firms (Sidlovics, 1979) at the location of a bauxite mine and aluminium factory at the Phutka-Pahar mountain and adjacent community of Amarkantak and Korba city. It remains significant to highlight this project here even though the great majority of structures were of steel, and concrete was only applied in the foundations and in the terminal buildings.

5. CONCLUSION

In spite of geographic distances and differences in size of land mass and population, there are diverse connections between India and Hungary. It would be difficult to enumerate all common features in culture and science. We have mentioned a few examples in concrete. The international meetings gave opportunity for engineers from both countries to become better acquainted. There were items of concrete technology that were first introduced into India by Hungarian engineers, such as tunnelling with concrete elements and prefabrication of large reinforced concrete members for industrial buildings.

We hope that the *fib* Congress Mumbai 2014 will be successful for the hosts and for all national groups of *fib*.

6. ACKNOWLEDGEMENT

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7. REFERENCES

Alimchandani, Ch. R., Alimchandani, A. C. (2005): "Beyond contemporary architecture and integration of architecture and engineering", *Keep Concrete Attractive. Proceedings of the fib Symposium,* Budapest, pp. 51-56.

Baktay, E. (1958): "*The art of India* I.-II." (In Hungarian), Képzőművészeti Alap Kiadóvállalata, Budapest

Baktay, E. (translator) (1971): "*Vatsyayana, Kama-Sutra"* (In Hungarian) Medicina Könyvkiadó, Budapest

Baktay, E. (2004) *"Tales and legends of India"* (In Hungarian), Móra Kiadó, Budapest

Baktay, E. (2006) "*The wisdom of India, Sanatana Dharma, The eternal law*" (In Hungarian), Könyvfakasztó Kiadó, Budapest

Baktay, E. (2009): "*Sándor Kőrösi Csoma*" (In Hungarian) Akkord Kiadó, Budapest

Baktay, E., Apostol, A. (editors) Baktay, E. (translator) (1960):"Ramayana and Mahabharata" (In Hungarian), Európa Könyvkiadó, Budapest

Bodai, B. (editor) (1980): "*Tunnelling tubbing elements*" (In Hungarian), BVM Budapest

Crozier, W.F.G. (compiler) (1986): "C. R. Alimchandani", *FIP Notes* 1986/2 p.6.

Crozier, W.F.G. (editor) 1992/3: "FIP Medals", *FIP Notes* p. 19.

Fodor,E. Translated by V. Gáthy) (1976): "*India guidebook* (In Hungarian), Panoráma, Budapest

Fogarasi, Gy. (1986[1]): "*Prestressed concrete Technology"* Akadémiai Kiadó, Budapest

Fogarasi, Gy.(1986[2]): "The moving mould conveyor, flow line pretensioning method", *Procedings of the Tenth Congress of the Fédération Internationale de la Précontrainte,* New Delhi, pp. 397-413.

G. Hajnóczy, R. (1943): "*Fire of Bengal"* (In Hungarian), Singer & Wolfner. Budapest

Garay, L. (1984): "Wagon factory in India (Integral Coach Factory-furnishing unit Madras-Perambur (India)" (In Hungarian), Ipari Építészeti Szemle (Az Iparterv közleményei) 21. pp. 7-15.

Gecsényi, Gy. (1986): "BVM-TIP building system" *Procedings of the Tenth Congress of the Fédération Internationale de la Précontrainte,* New delhi, pp. 383-395.

Germanus, Gy. (1934): "*The light of India (Mahatma Gandhi)*" (In Hungarian), Franklin, Budapest

Gylltoft. K., Derroch, A.S., McQueen, P.J., Tassi, G., Venuti, W.J., White, J.G., Zenobi, G. (1986): "*Concrete railway sleepers. A state-of-the-art report*," FIP Commission on Prefabrication, Working Group on concrete sleepers

Jayadeva (translated by S. Weöres on rough translation of Vekerdi J.) (1982): Gita Govinda (In Hungarian) *Proceedings of the International Conference on Shear, Torsion and Bond in Reinforced and Prestressed Concrete,* Coimbatore, India

Kipling, R.(1894): "The Junghle Book", Macmillan Publisher, London (Hungarian translation by Mikes, L. (1899), Lampel, Budapest

Orbán, Z. (1999): "Rational application of high strength and high performance concrete *5th International Conference on Concrete Technology for Developing Countries". Proceedings*, New Delhi, Vol.1. .IV. pp. 33-41.

Ötvös, Z. (2011): "Reinforced concrete dreams" (In Hungarian) *Népszabadság Magazin*

Rózsa, L. (1979): "Concrete tunnelling using precast elements with bolted joints" (In Hungarian) *Mélyépítéstudományi Szemle* 19.2.

Schmidt, J. (translator) (2010): "Panchatantra" (In Hungarian), Fapadoskönyv Kiadó, Budapest

Sidlovics, J. (1979): "Cableway in Korba India", *UVATERV Műszaki Közlemények,* Vol. 1. pp. 41-43.

Szabó, L. (2013): Swig (In Hungarian) (Leporello) Móra Kiadó, Budapest

Tassi, G. (1999): "Learning from design errors and construction defects of prestressed concrete structures". *5th International Conference on Concrete Technology for Developing Countries. Proceedings,* New Delhi, *Vol. 2.* VII. pp. 43-54.

Tassi, G., Baranyay-Horváth, M. (1969): "A simplified method to investigate the bond stresses at tension release in prestressed pre-tensioned beams", *Proceedings of the International Conference on Shear, Torsion and Bond in Reinforced and Prestressed Concrete,* Coimbatore, India, pp. 101-118.

Tassi, G., Bódi, I., Erdélyi, Ódor, P. (1986): "*Behaviour partially prestressed concrete beams". Tenth International Congress of the FIP,* New Delhi*, Poster Session* Vol.4. (Distributed in two separate pages)

Vámbéry, Á. (1905): "*Fairy Tales of India*" (In Hungarian) Franklin, Budapest

Vekerdi, J. (translator.): (1997) "*Ramayana*" (In Hungarian) Vörös, J., Lakatos, E., Fogarasi, I. (1986): "Rehabilitation of bridge structures by post-tensioning" *Procedings of the Tenth Congress of the Fédération Internationale de la Précontrainte,* New Delhi, pp. 275-285.

Prof. Géza Tassi (1925), Civil Engineer, PhD, D.Sc., FIP medalist, lifetime honorary president of Hungarian Group of *fib,* awarded at the first congress of *fib,* holder of Palotás László award (*fib* Hungarian Group), owner of Diamond Diploma of Budapest University of Technology and Economics (BME), owner of Golden Ring of BME. He is active (semi retired) at the Department of Structural Engineering of BME. Main field of interest: prestressed concrete, bridges and other structures.

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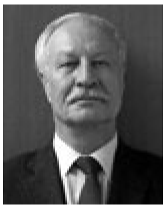
DESIGN OF THE KOSSUTH SQUARE DEEP-LEVEL GARAGE AND VISITOR CENTRE

Gergely HOLU – Csaba PETHŐ

This article presents design and reinforced concrete structural features of a grandiose establishment - an underground garage and a visitor centre aiming at renewing environment of the Parliament, realized as part of a high priority project. We have been involved in the work not only as designer but also within the framework of an on-site supervision by designer during the entire implementation work. Site: Budapest, Hungary.

Keywords: deep-level garage, securing excavation pit with diaphragm wall, monolithic reinforced concrete structure, special solutions applied, white cement

1. GENERAL DESCRIPTION OF THE PROJECT

1.1 HISTORY

Kossuth square accommodates one of the most beautiful and worldwide-known buildings of Hungary, the Parliament. The upgrading of the square aiming at realizing a space fitting with the quality of the building has been for a long time an issue of importance for the Hungarian public life and politics.

Efforts for the renewal of the square have already been made in the past (several concepts were elaborated on this purpose) but in the lack of financial possibilities and in the absence of a strong political will, these remained stuck in the design phase.

The intention of reconstructing the square was presented again to the political decision makers in 2011, as a result of which the National Assembly in a resolution decided about the reconstruction of the environment of the Parliament, Kossuth square (*Fig. 1*).

1.2 MAIN ACTORS INVOLVED IN THE PROJECT

The main tasks to be realized within the frame of the project and the deadline of completing the investment were determined in the resolution, and it was decided that the responsible body of the investment is the Office of the Hungarian National Assembly, as Client.

The specifications and requirements were further detailed in the Steindl Imre Program, elaborated by the Office of the National Assembly, and then set up the program office to coordinate the project. The winning tender of the public procurement was presented by Középülettervező Ltd. (KÖZTI), who involved in the project as design subcontractors UVATERV Ltd, FŐMTERV Ltd and S73 Ltd. The project management and technical supervision tasks were assigned to ÓBUDA-Újlak Ltd, within the frame of the public procurement procedure. The winner of the General Contractor tender was KÉSZ Építő Ltd, the construction works of the deep-level car

Fig. 1: Computer-aided visualization of Kossuth square reconstruction (image is presented with the consent of KÖZTI Ltd.)

park and other works related to construction site delimitation will be completed by Bohn Mélyépítő Ltd, as winner of the tender.

1.3 DESIGN TASKS

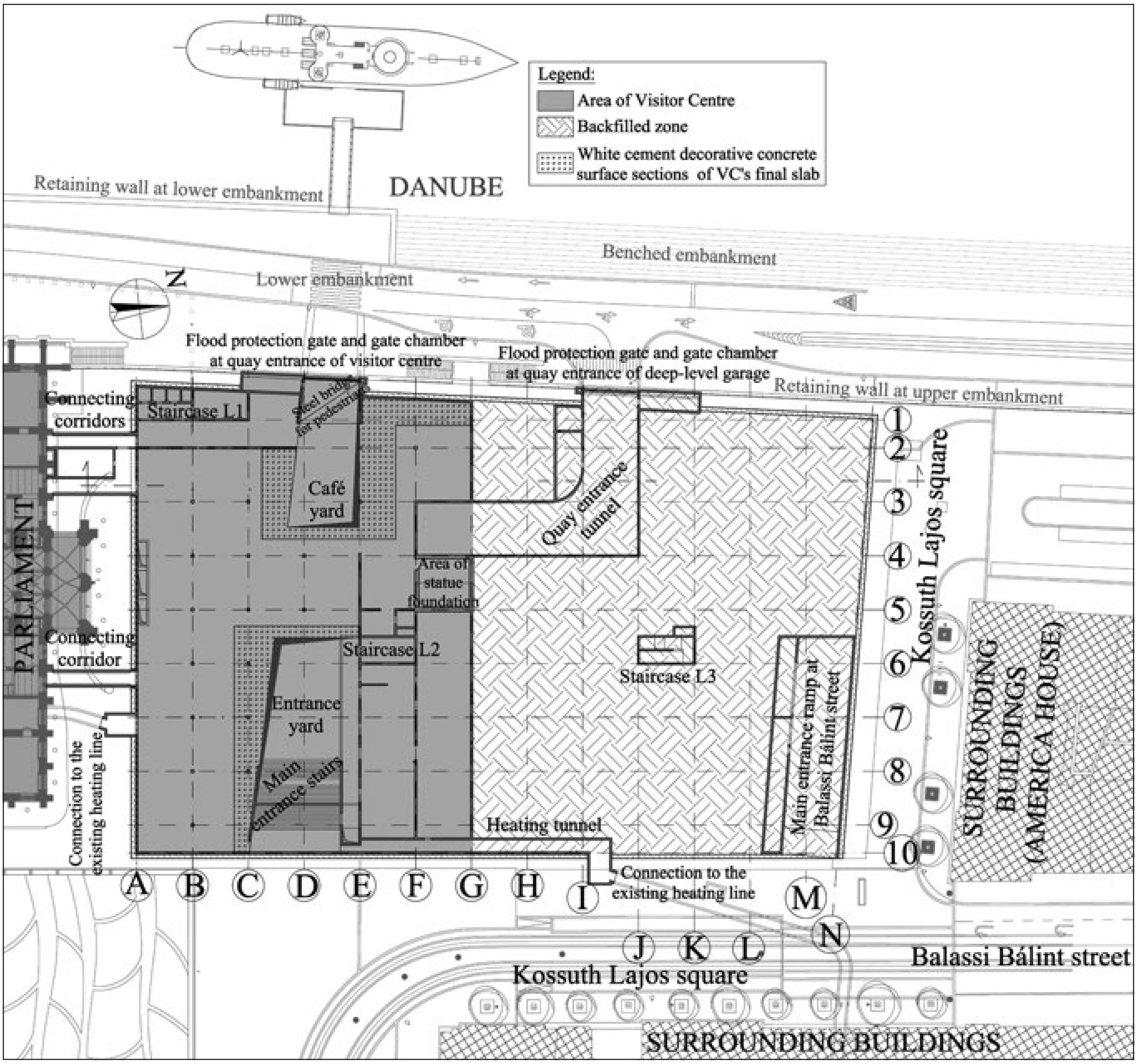
The task of the co-designers under the management of KÖZTI General Designer was to carry out the full range design activity related to the square landscape design, construction of the deep-level garage and visitor centre and the Parliament Museum, and other related work parts. The aim of the present article is to describe the structure of the reinforced concrete deep-level garage and visitor centre, and the design tasks related to structural engineering. The entire structural design of the facility was elaborated by the experts of UVATERV Engineering Consultants (Department of Metro Design and Structural Engineering).

2. GENERAL PRESENTATION OF THE DEEP-LEVEL GARAGE AND VISITOR CENTRE

2.1 LOCATION

The deep-level garage and visitor centre is located on the north side of the square, in the direct vicinity of the Parliament building. At the west side the facility is separated from the Danube floodplain only by the upper quay retaining wall. The north side of the facility is separated by a street from a six-storey building called America House. At the east side there is a green surface and pedestrian walkways, tram and car traffic streets and buildings. The 110 years old Parliament, which is located at the south side, has a basement of a two-level cellar. The huge building stands on a special plate foundation. The concrete foundation of variable thickness which, under the dome reaches 4,0 m, is a "cyclopean" layer realized with the technology of the epoch of construction, laid in several layers and filled with cement mortar mixed with granular aggregate, ensuring the transmission of the building loads to the load bearing subsoil. At the previous soil investigations, the

Fig. 2: Layout after reconstruction with the ground plan of level -1



quality of the different layers showed significant differences of strength. The public utilities disturbing the construction were demolished, reconstructed or treated with special attention. Perhaps it is not an exaggeration to say that the surrounding natural and built environment created special site conditions, resulting in difficult design requirements but a nice task at the same time. These were mainly the preservation of the Parliament building which is a historic monument, and the requirements related to flood risks due to the proximity of the Danube (*Fig. 2)*.

2.2 GENERAL DATA

On the upper level, basically on the half of the total area of the structure, a sophisticated-presence visitor centre was realized. This facility is connected with a corridor to the Parliament and to the Parliament Museum, which is constructed under the building (the structural design of the latter two facilities was elaborated by KÖZTI). The outdoor connections of the visitor centre are ensured from Kossuth square by the entrance stairs and elevator, and from a pedestrian entrance on the upper quay. On this same level, next to the visitor centre, the closing slab of the deep-level garage is provided with a backfill of 3,6 m, which will serve for future purposes of landscaping of the concerned area. The main entrance of the car park is situated at the Balassi Bálint street side, but driving in possibility is ensured also from the upper quay, where an entrance tunnel leads to the ramp of the deep-level car park. The additional levels can have access through ramps installed under the main entrance. The deep-level garage function is offered on 3 levels; levels -2, -3 and -4 simultaneously offer the capacity for 592 cars, 10 motorcycles and 30 bicycles. Clearance height of level -2 is raised for allowing van traffic, the remaining two levels offer access only for cars. Pedestrian traffic inside the car park is served by three staircases and by elevators installed next to staircase L1. Main inner dimensions of the structure: 72x111m.

The facility had a further important design criterion: the flood protection of the entrances of the visitor centre and the quay entrance of the deep-level car park. Considering the high risk values both places had to be provided with double protection lines: the outer line is a rail guided steel structured flood protection gate of special design, the other is a mobile flood protection wall, which is a finished product. Another important consideration was the static fine-tuning of the foundation zone and the supporting structure bearing the significant dead weight (1.300 metric tons) of the statue to be realized on the surface. The final reconstruction of the demolished district heating line section was also an aspect of great importance, a puzzling task due to the uncertainty of the connection points (*Fig. 3)*.

3. STRUCTURAL ARRANGEMENT

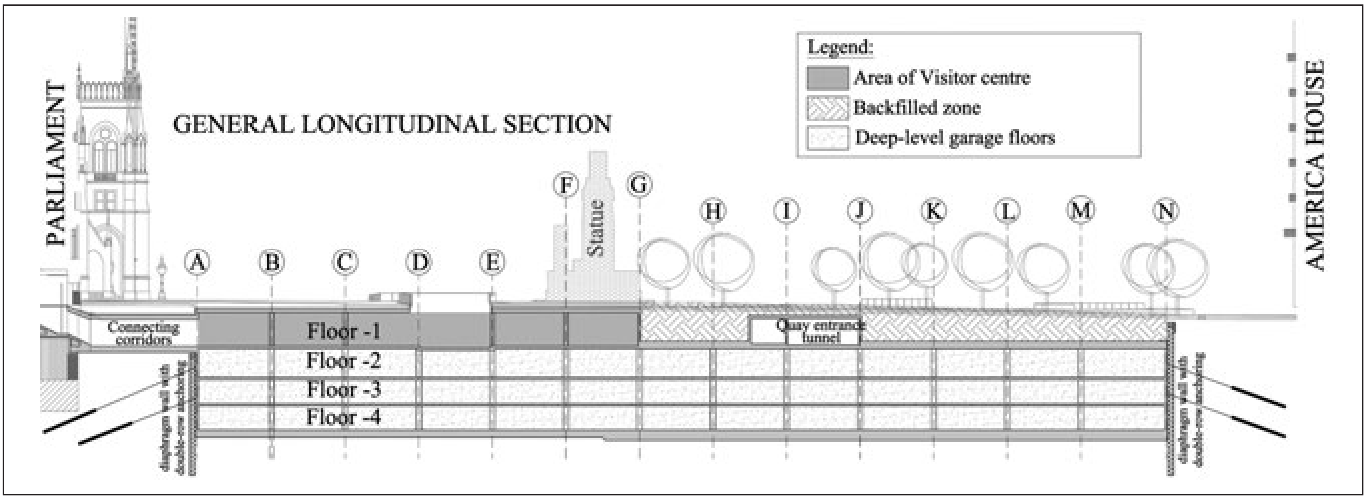
3.1 DELIMITATION OF CONSTRUCTION PIT AND RECORD FLOODING LEVEL ON THE DANUBE

According to the soil mechanical report the main characteristics of soil conditions are as follows: the upper layer consists of variable thick and solidity backfill, typically sandy gravel, in some places brick bat of organic materials content. Beneath the backfilling the load supporting layer is compact gravel, sand-sandy gravel. The base rock consists of Oligocene Kiscelli clay. The load bearing capacity of the soil is important, while the permeability capacity is low.

The earthwork surface of garage is determined at approx. 15,50-14,50 m under the existing surface level. Considering that the groundwater level is located significantly higher and the permeability capacity of upper soil layer is important, the construction cannot be realized but with watertight delimitation, therefore a watertight diaphragm wall is to be designed.

The dimensioning of diaphragm wall was realized by using the geotechnical software Plaxis 8 according to the typical cross sections. On the northern, western and eastern sides sizing of one cross section for each was sufficient due to the similar soil structural and surface characteristics. On the diaphragm wall section located on the Parliament side three different cross sections can be found due to the difference between the Parliament building and the structure being constructed. At the load calculations we considered the buildings in the concerned area by a steady loading force exerted on the foundation level, the technological loading of the surface and – for having an economical structure – the water pressure of middle water level of Danube of a 10 years period. At the dimensioning an important attention was given to minimize the deformation of the diaphragm wall and to respect the limit value of 0,2mm for the crack width. The delimitation of the construction was realized by a diaphragm wall of 60 cm thickness with a double-row anchoring, in the angles multi-row steel pipe propping was applied. We designed the diaphragm wall with 4,50 m overlapping beyond the earthwork plane. Due to the

Fig. 3: General longitudinal section



importance of the effective watertightness a strict requirement was that the diaphragm wall can be installed everywhere into intact clay – constituting the case rock – at a depth of 3,0 m at least (*Fig. 4*).

We defined the working level of the diaphragm wall at the design water level, 104,53mBf. ensuring the appropriate alignment of waterproofing against the groundwater pressure of the structure. The diaphragm wall was constructed from watertight concrete provided with joint tapes for a better watertightness. The earthwork was realized in three phases according to the anchoring work. The upper anchor line started above the construction water level, while the anchor body is significantly deeper than the design water level. The bottom anchors were realized under water with appropriate technology applied. The distressing of the anchors was approved phase by phase according to the required completion level of the internal floor slabs. After the load release the crossings were to be provided with watertight closure (*Fig. 5*).

In order to monitoring the motion of the structure we designed installation of 2-2 inclinometer pipes into the approx. third part of the wall on all four sides. The monitoring was to be realized in proper frequency according to the construction phases.

Since there is no appropriate loadable mass above the garage, groundwater level regulation is applied to prevent from uplift. A draining system embedded into a sandy-gravel layer under the base slab of the garage ensures the drainage of the water infiltrating through the soil. Between the diaphragm and inner lining wall the water is drained to the lower levels by superficial drain system. The water collected in wells is pumped by pump facilities into the water drainage system or it is used for the operation of the building. The watertight concrete layer of base slab ensures the appropriate watertightness.

According to the Client's requirements we had to prepare – in case of diaphragm wall construction area delimitation technology – a uniquely detailed risk analysis determining the measures to be taken in function of – among other factors – Danube water level and the progress of construction. Due to the varied environment, an important part of the factors presented in the risk analysis manifested effectively. Everyday problems emerged concerning anchoring, diaphragm stability, unknown public utility, ammunition, etc. In an optimal case the flood risk did not involved but an accurate measurement and observation. In the most critical cases – important earth excavation and high water level – we also had to consider the artificial flooding during the operation of temporary diaphragm wall. The critical limit was determined in a water level reaching the lower embankment during the earthwork up to the foundation level. The preparedness levels indicated by flood risk analysis were determined in function of overloading allowed and tolerable displacements. The different degrees – according to displacement measurements – were "alert", "preparedness" and "emergency". The designed stress capacities ensured the safety requirements. In case of the anchor stresses the safety was tested at the qualification of each anchor. In case of flood, as extraordinary load, the safety requirement approached the basic value of 1,0. The necessary measures to be taken were principally determined in order to limit the deformations to an optimal level. By the increasing of monitoring up to a daily basis the evaluation became a real time process.

The extraordinary flood of 2013 required a special approach of all design. The measures required in frame of the risk analysis were to be realized according to the prognostic of end of May (*Fig. 6*). Only a short week was given for the conscious realization between the typical and the emergency case including the four days necessary for the backfilling. The backfilling work was completed only just few hours before the total flooding of the embankment. Due to the accurate preparations and the construction of quality according the design, the record height flood – exceeding by 40 cm the maximal level ever observed – had subsided without any definitive damage however involving a delay of 20 days. The pressure measurements realized by the monitoring system ensured information not only for the momentary safety measures but for solving later, similar situations and correcting design parameters (*Fig. 7*).

Fig. 4: *Construction pit opened up to the base slab on the Parliament side. Angle pipe carriers, first phase steel construction on 9th May 2013*

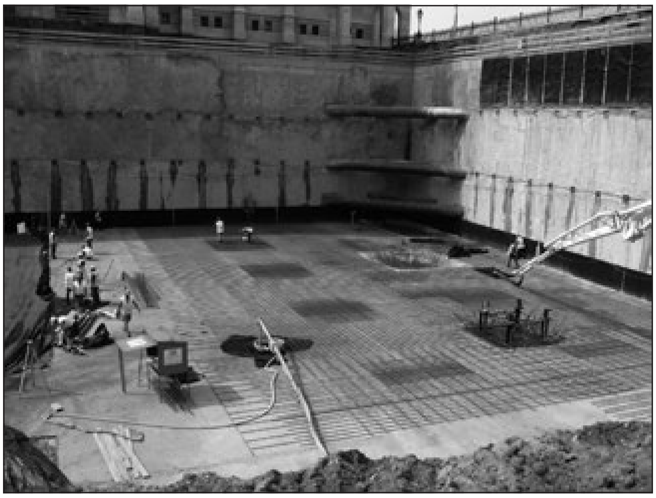


Fig. 5: *Earth excavation up to the 2nd anchor level on the Danube side, boring of underwater anchors on 18th April 2013*

Fig. 6: *Base slab completion level before flooding, from Balassi Bálint Street on 30th May 2013*

Fig. 7: Artificial flooding measures taken in the construction pit, with a record outside flood on 10th June 2013

Fig. 8: An intermediate state of construction on 13th August 2013

3.2 THE DEEP-LEVEL GARAGE CONSTRUCTION

The deep-level garage has been built in an open excavation pit, heading upwards from the bottom, with permanent surface water evacuation, and a conventional reinforced concrete structure. The inner lining wall and base slab have not been tied to the diaphragm wall. Around the external perimeter structures, beds allowing for drainage (a surface drain between the diaphragm wall and the inner lining wall, and a sand gravel drain bed beneath the base slab, complete with an appropriate drainage network) have been installed. They are used to deliver infiltrated water into drainage wells built into the base slab. From these well, water is removed by pumping. So, no water pressure on the deep-level garage structures will build up, and no upfloat had to be reckoned with. In order to ensure an internal water drainage, both the base slab and the intermediate slabs have been built to have a crossfall of 0.5% created from two summits led all along the engineering structure. At the deepest points, small depth gutters have been foreseen. Inner water is evacuated into collectors built in the base slab from where they can be removed by pumping as necessary.

The base slab has been made as an impermeable reinforced concrete structure, in two major structural thicknesses – a thickness of 85 cm at the part beneath visitor centre, and 120 cm at the parts concerned by the deep-level garage with backfill and by the statue foundation – adjusted to loads transferred from above. Staircases and sumps have a 60 cm thick bottom plate.

Reinforced concrete columns and wall structures, the mixture of which forms a vertical supporting structure framework, have been built floor by floor by using starter bars out of the base slab *(Fig. 8)*. In the general field with intermediate support, the elongated round piers typically with a cross section of 60/110 cm have been arranged in a raster of 8.4 x 8.1 m. At the part affected by the statue foundation, it was necessary to increase pier cross section and to install piers at one corner of staircase L2. In staircases, near the main ramps and at the edges, vertical loads walls are borne by walls with a thickness of 30 cm running all along in full height. In addition, the wall near main ramps running parallel to the diaphragm wall is loaded not only in its plain but it also carries horizontal loads transferred from ramps and ensuring a final support for the excavation pit. Some wall sections have been reckoned with as a wall bearer.

The horizontal load bearing structures are – depending on the load imposed on them – reinforced concrete slabs with different thicknesses which are also used as a final supporting structure for the diaphragm wall. The thickness is 25-30 cm for car park floors, and 85 cm for the final slab having an earth backfill.

Fig. 9: Demolishing diaphragm wall and upper retaining wall at the entrance to Visitor Centre and deep-level garage on 5th September 2013

At some locations, beams are used to transfer load onto piers. Walls and slab structure of embankment tunnel starting from the final slab form a box type framework. At the tunnel exit towards the embankment, and the affected parts of diaphragm wall and existing retaining wall at the upper embankment had to be demolished in order to allow the newly built reinforced concrete structure of deep-level garage exit and gate chamber to be built *(Fig. 9)*. A cap beam has been installed on the demolished diaphragm wall. The demolished portion of retaining wall had to be restored by connecting it to the new structure. Waterproofing concrete mass insulation and surface insulation are connected on the cap beam. Considering that the entrance is found beneath flood level, a double flood control gate system including a movable slide gate and a mobile mounted wall has been installed. Each flood control system component had to be concreted into the structure in a watertight way, with strict tolerances.

In the structural calculations, each structure has been tested through modeling on a floor by floor basis, using AxisVM 10 structural analysis & design software. For the final slab supported with piers and the intermediate slabs, spot-like and surface supported models have also been developed. The former has been used to calculate bearer reactions and deformations, whereas the latter to perform surface steel reinforcement and crack width tests. Bearers have been taken into consideration with a rigidity calculated from their restraint conditions, network length and cross sections. Support conditions have always been reckoned with according to a vertical supporting structure framework, and for the base slab, a continuous surface support has been used for modeling the load bearing base. Dead loads and payloads have been used to load structural models. For the former, the own weight of structures and the weight of pavement courses overlying them have been reckoned with whereas for the latter, loads imposed by vehicles have been considered as governing combinations of loads. Forces arising in vertical supporting structures have been calculated floor by floor, then, by summing them separately for each type of load, the base slab has been loaded. Pier rating has been calculated floor by floor, dividing them into groups depending on the type of mechanical stress. The concrete applied is of C30/37 strength class, with conventional grey cement dosing. Each rating has been calculated according to latest Eurocode standards.

3.3 VISITOR CENTRE STRUCTURE

Visitor centre is found on floor -1, between axes A and G, and is directly connected to the deep-level garage structure. Its floor slab typically has a thickness of 30 cm but at locations where it is not sufficient due to a higher load – for instance, beneath power electric machinery rooms – the thickness has been increased to 35 cm. The walking level of slab is flushed but at several locations a step in level had to be realized due to the different pavement courses. For instance, the entrance and café yard areas are lowered by 20 cm with respect to the general slab level due to waterproofing and thermal insulation. On the Eastern and Western sides, a reinforced concrete structure that supports the main entrance stairs and the steel bridge for pedestrians will transfer their loads onto the slab. Here, a system of beams had to be applied in order to impose the loads transferred, onto piers.

Supporting the final slab of visitor centre is generally provided by reinforced concrete piers with a diameter of 50 cm, adjusted to 8.40 x 8.10 m raster of the deep-level garage. At locations where it has not been feasible, reinforced concrete walls have been built a part of which is a continued wall structure of staircase and inner lining walls on floor -1, and the other part is a single-floor wall bearer built on that floor only. Internal height varies in the range ~3.70 to 4.15 m. Walls generally have a thickness of 30 cm but at locations exposed to a higher load, they have an increased cross section: the wall bearers beneath statue foundation and the walls supporting the steel bridge have a thickness of 60 cm, whereas the structural wall closing the visitor centre and exposed to a ground pressure arising from backfill is 35 cm thick.

There is just room enough for the minimum course thickness pavement above the waterproofing and thermal insulation layers installed on the final slab, therefore, the slab is designed to have a slope of 1.5% transversally, following terrain conditions of a new surface to be realized. Typically, it has a thickness of 50 centimeters but at some locations – adjusted to the load – a locally different thickness has been made.

The East-West oriented main corridor of visitor centre is roofed in part only: on the Eastern and Western sides, there is no slab above the area concerned by the descending stairs and the steel bridge.

The slab portion and wall system supporting the statue is connected to the final slab of visitor centre. The weight of the planned statue featuring a heavy weight (the statue including its supporting structure are out of the scope of a sectoral design by UVATERV) will be transferred as a distributed load, through a base slab, to the reinforced concrete slab of final ceiling slab. At the statue foundation, the supporting system for visitor centre and deep-level garage has been adjusted to it, by means of structural solutions already presented.

There are three connections built from the visitor centre to the Parliament: a combined corridor of entrance for members of Parliament and visitors, an exit corridor for visitors and a tunnel for public utilities channel. On the side towards river Danube, there is an entrance for pedestrians from the embankment, whose flood control and insulation connections are the same as those of the deep-level garage.

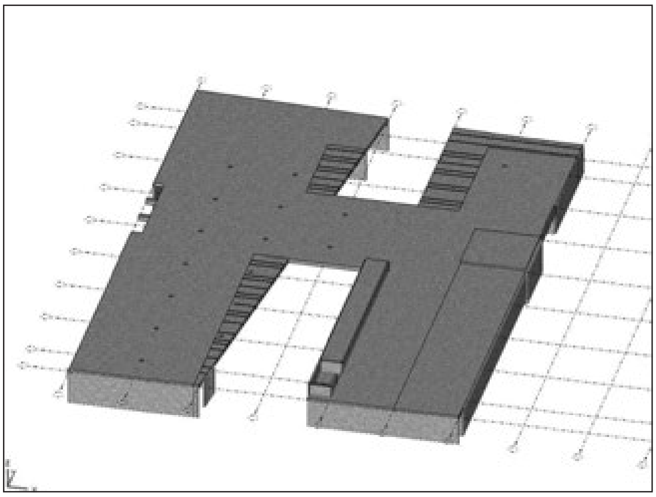


Fig. 10: A mass model for Visitor Centre final slab

The section of public utilities tunnel for providing the Parliament with heating, crossing the engineering structure has been demolished and aligned in a new route adjusted to the structure.

The structural engineering modeling and calculation of structures have been performed in the same way as described in the previous chapter. As for material quality, strength grade C35/45 has appeared. In addition, several structural elements have been made with white cement dosing, in decorative concrete quality making crack width requirements even stricter *(Fig. 10)*.

3.4 SPECIAL SOLUTIONS APPLIED FOR REINFORCED CONCRETE STRUCTURES

Considering that the design and implementation schedule has been extremely tight, even during the design we have striven for applying state-of-the-art technologies and solutions allowing for an acceleration of the pace of implementation, offering a technically appropriate solution and providing cost-effectiveness at the same time.

a) Using finished items and products from construction industry

During the design, finished items and products from construction industry not so widely used in the construction industry practice in Hungary have been used consciously. For the major part, they are not absolutely necessary, with respect to the conventional solutions, in a standard construction environment due to their extra price but in our case their application has been by far economic for the contractor, due to the tight schedule of implementation and to the strict time limits subject to penalties. The products applied have greatly contributed to a rapid, simple and easy-to-overview steel reinforcement installation, thereby reducing manpower demand and time. In addition, within groups of products – in the case where some parameters match – some product are interchangeable with each other, so in certain cases, a "waste" otherwise realized as a loss could also be utilized with a consent thereto from the designer.

For plain plate slabs, the great span, high load and the restricted crack width have resulted in a dense steel reinforcement at pier heads steel reinforced using standard and reinforcing steel reinforcement. Installing the dimensioned puncture type steel

Fig. 11: Puncture type steel reinforcement with radial arrangement in an intermediate slab on 11th July 2013

Fig. 12: Built-in, composite action pins with clamping edge in the main entrance ramp on 2nd December 2013

reinforcements would have required a great amount of manpower and time if using conventional elements. Realizing puncture type reinforcements by using factory-made products *(Fig. 11)* has greatly contributed to a rapid and convenient work.

At wall and slab connections not withstanding torque, the "zip-fastener" type reinforcement that can be installed into a formwork has become standard by now, forming a basis for large panel technology. Connections of this kind have also been widely used in this work. In addition, a special version thereof is represented by the so-called composite action anchor hook with clamping edge that connects the structurally dimensioned supporting structure and the concrete slab concreted onto it subsequently in such a way that the waterproofing to be made between both concrete layers can be led through the bars in a watertight manner *(Fig. 12)*.

Chair distance pieces as standard for slabs have only been used for slabs with great thickness, whereas the steel reinforcement for inner slabs could be accelerated by installing prefab corrugated distance pieces. For an enhanced watertightness, optimum components have been foreseen in each case to seal working joints: starting from surface or inner gap tapes to swelling and/or injectable ones. Concrete placement working joints have also been made using a finished product from construction industry: "streckmetall" (ribbed expanded sheet) tapes have been installed perpendicular to the slab plain in order to obtain a possible fastest and simplest design of working joints.

Armature type installation of steel reinforcement of piers – although they do not represent a product – can also be assigned here. Pier steel reinforcement has been at factory assembled, and clevises fixed using spot-welding on the main steel reinforcement. So, a complete pier armature has been delivered to site. All we had to do was to position it to the right place by means of a crane.

b) Using precast reinforced concrete pieces

The entire underground garage structure can be basically built in an efficient way by using a monolithic technology. For large slab and wall surfaces, it was not justified to use precast elements. However, in order to accelerate the work, we had to do our best, so, for smaller structural units – if allowed by the structural framework as well – it had to be checked for which elements it would be recommended to use precast structural elements. As a result thereof, flights of stairs have been foreseen to be made from precast reinforced concrete units connected through connecting steel reinforcement pieces to monolithic landings concreted simultaneously with them *(Fig. 13)*.

c) Working joints

For working joint, the primary aspect was to reduce concrete placement stages to a minimum (that is, to maximize the size of panels placed into concrete at the same stage), and to optimize assembly of reinforcement steel pieces for each stage so that the subsequent stage can be concreted as soon as possible. The upper limit of concrete placement limits have been jointly determined by the maximum panel size that can be concreted at the same stage (<35x35 m) and by the amount of concrete that can be worked in on the maximum (<1000 m^3). Another condition was represented by a time-limit for concreting opportunity for adjacent panels (according to concrete placement technology, at least 4 days must be provided between the time at which two adjacent panels are concreted), and by selecting the place of working joints adjusted to stresses.

d) Shuttering

For shuttering, the construction meant no particularity. For the major part, routine professional elements have been used. Slabs and walls were shuttered using large panel elements, with the shortest turning time due to the tight schedule. For poles unified in size, the sample shutters could be moved simply. When shuttering a slab, it was a special demand to ensure a temporary support due to the final slab having a great thickness, which remained in place at lower floors as a trimming until the slab has hardened. Here, the use of Peri Skydeck slab shutter system offered advantages, allowing for a recovery of formwork panels with the supporting columns left in place.

Decorative concrete columns at visitor centre are made of white concrete. For an aesthetic appearance of round piers, impregnated paper tube has been used as a formwork whose internal coat could ensure a smooth surface without joint.

For the decorative concrete slabs, even the new factory-made

Fig. 13: Built-in and positioned precast flights of stairs on 19th September 2013

panels were not smooth enough, therefore, a layer with tight joint has been fastened separately onto the plank formwork.

3.5 WHITE CEMENT DECORATIVE CONCRETE SURFACES

Specified walls of the visitor centre, round piers with a diameter of Ø50 cm and some sections of the final slab are designed to have a white cement decorative concrete surface. Due to its shrinking features, the concrete made of white cement only allows for a concrete placement of large surface slab portions in a limited way which has caused a problem requiring a structural intervention mainly for the final slab where – at the sections concerned – a horizontal working joint had to be used to divide slab cross section. The lower decorative concrete crust has been made with white cement dosing, enhanced crack-distributing steel reinforcement and plastic fiber dosing. The upper conventional reinforced concrete bed has been connected to the lower surface through a connecting steel reinforcement. Due to a properly small cross section thickness of the white cement cross sectional part, no harmful shrinking typical of that material could develop, and the high quality surface could be completed without any crack *(Fig. 14)*. The connecting steel reinforcement for complex cross section has been sized for a sliding force according to principles applied for composite structures *(Fig. 15)*. The concrete for decorative concrete surfaces has been made using a maximum additive grain size of 8 which is significantly smaller than the usual grain size.

Fig. 14: Visitor Centre final slab - conventional grey cement surface and white cement surface of decorative concrete quality even before cleaning on 2nd December 2013

Fig. 15: Crustal concrete poured with white cement decorative concrete and a more dense distance piece steel reinforcement applied at general section to realize composite action of cross section on 5th November 2013

The white cement decorative concrete surfaces have also forced the contractor to apply special solutions. Of them, it is noteworthy as a peculiarity that the steel reinforcement installed into the crust concrete has – in a completely unique manner – not been seated on the formwork but suspended from a temporary supporting structure made from shuttering beams above the white cement field. The reason why a “floating” steel assembly has been worked out was to ensure that even the distance pieces of concrete cover should not appear on the decorative concrete surface having high aesthetic demand.

4. SUMMARY AND CONCLUSIONS

The underground garage – visitor centre complex at Kossuth tér in Budapest seems to be a simple underground reinforced steel structure. Nevertheless, due to its complexity and highlighted central situation it has had a couple of challenges. The short deadlines for design and construction work could have not been realized without an enthusiasm and high level professionalism of those involved, a proper management and control of the project and a continuous on-line data flow between participants. Due to the short deadline for implementation, it was necessary to foresee every up-to-date component that allowed for an industrial implementation despite custom-designed constructions. Civil engineering requirements have demanded high quality as a basic expectation. Structures have had to comply, with the same extent as that of load bearing capacity, with high aesthetic requirements as well. In practice, every structure had to meet a quality met by decorative concrete. The only difference was in the degrees thereof. Using white cement to a great extent – due to the known sensitivity of that material – has forced to work out customized technologies for steel reinforcement and concrete placement.

With respect to the basic challenges, even the excitements during a 20-day relaxation period when the site was flooded due to a high water level of river Danube never experienced before are not noteworthy.

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FIRE DAMAGED RC STRUCTURES – NON DESTRUCTIVE TESTING POSSIBILITIES


Éva Lublóy – György L. Balázs

Recent fire cases indicated again the importance of fire research. Initiation and development of fire are strongly influenced by the choice of construction materials. In addition to their mechanical properties, their behaviour in elevated temperature is also of high importance (Janson, Boström, 2004). Residual compressive strength of concrete exposed to high temperatures is influenced by the following factors (Thielen, 1994): water to cement ratio, cement to aggregate ratio, type of aggregate, water content of concrete before exposing it to high temperatures and the fire process. Therefore, mix design and composition of concrete is of high importance for high temperatures.

Fire can cause damage in concrete structures. The level of damage depends on several factors like maximal temperature, duration of fire, constituents of concrete etc. We have to design buildings for fire and deal with the possible reconstruction after a fire case if it happens. Both require special knowledge and special characterization. For reconstruction purposes it is very important to determine the level of damage and the necessary steps after fire. If the building can be renovated, non-destructive test methods are of high importance in analysis of actual damages. For the analysis of a practical case several core samples were taken of a 2-hour hydro-carbon fire loaded structural element and CT investigations were carried out. Considerable reductions of measured Hounsfield-values were observed close to the concrete surface subjected to fire. This region coincides well with the region where the photographic picture shows light pink colour of concrete instead of grey.

Keywords: concrete structures, fire damage, non-destructive testing, Computer Tomography (CT)

1. INTRODUCTION

Concrete has excellent properties in regards of fire resistance compared with other materials. *(Khoury, Grainger, Sulivan, 1985)*.

Effects of high temperatures on the mechanical properties of concrete have been investigated as early as the 1940s (Schneider, 1988). 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 and physical 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: (1) local damage in the material itself (2) and global damage resulting in the failure of the elements.

Recent fire cases indicated again the importance of fire research. In the night of 12 February 2005, a fire started in the Windsor building in Madrid, Spain, a 32-story tower framed in steel-reinforced concrete. The fire burned for almost a day. The observation that the Windsor Building is the only skyscraper to have suffered even a partial collapse as a result of fire suggests that the use of steel-reinforced-concrete framing was responsible. A closer look at the incident shows the reality to be more complex. The portion of the building that collapsed consisted of the outer portions of floor slabs and perimeter walls throughout the upper third of the building (the 21st through 32nd floors). The outer walls consisted of steel box columns arranged on 1.8 meter centers and connected by narrow spandrel plates. The columns had square cross-sections 120mm on each side, and were fabricated of 7mm thick C-sections welded together (http://911research.wtc7.net/wtc/analysis/compare/windsor.html).

The most recent example of a spectacular skyscraper fire was the burning of the Hotel Mandarin Oriental starting on February 9, 2009. The nearly completed 158.5 m skyscraper in Beijing caught fire around 8:00 pm, was engulfed within 20 minutes, and burned for at least 3 hours until midnight. Despite the fact that the fire extended across all of the floors for a period of time and burned out of control for hours, no large portion of the structure collapsed (http://911research.wtc7.net/wtc/analysis/compare/fires.html).

Not only the inflammable materials were destroyed or damaged by the fire of 15 December 1999, but the load bearing structure suffered also a permanent deformation and a structural damage in the materials of the Budapest Sport Hall in Hungary.

The structural system of the Sport Hall was mainly composed of two main parts: the circularly positioned rows of pillars bearing the purely steel structure cable suspension roof and the steelwork supporting the upper floors of the ring-shaped building, moreover the reinforced concrete

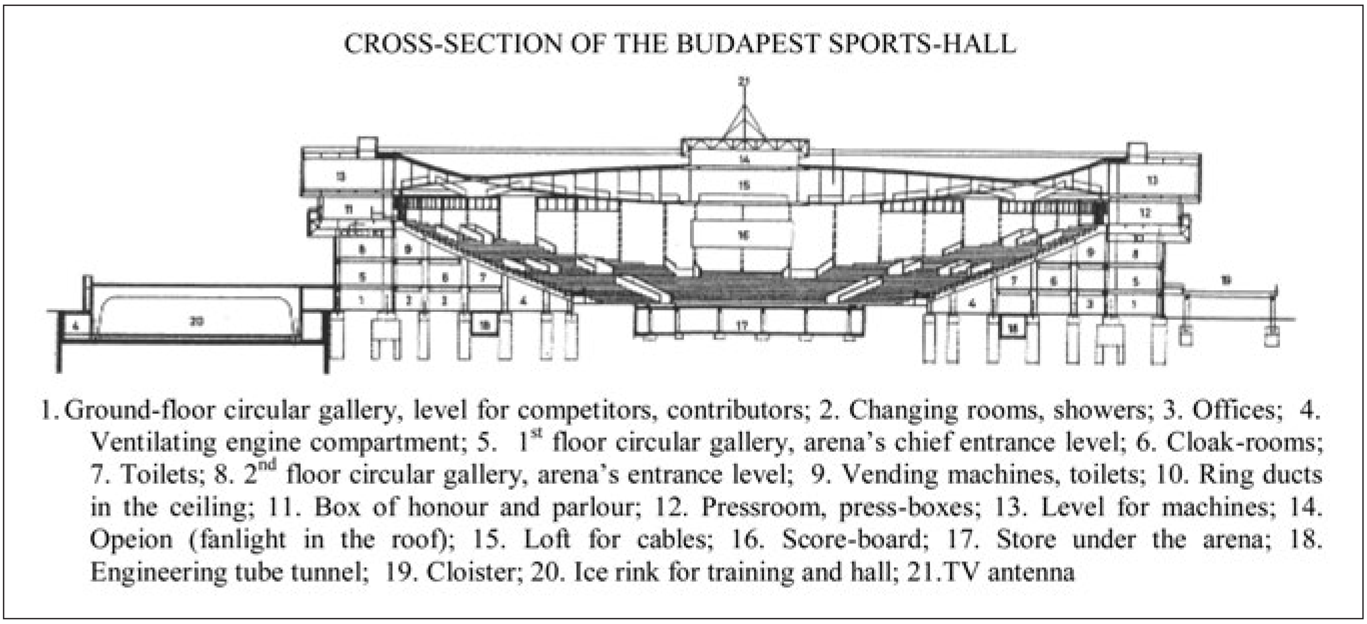


Fig. 1: *The structure of the Budapest Sports Hall (Kiss, 1986)*

structure grandstand encircling the arena, as well as the connected 1st and 2nd floor ceiling-elements. In general, the bearing structural elements of the building were of reinforced concrete *(Fig. 1)*.

The intensity of the fire can be judged from its long duration (3 hours), and from the extent of the damage caused to the different structures. On the basis of the damages caused by this particular fire case, the general conclusion may be drawn that the large part of the structures was exposed to a long duration of high temperatures (800 to 900 °C, or incidentally even higher).

The most intensive fire effect was experienced at the arena's roof, which tumbled thereupon. Presumable reason of tumbling was the relatively fast melting of the lead cast at the cable heads *(Fig. 2)*. In case of appropriate technological solution the tumbling would probably have happened much later.

2. CHEMICAL TRANSFORMATIONS OF CONCRETE

Fire can cause damage in concrete structures too. The level of damage depends on several factors like maximal temperature, duration of fire, constituents of concrete etc. We have to design buildings for fire and deal with the possible reconstruction after a fire case if it happens.

Fig. 2: *Destroyed cable heads (ÉMI, 2000)*

Concrete is a composite material that consists of 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. 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 ($3CaO\cdot Al_2O_3\cdot 3CaSO_4\cdot 32H_2O$) 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, *Weiβ*, 1977).

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, Weiβ, 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, 1973). This transformation is followed by 5.7% volumetric increase.

Dehydration of calcium-silicate-hydrates was found at the temperature of 700 °C (Hinrichsmeyer, 1989, *Fig. 3*).

3. RESIDUAL COMPRESSIVE STRENGTH OF CONCRETE

Temperature influences are reflected in changes of physical and mechanical properties of concrete.

Residual compressive strength of concrete exposed to high temperatures is influenced by the following factors *(Thielen, 1994)*:

(1) water to cement ratio,
(2) cement to aggregate ratio,
(3) type of aggregate,
(4) type of cement,
(5) water content of concrete before exposing it to high temperatures and
(6) fire process.

The stress-strain relationship characterises the stresses and deformation capacities of fire exposed concrete (*Fig. 3*). In *Fig. 3* it is observed increasing strain, a decrease of stress is after the peak stress.

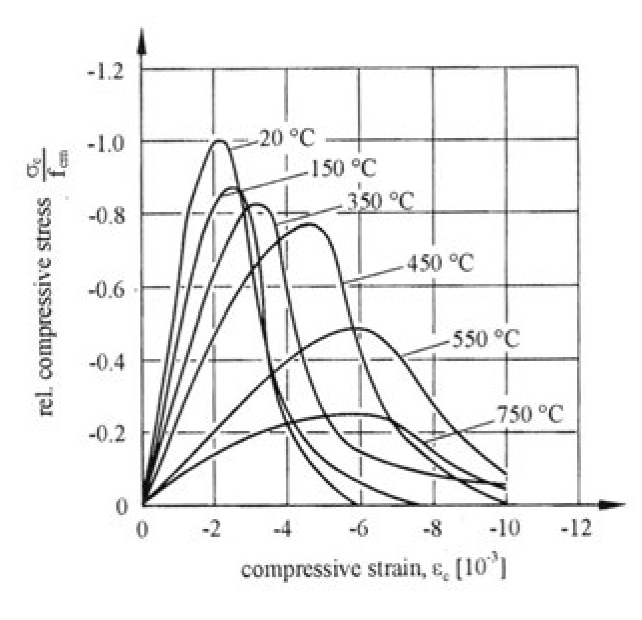


Fig. 3: Stress-strain relationship for concrete with quartz gravel aggregate as a function of temperature (Schneider, Lebeda 2000)

In *Fig. 4* are the density and porosity values in function of temperature demonstrated. The porosity and true density values increased and the bulk density decreased in function of temperature.

4. X-RAY TESTING

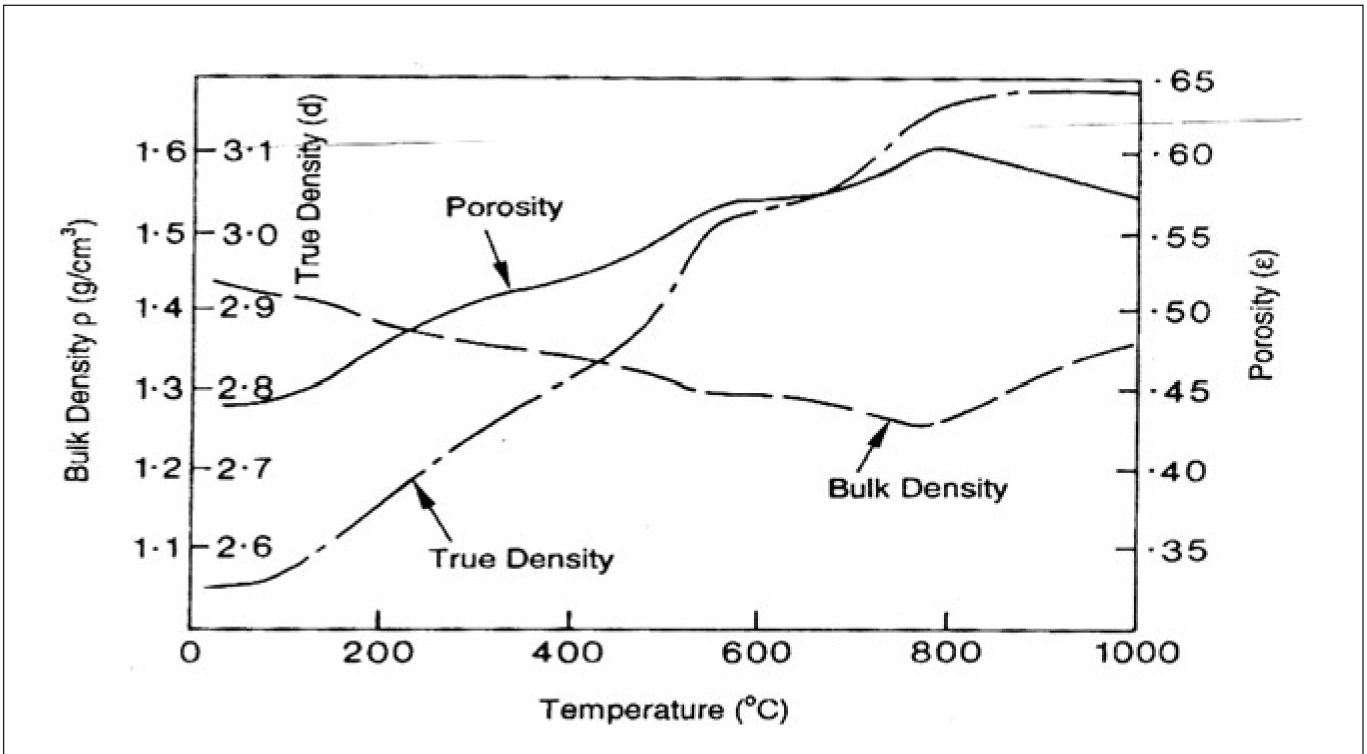
As a new, non-destructive analysis method, the Computer Tomography (CT) technique originally was used for medical analysis. During the time of data processing the quality of the picture depends on many factors. The displayed image is calculated from electrical signals by the imager, so it is not a real picture like a traditional photograph or a radiogram. The theoretical foundation of the CT measuring technique was made by Hounsfield and Cormack in the '70s (Bogner, Földes, Závoda, Repa, 2003).

X-ray is weakening as it goes through different materials and textures. The degree of absorption is smaller or higher in materials of different densities, therefore, it depends on the attributes of the measured materials. The capability of X-ray absorption can be characterized by the coefficient of X-ray absorption. If the transfer of energy is constant, the absorption of X-ray depends only on the material through which it goes. This degraded radiation reaching the detectors generates electrical signals dependent on the intensity of radiation.

As the system of tube detector turns around the analysed object, during the time of data collection hundreds or thousands of measurements are made by the CT while the incoming data is organized into a matrix. At the end of the process the imager calculates each element of the matrix and assigns a scale to the points of the matrix whose points are actually the coefficients of X-ray absorption. This scale is the so called Hounsfield scale, its unit is the Houndsfield unit. (Nobel Prize was awarded jointly to Alan M. Cormack and Sir Godfrey N. Hounsfield for the development of computer assisted tomography in 1979.) Assigning the different values of the matrix to the appropriate values of the Hounsfield-scale the image can be displayed. We can visualize the image using predefined colour tables or our own colour ones (Földes, Kiss, Árgyelán, Bogner, Repa, Hips, 2004).

The experiments were made with Simens Sensation CT with multislice technique. The resolution of a matrix depends on several factors. Using the best resolution of our CT the smallest size of a cell of a slice can be 0.1 mm x 0.1 mm x

Fig. 4: The density and porosity in function of temperature (Harmaty, 1972)



0.8 mm in reality. The duration of time of the measurement can be set within the range of 0.1 to 1 second for one slice (Földes, 2011).

4.1 MATERIAL TESTING

For the analysis of fire damage we applied a new test method: the computer tomography (CT) method. The method is applicable because the density and porosity values in function of temperature are changed *(Fig. 4).*

During the laboratory tests the specimens were fire loaded in 5 heat steps (20 °C, 50 °C, 150 °C, 300 °C, 500 °C). After the fire loading the specimens were inspected by computer tomography. The Hounsfield unit (HU) values are conform to the density (Lublóy, Földes, Balázs, 2011).

The amounts of constituents, cement, water, aggregate and plasticizer are given in *Table 1*.

Table 1: Experimental concrete mix (kg/m3)

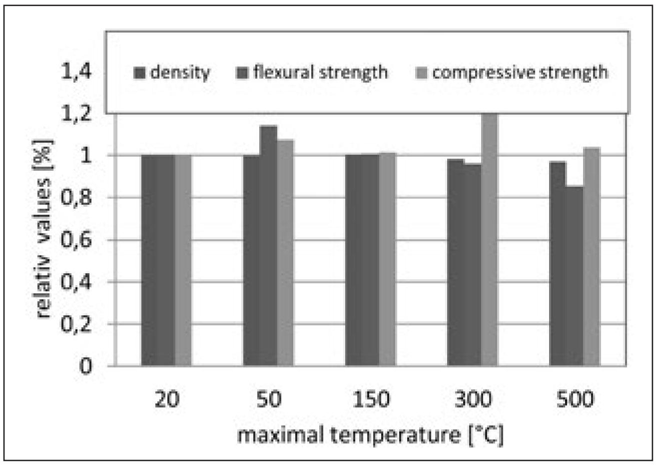
Material		V%	kg/m^3
aggregate	0/4 mm	40%	755
	4/8 mm	25%	472
	8/16 mm	35%	661
cement	CEM I 42,5 N		400
water	m_w/m_c=	35%	140
plasticizer cem. m%		1,50%	6

After heating up to given temperatures the specimens were kept at these maximum temperatures for two hours. Specimens were then cooled down in laboratory conditions. After cooling down the specimens the compressive strength were measured on cylinders (Ø50 mm, h=100 mm) and the flexural test on beams (70·70·250 mm). The test results are demonstrated in *Fig. 5*.

Development of residual compressive strength after fire is demonstrated in *Fig. 5*. The following conclusions could be drawn:

- 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. This valley might be explained by the pore water content of tested concretes at an age of 28 days. The valley ends up with about 100% strength. The valley could be explained by the relatively high pore water content of the 28 days concrete specimens.

Fig. 5: Density (based on CT measurments) as well as flexural and compressive strength measurements (based on mechanical test)



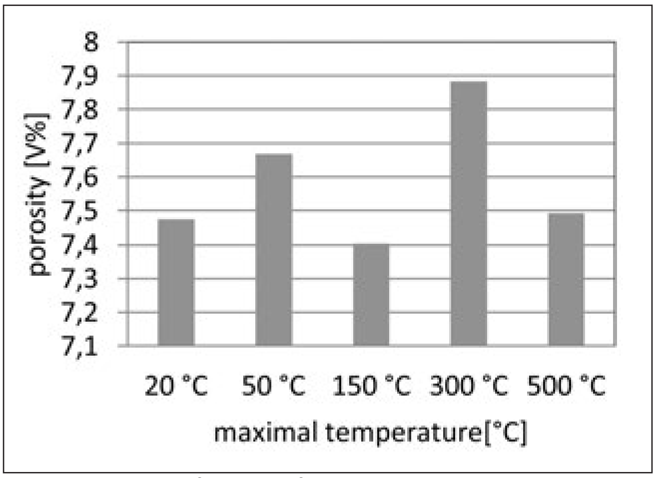


Fig. 6: Porosity as a functions of temperature

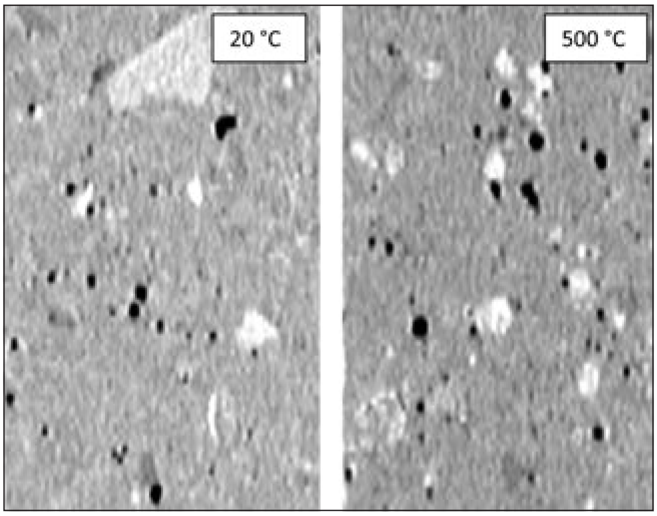


Fig. 7: Pore distribution as a function of temperature (CT images)

- This valley is followed by decrease of compressive strength for higher values of maximal temperatures.
- Most considerable reduction of compressive strength took place between 300 °C and 500 °C.

After temperature loading the specimens were tested with computer tomography. Computer Tomography (CT) seems to able to demonstrate porosity differences in concrete (*Figs. 6 and 7*).

In *Fig. 7* CT images shown for cross-sections of concrete specimens at room temperature (20 °C) as well as after 500 °C temperature loading. The CT image after 500 °C indicates higher numbers and higher diameter of pores compared to room temperature.

4.2 SPECIMENS FROM A STRUCTURAL ELEMENT

For a practical analysis core samples were taken and CT investigations were carried out a 2-hour hydro-carbon fire loaded specimen. It was observed that the Hounsfield-values are also provided under CT scans which are linearly proportional to the density. Fire loading has an effect on the change of HU values of the outer layers, so this method is suitable for the analysis of fire damaged structures.

Assessing the residual capacity of concrete structures exposed to fire is then a quite difficult task, because the traditional destructive or non-destructive testing techniques are generally not completely suitable for the inspection of such a highly heterogeneous *(fib, 2007).*

The tested specimen was a concrete core which was drilled

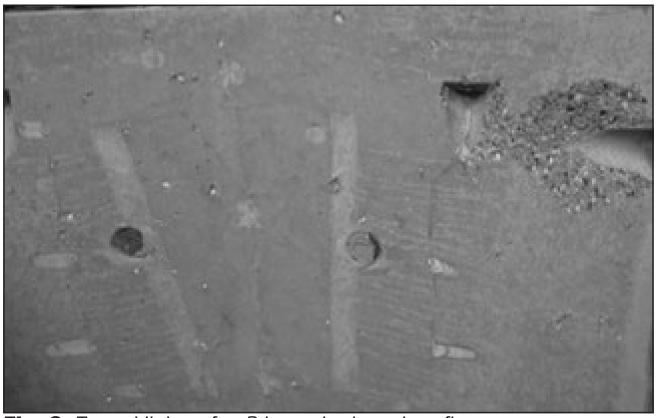


Fig. 8: Tunnel lining after 2 hours hydrocarbon fire

from a prefabricated tunnel lining element that was subjected to hydrocarbon fire of 2 hours. *Fig. 8* presents the photo of the lining element after fire.

Fig. 9 presents the photo of concrete core *(Fig. 9 a),* the CT image of concrete core in black and white *(Fig. 9 b),* the CT image of concrete core in colours *(Fig. 9 c)* and the distribution of Hounsfield-units along the axis of concrete after the fire test *(Fig. 9 d)*.

The measured Hounsfield values *(Fig. 9 d.)* show some undulation according to the variation of aggregates and cement stone. Considerable reduction in measured Hounsfield values is observed close to concrete surface that was subjected to fire (from slices 180 to 260). This region fits well to the region of change in colour of concrete in *Fig. 9a* (from grey to pink). (The peak in the measured Hounsfield values at slice Nr. 30 indicates existence of steel reinforcement).

5. CONCLUSIONS

In addition to the mechanical properties of structural materials, their behaviour during and after elevated temperatures are also of high importance *(Janson, Boström, 2004)*. Residual compressive strength of concrete exposed to high temperatures is influenced by the following factors *(Thielen, 1994):* water to cement ratio, cement to aggregate ratio, type of aggregate, water content of concrete before exposing it to high temperatures and the fire process.

High temperatures produce changes in material structure as well as changes in the mechanical properties. It is often a difficult question, how these changes can be detected and how pronounced are they.

Our study concentrated on the extension of applicability of non-destructive testing methods. The potential in use of X-ray computer tomography (CT) was demonstrated with material testing on concrete prism cast in our laboratory as well as on a concrete core drilled from a fire tested prefabricated reinforced concrete tunnel lining subjected hydrocarbon fire for two hours. The core was drilled from the tunnel lining after fire and was tested with CT. CT images and distribution of measured Hounsfield-values are presented in the paper in addition to the photo of the core. Considerable reduction of Hounsfield-units was observed towards the concrete surface that was subjected to fire. This region coincides well with the region were the photographic picture show light pink colour of concrete instead of pink.

6. REFERENCES

Bogner, P., Földes, T., Závoda, F., Repa, I., (2003), "Application of CT and MR in the carbon hidrogen research", *Hungroise Radyology* 2003. pp. 231-237.

ÉMI (2000), "Expertise on the fire-damaged structures of the Budapest Sports Hall" (Szakértői vélemény a Budapest Sportcsarnok tűzkárt szenvedett épületszerkezeteiről), *Report*

fib bulletin 38 (2007), „Fire design of concrete structures, materials, structures and modelling", ***fib*** bulletin 38, Lausanne, ISBN: 978-2-88394-078-9

Földes, T., Kiss, B., Árgyelán, G., Bogner, P., Repa, I., Hips, K. (2004), "Application of medical computer tomograph measurements in 3D reservoir characterization", *Acta Geologica Hungarica*, Vol.47/1, pp-63-73

Földes, T. (2011), „Integrated processing based on CT measurement", *News in geometry*, Vol 1, pp. 23-41.

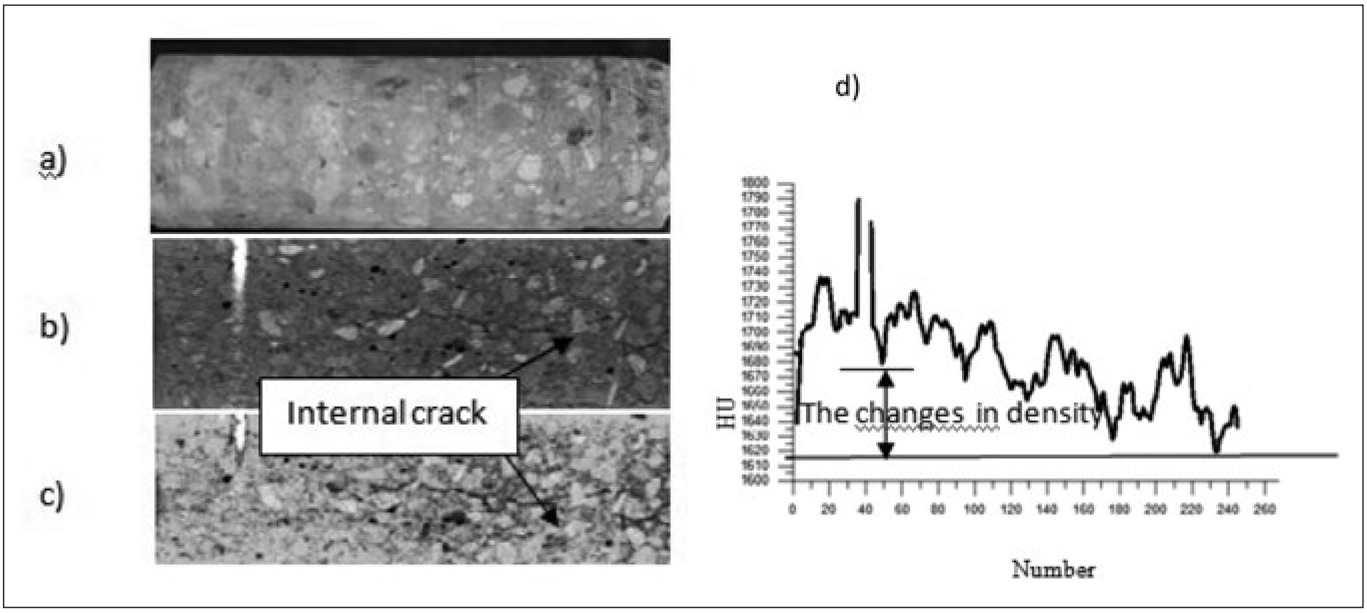
Hinrichsmeyer, K. (1987), „Strukturorientierte Analyse und Modellbeschreibung der thermischen Schädigung von Beton", *Heft 74 IBMB*, Braunschweig

Janson, R., Boström, L. (2004), "Experimental investigation on concrete spalling in fire", *Proceedings for Fire Design of Concrete Structures: What now?, What next?* (Eds.: P. G. Gambarova, R. Felicieti, A. Meda, P. Riva) Milan, 2-3 Dec. 2004, pp.115-120.

Kiss, I. (1986): The Budapest Sports Hall, Hungarian Architecture (Budapest Sportcsarnok, *Magyar* Építészet) 1986/1, pp. 20-22.

Khoury, G. A., Grainger, B. N., Sullivan, P. J. E. (1985): "Transient thermal strain of concrete: literature review, conditions within specimen and behaviour of individual constituents", *Magazine of Concrete Research*, Vol. 37, No. 132, 1985 pp. 37-48.

Fig. 9: The CT section of core and the Hounsfield values
a) concrete core from tunnel lining b) CT image in black and white c) CT image in colours d) distribution of measured Hounsfield values



Kordina, K. (1997): „Fire resistance of reinforced concrete beams“ (Über das Brandverhalten punktgeschützter Stahbetonbalken), *Deutscher Ausschuss für Stahlbeton, Heft 479*, ISSN 0171-7197, Beuth Verlag GmbH, Berlin, 1997.

Lublóy, É., Földes, T., Balázs, L. Gy. (2011), “Potencials in use of X-ray computer tomography. *Proceedings, Central European Congress on Concrete Engineering CCC2011* (Eds.: Balázs, L. Gy., Lublóy, É.), 22-23 Sept. 2011, Balatonfüred, Hungary, pp. 37-40, ISBN: 978-963-313-036-0

Thielen, K. Ch. (1994), „Strength and Deformation of Concrete Subjected to high Temperature and Biaxial Stress-Test and Modelling“ (Festigkeit und Verformung von Beton bei hoher Temperatur und biaxialer Beanspruchung - Versuche und Modellbildung), *Deutscher Ausschuss für Stahlbeton*, Heft 437 Beuth Verlag GmbH, Berlin

Schneider, U., Lebeda, C. (2000), „*Baulicher Brandschutz“*, Stuttgart; Berlin; Köln: Kohlhammer, 2000

Schneider, U. (1988), “Concrete at high temperatures - a general review”, *Fire Safety Journal*, Vol. 13, pp. 55-68

Schneider, U., Weiß, R. (1997), „Kinetische Betrachtungen *über* den thermischen Abbau zementgebundener Betone und dessen mechanische Auswirkungen”, *Cement and Concrete Research*, Vol 11, pp. 22-29

Waubke, N. V. (1997), „Physical analysis of strength reduction of concrete up to 1000 °C“ (Über einen physikalischen Gesichtspunkt der Festigkeitsverluste von Portlandzement−betonen bei Temperaturen bis 1000 °C − Brandverhalten von Bauteilen), *Dissertation*, TU Braunschweig, 1973.

http://911research.wtc7.net/wtc/analysis/compare/windsor.html
http://911research.wtc7.net/wtc/analysis/compare/fires.html

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EFFECT OF COMPACTION, CURING AND SURFACE MOISTURE CONTENT ON THE REBOUND HARDNESS OF CONCRETE



Katalin Szilágyi – Adorján Borosnyói – Tamás Mikó

Compressive strength of structural concrete is influenced not only by the composition of the material but the preparation of the structure as well. The degree of compaction, the method and intensity of curing have major effect on structural performance. Non-destructive testing (NDT) can be performed during in-situ testing; hardness testing practice of concrete exclusively applies nowadays the rebound hammer. With the rebound hammer, the properties of the concrete can be studied in the very vicinity of the device. Since hardness and strength of concrete are interrelated material properties, the rebound hardness is mostly influenced by the same parameters as strength. The present experimental tests intended to study the influence of the degree of compaction, the duration of curing and the surface moisture content on the rebound hardness of concrete. It was revealed that the decrease in compressive strength caused by the compaction deficiency can not be detected by the rebound hammer. The influence of insufficient curing can be detected by the rebound hammer as a decrease in the rebound index, however, the decrease of the rebound index is lower than the decrease of the compressive strength. Results highlight the importance of moisture content of the concrete surfaces and demonstrate the magnitude of its influence for normal strength concrete at mature age. Findings are of great practical importance as the studied phenomena would result unsafe strength estimation in a practical situation.

Keywords: concrete strength, rebound hardness, curing, compaction, moisture content

1. INTRODUCTION

Compressive strength is one of the main technical characteristics of hardened concrete and it is used most often also for the qualification and designation of concrete. During in-situ testing of concrete structures, non-destructive testing (NDT) can be performed and the strength of concrete can be estimated from the measured results with limited reliability (Szilágyi et al, 2014). Hardness testing practice of concrete exclusively applies nowadays the rebound hammer. The hardness testing devices have been developed for in-situ testing of concrete based on the observation that the surface hardness of concrete can be related to the compressive strength of concrete.

The compressive strength and parallel the surface hardness of concrete depend on several factors related to the composition of the material and the preparation of the structure: the type of aggregate and cement, the water-cement ratio, the degree of compaction, the method and duration of curing, the quality of the surface and the actual water content in the pores of the structural concrete during testing. Surface hardness and compressive strength of concrete are interrelated material properties (Szilágyi et al, 2011a; 2011b), therefore rebound hardness is influenced by the parameters introduced.

2. PARAMETERS INFLUENCING THE REBOUND HARDNESS OF CONCRETE

2.1 INFLUENCES BY THE REBOUND HAMMER

In the Schmidt rebound hammer, mechanical parts (i.e. springs, sliding hammer mass, etc.) provide the impact load and mechanical (Original Schmidt hammer) or digital (DIGI-Schmidt hammer, Silver Schmidt hammer) parts are responsible for readings. The value of the Schmidt rebound index depends on energy losses due to friction during acceleration and rebound of the hammer mass and that of the index rider, on energy losses due to dissipation by reflections and attenuation of mechanical waves inside the steel plunger; and of course, on energy losses due to dissipation by concrete crushing under the tip of the plunger. This latter loss of energy makes the Schmidt rebound hammer suitable for strength estimation of concrete.

2.2 INFLUENCES BY THE CONCRETE STRUCTURE

The energy dissipated in the concrete during local crushing initiated by the impact depends on the properties of the concrete in the very vicinity of the tip of the plunger. Therefore, the measurement is sensitive to the scatter of local strength of concrete due to its inner heterogeneity (Szilágyi et al, 2013; Borosnyói, Szilágyi, 2013).

The amount of energy dissipated in the concrete can be higher for a concrete of lower strength/lower stiffness compared to lower energy dissipation in a concrete of higher strength/higher stiffness. As it is possible to prepare concretes of the same strength but having different Young's modulus, it is also possible to measure the same rebound index for different concrete strengths or to measure different rebound indices for the same concrete strengths. Young's modulus of the aggregate has considerable influence on the rebound index.

The most significant influence on strength of concrete was found to be the water-to-cement ratio (*w/c*) of the cement paste. Further important influencing parameter is the carbonation depth of the concrete additionally to the parameters mentioned above.

The present paper focuses on the effect of compaction, curing and moisture content of concrete.

3. EFFECT OF COMPACTION ON THE COMPRESSIVE STRENGTH OF CONCRETE

Strength of concrete is increased with increasing degree of compaction. Compaction is the process which expels entrapped air voids from fresh concrete and packs the aggregate particles together so as to increase the body density of concrete (CCAA, 2006b). When fresh concrete is placed in the form, air voids can occupy 5 to 20 percent and of the total volume (Neville, 2000).

The usual method of compaction is vibration. Vibration has the effect of fluidifying the mortar component of concrete so that internal friction is reduced and packing of coarse aggregate takes place. Continuing vibration expels most of the air voids, but total absence of entrapped air can not be achieved normally. Vibration must be applied uniformly to the entire concrete mass, otherwise some parts would be fully compacted while other might be segregated due to over-vibration (Neville, 2000). Overvibration brings excess paste to the surface and enhances bleeding (Mindess, Young, 1981).

The aggregate particles, although coated with mortar, tend to arch against one another and are prevented from consolidating by internal friction. Compaction of concrete is, therefore, a two-stage process. First the aggregate particles are set in motion and slump to fill the form. In the second stage, entrapped air is expelled. Initial consolidation of the concrete can often be achieved relatively quickly. Entrapped air takes a little longer to rise to the surface. Compaction must therefore be prolonged until air bubbles no longer appear on the surface (CCAA, 2006b).

Fig. 1 indicates the considerable effect of compaction on compressive strength. For example, the strength of concrete containing 10% of entrapped air voids may be as little as 50% that of the concrete when fully compacted. Permeability may be similarly affected since compaction promotes a more even distribution of pores and they become discontinuous. As a result, permeability is reduced and durability is improved (CCAA, 2006b).

4. EFFECT OF CURING ON THE COMPRESSIVE STRENGTH OF CONCRETE

Curing is the name of procedures used for promoting the hydration of cement paste. It consists of control of temperature and moisture movement. The object of curing is to keep concrete saturated as long as possible, until the originally water-filled space in the cement paste is filled to the desired extent by hydration products (Neville, 2000).

Since the hydration of cement does take time – days, and even weeks – curing must be undertaken for a reasonable period of time if the concrete is to achieve its potential strength and durability (CCAA, 2006a). In the case of site concrete, active curing usually stops long before the maximum possible hydration takes place.

The influence of curing on strength can be demonstrated by a comparison of the strength of specimens stored in water with the strength of those stored under other conditions. An example is shown in *Fig. 2*, obtained for concrete with w/c = 0.50 (Neville, 2000). *Fig. 2* shows the effect of limited moist curing on the development of the compressive strength of concrete. When moist curing is stopped, the rate of strength development slows down, and further strength gain soon ceases. A 3-day period of moist curing will only allow the concrete to reach 75-80% of the potential 28-day strength which can be achieved

Fig.1: Loss of strength of concrete owing to insufficient compaction (CCAA, 2006b)

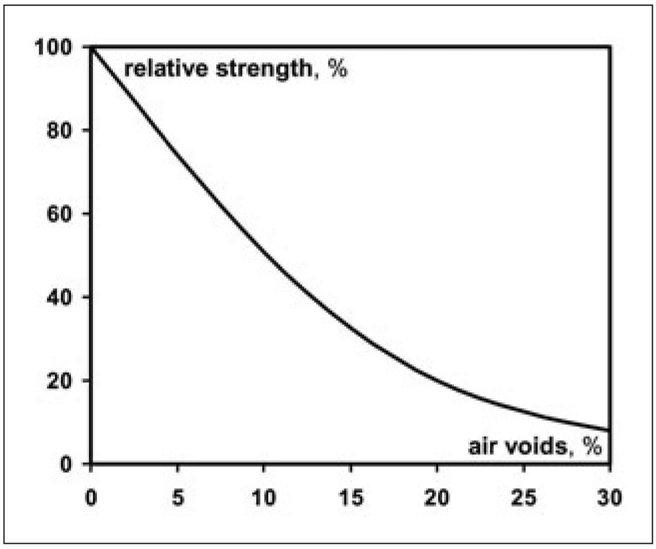
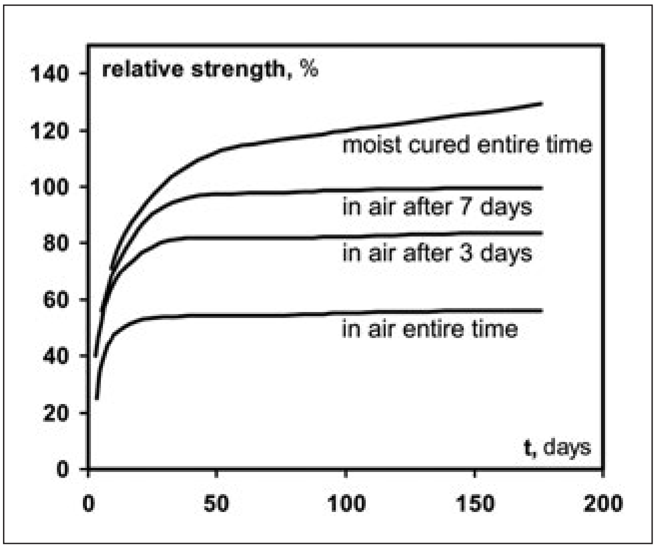


Fig. 2: Effect of the duration of curing on the compressive strength of concrete (Mehta, Monteiro, 2006)



with continuous moist curing (Neville, 2000). Furthermore, none of the additional 25-30% of strength gain that can be expected beyond 28 days will be realised (Mindess, Young, 1981).

The period of curing depends on the properties required of the concrete, the purpose for which it is to be used, and the ambient conditions (temperature and relative humidity). Curing is designed primarily to keep the concrete saturated, by preventing the loss of moisture from the concrete. Curing may be applied in a number of ways either by leaving formwork in place, covering the concrete with an impermeable membrane after the formwork has been removed, by the application of a suitable chemical curing agent, or by a combination of such methods (CCAA, 2006a).

5. EFFECT OF MOISTURE CONTENT ON THE COMPRESSIVE STRENGTH OF CONCRETE

Most concrete specifications, such as EN and ASTM Standards require that concrete specimens should be stored and tested in a water saturated condition (Mehta, Monteiro, 2006; Mindess, Young, 1981). This condition has the advantage of being better reproducible than a dry condition which includes widely varying degrees of dryness (Neville, 2000).

Testing in a dry condition leads to higher strength. The reasons for this are not completely understood. Three partly different explanations are given here from the technical literature.

According to (Mindess, Young, 1981) it may have something to do with the change in the structure of the C-S-H on drying, or it may simply represent a change in the internal friction and cohesion on a macroscopic scale; that is, moisture may have a lubricating effect, allowing particles to slip by each other in shear more easily. The lower compressive strength of wet concrete may also be due to the development of internal pore pressure as a load is applied.

According to (Neville, 2000) the loss of strength due to wetting of a compression test specimen is caused by the dilatation of the cement gel by adsorbed water: the forces of cohesion of the solid particles are then decreased. Conversely, when on drying the wedge-action of water ceases, an apparent increase in strength of the specimen is recorded.

In the view of (Mehta, Monteiro, 2006) the lower strength of the saturated concrete is attributed to the disjoining pressure within the cement paste.

The quantitative influence of drying varies: with 34 MPa concrete an increase in compressive strength up to 10% has been reported on thorough drying, but if the drying period is less than 6 hours, the increase is generally less than 5%. Other tests have shown the decrease in strength in consequence of 48-hour wetting prior to test, to be between 9 and 21% (Neville, 2000).

In compression tests it has been observed that air-dried specimens show 20-25% higher strength than corresponding specimens tested in a saturated condition (Mehta, Monteiro, 2006).

For an oven-dried specimen, the increase in strength is of the order 10-15%. This increase in strength appears to be reversible, as subsequent resaturation will return the concrete to its original strength at water saturated condition (Mindess, Young, 1981).

The effect of moisture content on strength becomes an important consideration when testing drilled cores.

6. LABORATORY TESTS

An experimental programme was completed on normal weight concretes at the Budapest University of Technology and Economics (BME), Department of Construction Materials and Engineering Geology to study the influence of the degree of compaction, the duration of curing and the surface moisture content on the rebound hardness of concrete.

6.1 EFFECT OF THE DEGREE OF COMPACTION

The concrete was mixed from Danube sand and gravel using CEM I 42.5 N cement with the water-cement ratio of w/c=0.50.

The tested concrete mix was designed in accordance with present concrete construction needs, i.e. slightly paste over-saturated mix. Consistency of the tested concrete mix was constant: 450±20 mm flow provided by superplasticizer admixture. Design air content of the properly compacted fresh concrete for the tested mixes was 1.0 V%.

150 mm cube specimens were cast into steel formworks with three different compaction procedures (resulting three different degrees of compaction): 1) no compaction, 2) manual compaction and 3) compaction by a vibrating table.

The specimens were stored in water for 7 days as curing. After 7 days the specimens were stored at laboratory condition. Tests were carried out at two different ages of concrete: 28 days and 240 days. The compositions tested at 2 different ages with double repetitions resulted a total number of 12 specimens.

Surface hardness tests were carried out by N-type and L-type original Schmidt rebound hammers at the age of 28 days and 240 days. Altogether twenty individual readings were recorded with the rebound hammers used in horizontal direction on two parallel vertical sides of the 150 mm cube specimens restrained by 40 kN force into a hydraulic compressive strength tester just before the compressive strength tests (according to EN 12390-3) were performed.

6.2 EFFECT OF THE DURATION OF CURING

The composition of concrete was the same as was designed for studying the influence of the degree of compaction.

150 mm cube specimens were cast into steel formworks by a vibrating table.

Water curing for three durations was applied: 1) no curing, 2) two days of water curing and 3) seven days water curing. The specimens with no curing and the water cured specimens after water curing were stored at laboratory condition. Tests were carried out at two different ages of concrete: 28 days and 240 days. The compositions tested at 2 different ages with double repetitions resulted a total number of 12 specimens.

The same surface hardness tests were carried out as it was introduced in the previous *section* (*6.1*).

6.3 EFFECT OF THE MOISTURE CONTENT OF THE SURFACE

The influence of the moisture content was investigated on 11 individual, 4 to 6 years old standard cube specimens made of unknown concrete mixtures. Two moisture conditions were examined, air dry laboratory condition and a subsequent water saturated condition by immersing the specimens into water tank until reaching water saturated condition.

The same surface hardness tests were carried out as it was introduced in section 6.1.

7. EXPERIMENTAL RESULTS

7.1 RESULTS ON THE EFFECT OF THE DEGREE OF COMPACTION

A volumetric property is measured during compressive strength testing and the compressive strength of concrete is primarily determined by the properties of the interfacial transition zone between the aggregate particles and hardened cement paste since it is the weakest part of concrete as a heterogeneous material system.

As mentioned above, 1 V% (10 liter/m^3) increase in air content of concrete resulted in 5% decrease in compressive strength (Neville, 2000). Air is entrapped within the fresh concrete during mixing, which cannot be completely removed during the casting and compaction process. When fresh concrete is placed into the form, the air content can be about 5-20 V% depending on the actual composition and consistency. In case of normal concrete (not frost resistant with air entraining admixture), about 0.5 to 2.5V% air content can be realised, therefore, design air content is prescribed in this range during mix design.

Design air content of the concrete mixture used for the present tests was 1.0 V%.

During the present experimental study, it was intended to determine if there is any influence of the compaction on the rebound hardness or how much the influence is.

Compaction methods described in *section 6.1* were chosen to provide significant differences in compressive strength. The cube specimens compacted by the different methods resulting different degrees of compaction can be observed in *Fig. 3*.

Results on compressive strength and rebound index can be studied in *Fig. 4*. Due to the identical duration of curing in case of all three compaction methods, the degrees of hydration were also identical.

Fig. 4a, *Fig. 4c and Fig. 4e* show the compressive strength and rebound index provided by N-type and L-type Schmidt-hammer, for the specimens compacted with the three different methods, at the age of 28 and 240 days. *Fig. 4b*, *Fig. 4d* and *Fig. 4f* show the values of the three tested parameters related to the parameter measured on specimen compacted by vibrating table. As it was expected, the compressive strength of specimens with insufficient compaction was lower than that of the specimens compacted by vibrating table.

The compressive strength of the specimens compacted manually decreased by 2.3 N/mm^2 (3.9%) at the age of 28 days and 4.8 N/mm^2 (6.9%) at the age of 240 days related to the compressive strength of the specimens compacted by vibrating table.

The compressive strength of the specimens with no compaction decreased by 9.0 N/mm^2 (15.4%) at the age of 28 days and 12.1 N/mm^2 (17.4%) at the age of 240 days related to compressive strength of the specimens compacted by vibrating table.

Despite of the decrease in compressive strength, the average rebound indices recorded by both types of Schmidt hammer are almost identical in case of specimens compacted by all three methods, independently of the age of concrete at testing. Thus, it can be concluded that the decrease in compressive strength caused by the compaction deficiency *can not be detected* by the rebound surface hardness testing. The increase in air content of concrete caused by insufficient compaction are not reflected in the rebound indices.

The equality of the rebound indices recorded on specimens compacted by different methods can be understood and explained by the manner of performing the rebound hammer test. During testing the test locations are chosen on the concrete surface where intact hardened cement paste is visible; measuring on air bubbles and compaction cavities should be avoided.

Therefore, the rebound hammer measures the hardness of the hardened cement paste, which is assumed to be not influenced by the degree of compaction of concrete.

7.2 RESULTS ON THE EFFECT OF THE DURATION OF CURING

During the experimental present study it was intended to determine how much is the influence of the duration of curing on the rebound hardness of concrete. The durations of water curing described in *section 6.2* were chosen to study the differences in compressive strength.

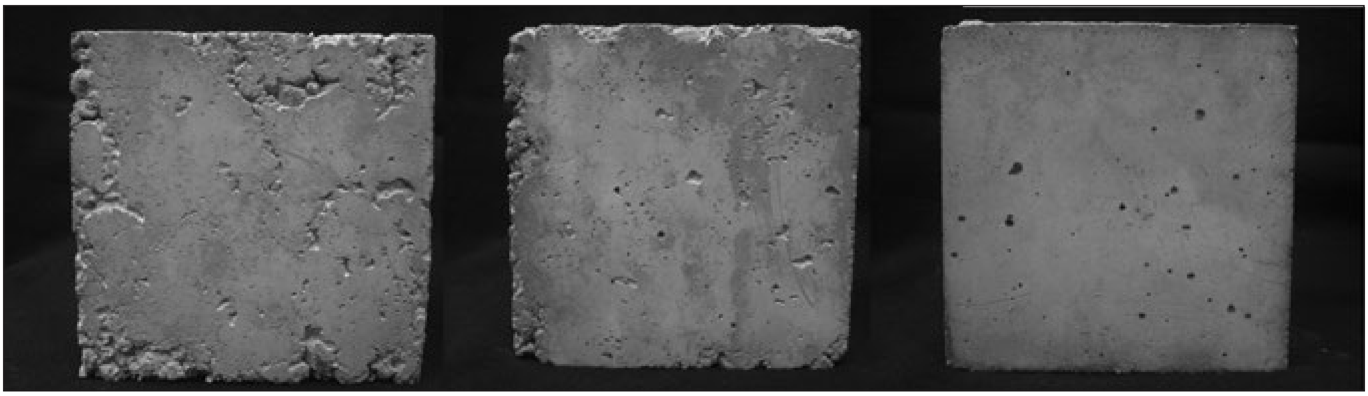
It can be realised in *Fig. 5* that the degree of hydration of the cement paste is increasing with the increase of the duration of curing that results the increase of compressive strength, as well as the increase of the rebound index.

Fig. 5a, *Fig. 5c* and *Fig. 5e* show the compressive strength and rebound index provided by N-type and L-type Schmidt-hammer for the specimens cured for three different durations of time, at the age of 28 and 240 days. *Fig. 5b*, *Fig. 5d* and *Fig. 5f* show the values of the three tested parameters related to the parameter measured on properly cured (i.e. for 7 days) specimens. As it was expected, the compressive strength of specimens of lower duration of curing was lower than that of specimens cured properly (for 7 days).

The compressive strength of the specimens cured for 2 days decreased by 3.2 N/mm^2 (5.6%) at the age of 28 days and 5.7 N/mm^2 (8.3%) at the age of 240 days related to the compressive strength of the specimens cured for 7 days.

The compressive strength of the specimens with no water curing decreased by 9.7 N/mm^2 (16.9%) at the age of 28 days

Fig. 3: Surfaces of specimens prepared by different compaction methods



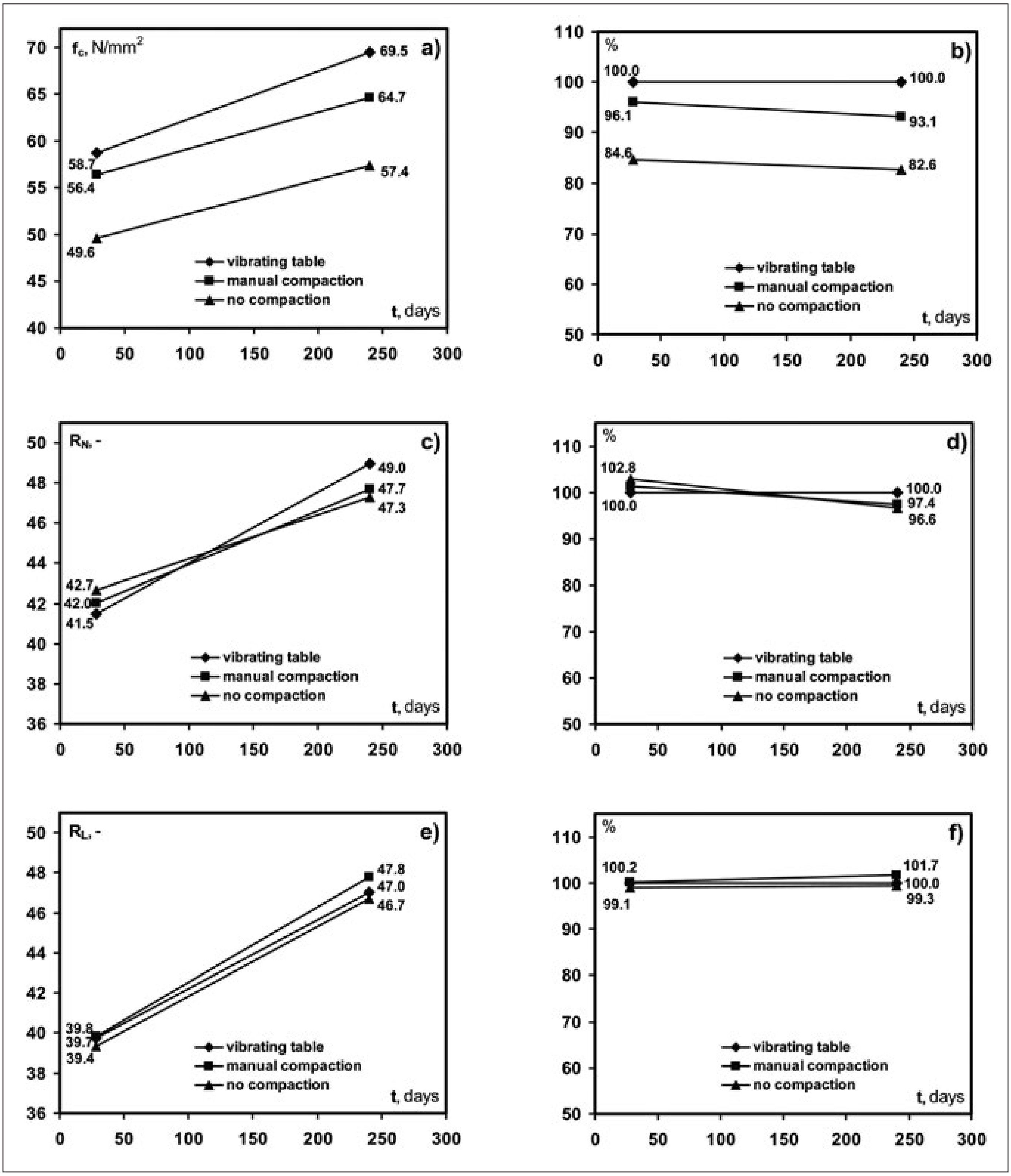


Fig. 4: *Effect of the degree of compaction on the compressive strength and average rebound index of concrete*

and 16.9 N/mm^2 (24.8 %) at the age of 240 days related to the compressive strength of the specimens cured for 7 days.

The average rebound index recorded by the N-type Schmidt hammer on the specimens cured for 2 days decreased by 1.2 units (3.0%) at the age of 28 days and 2.2 units (4.6%) at the age of 240 days related to the average rebound index of the specimens cured for 7 days.

The average rebound index recorded by the N-type Schmidt hammer on the specimens with no water curing decreased by 3.1 units (7.9%) at the age of 28 days and 5.7 units (12.0%) at the age of 240 days related to the average rebound index of the specimens cured for 7 days.

The average rebound index recorded by the L-type Schmidt hammer on the specimens cured for 2 days decreased by 0.1 units (0.3%) at the age of 28 days and 2.3 units (4.8%) at the age of 240 days related to the average rebound index of the specimens cured for 7 days.

The average rebound index recorded by the L-type Schmidt hammer on the specimens with no water curing decreased by 1.8 units (5.0%) at the age of 28 days and 8.1 units (17.2%) at the age of 240 days related to the average rebound index of the specimens cured for 7 days.

The influence of insufficient curing *can be detected* by the Schmidt hammer as a decrease in the rebound index, however, the decrease of the rebound index is lower than the decrease of the compressive strength. In case of no water curing the

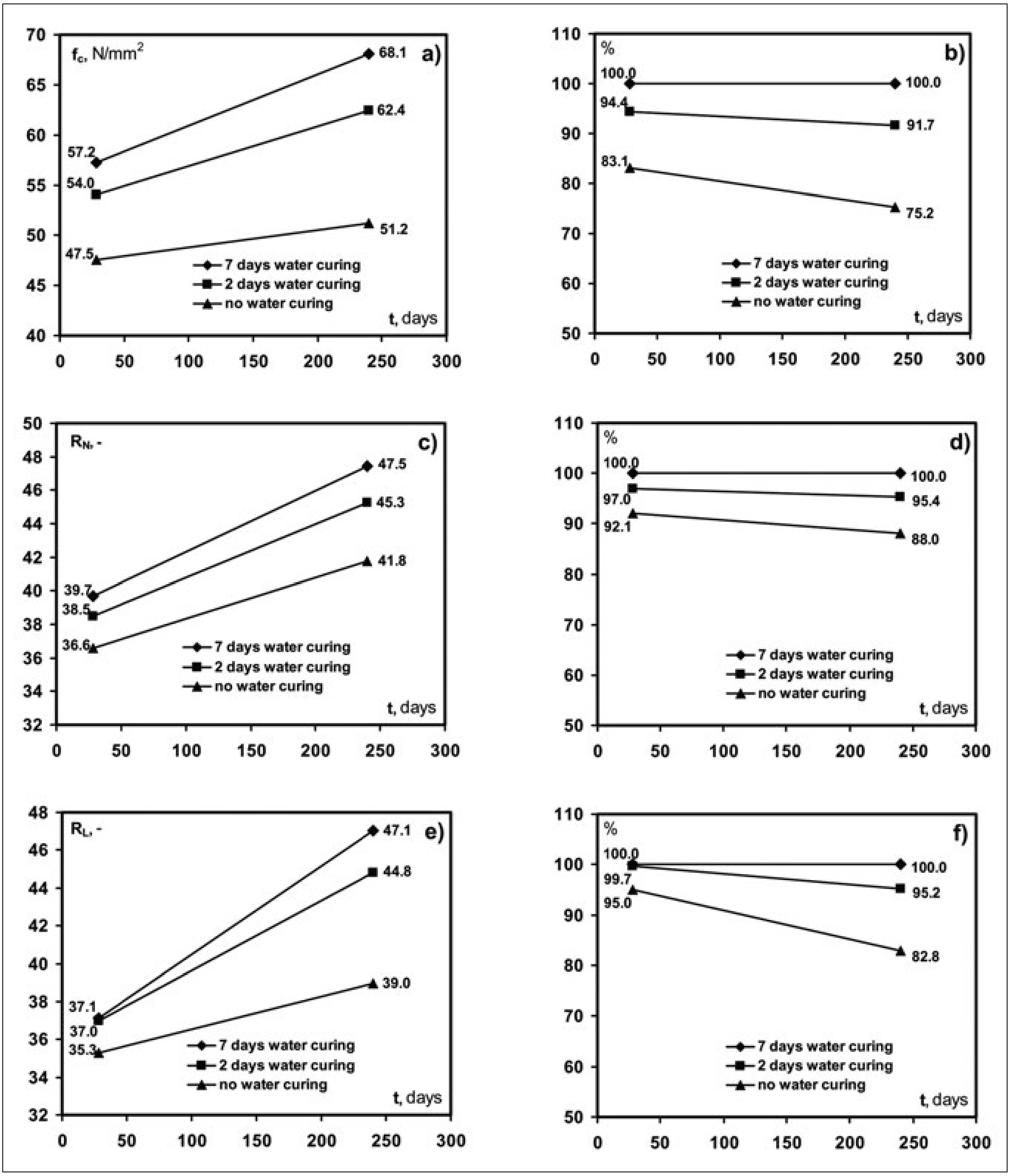


Fig. 5: *Effect of the duration of water curing on the compressive strength and average rebound index of concrete*

decrease of compressive strength at the age of 28 days is 16.9%, while the decrease of rebound index (recorded by N-type rebound hammer) is about the half of it (7.9%).

7.3 RESULTS ON THE EFFECT OF MOISTURE CONTENT OF THE SURFACE

Several scientific papers indicate in the technical literature that the increase of moisture content of concrete results decrease in the rebound index. It can be found that the difference can reach 20 % related to the dry surface, however, numerical data is given in only one reference. *Samarin* published numerical data in (Malhotra, Carino, 2004). It introduces his observations on concrete mixes that have f_{cm} = 35.0 MPa compressive strength (after wet curing of 28 days). It was concluded based on specimens of different curing methods and different moisture contents of their surface that the average rebound index is by 7 % lower (R_m = 27.2) in case of the saturated concrete surface than that of the dry concrete surface (R_m = 29.3) (Malhotra, Carino, 2004).

Compressive strength of concrete specimens of the present research was f_{cm} = 67.5 MPa in dry condition.

The average rebound index, the standard deviation of the rebound index and the coefficient of variation of the rebound index can be studied in *Fig. 6*. Results demonstrated a considerable decrease in the rebound index of the specimens having water saturated surface, which is higher than that can be found in the technical literature.

The average rebound index recorded by the N-type Schmidt hammer of the saturated specimens was 26.8% lower than that of the air dry specimens (*Fig. 6a*).

The average rebound index recorded by the L-type Schmidt hammer of the saturated specimens was 20.6% lower than that of the air dry specimens (*Fig. 6a*).

After the statistical analysis of the results, it can be also observed that the *measurement uncertainty* of the tests performed on the saturated concrete surfaces is *higher* than that of the tests performed on air dry surface. The standard deviation of the rebound index recorded by the N-type Schmidt hammer of the saturated specimens increased by 7.6% related to that of the air dry specimens, and the standard deviation of the rebound index recorded by the L-type Schmidt hammer of the saturated specimens increased by 16.6% related to that of the air dry specimens (*Fig. 6b*).

The coefficient of variation of the rebound index recorded by the N-type Schmidt hammer of the saturated specimens increased by 47.0% related to that of the air dry specimens, and the coefficient of variation of the rebound index recorded by the L-type Schmidt hammer of the saturated specimens increased by 46.8% related to that of the air dry specimens (*Fig. 6c*).

Results highlight the importance of moisture content of the concrete surfaces and demonstrate the magnitude of its influence for normal strength concrete at mature age.

Although the testing standards of the rebound hammer do not allow measurements on wet surfaces, it would be reasonable to examine wet surfaces of different moisture contents to be able to specify an upper limit of moisture content, which does not influence significantly the rebound index readings.

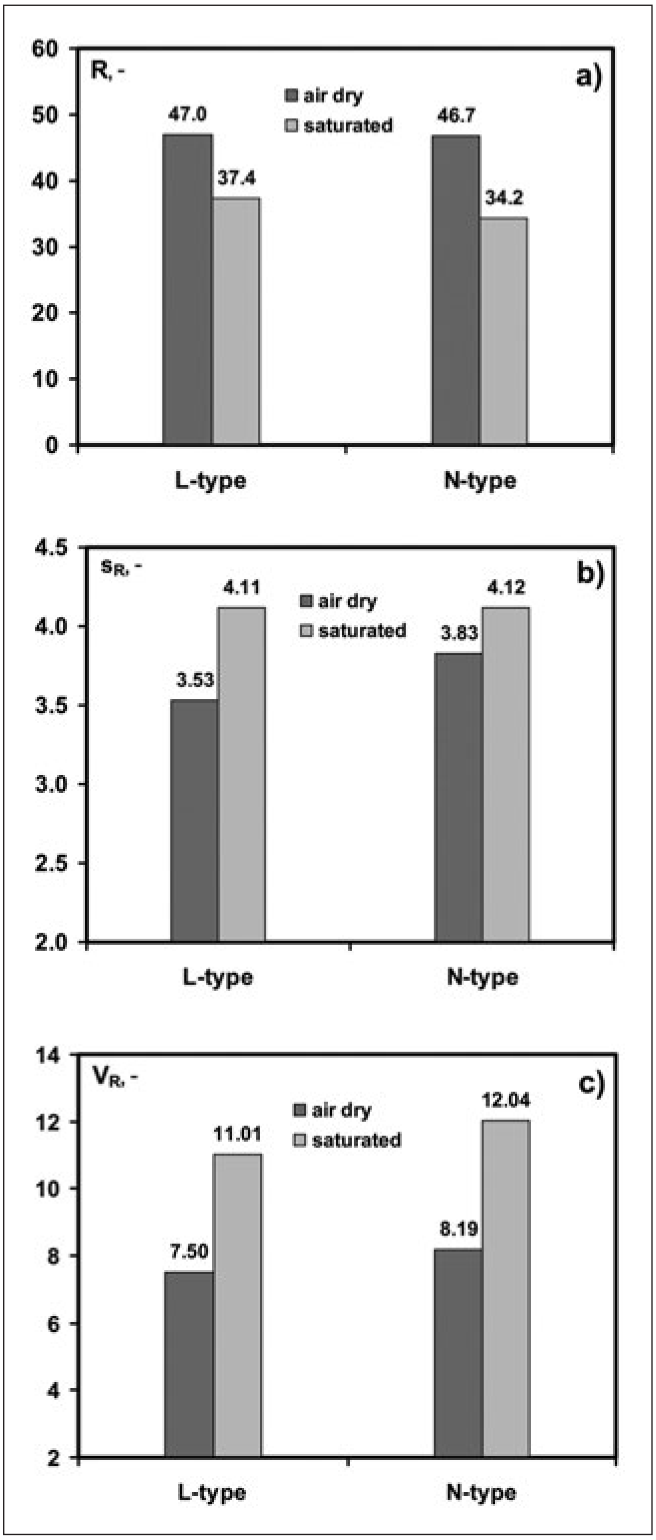


Fig. 6: *Effect of moisture content of the surface on the rebound index and the variability of the rebound index*

8. CONCLUSIONS

Findings related to the effect of the degree of compaction and duration of curing are of great practical importance.

The compressive strength of concrete can decrease even by 15-17% due to the insufficient compaction, but results demonstrated that no decrease in the rebound index can be realised. In a practical situation it would result unsafe strength estimation.

The technical literature indicates that the uniformity of strength of concrete is possible to be detected by rebound hammer, however, this claim is suggested to be restricted. The uniformity of concrete strength can be detected only when the reason is either the change of the concrete composition (primarily the water-cement ratio) or the change of curing, since the decrease in strength as a result of insufficient compaction can not be detected by the rebound hammer. In a practical situation, the near surface layer of the structural concrete is usually richer in cement paste and more compact than the internal parts, therefore, the presence of possible entrapped air voids due to insufficient compaction can be covered. Ideally, the excessive entrapped air content as a result of insufficient compaction can be noticed on the moulded surface during visual inspection, but usually, this can be only recognised when core samples are drilled and the body density is measured.

It was demonstrated that the degree of hydration of the tested concrete mixes increases with the increase of the duration of curing which results an increase not only in compressive strength but also in rebound index. Thus, the lack of curing can theoretically be detected by rebound hammer testing, however, it should be added that the extent of the decrease in rebound index due to insufficient curing is lower (8-12%) than the decrease in compressive strength (17-25%) and it would result unsafe strength estimation.

It was demonstrated that not only the rebound index decreases considerably (by 21-27%) as a result of the moisture content of the concrete surface, but also the variability parameters (standard deviation, coefficient of variation) of the rebound index increase (by 8-16% and 46-47%, respectively).

9. ACKNOWLEDGEMENTS

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10. REFERENCES

Borosnyói, A., Szilágyi, K. (2013) “Studies on the spatial variability of rebound hammer test results recorded at in-situ testing”, *Építőanyag - Journal of Silicate Based and Composite Materials,* 2013/4, Vol. 65, No. 4, pp. 102-106. http://dx.doi.org/10.14382/epitoanyag-jsbcm.2013.19

CCAA (2006a) Curing of Concrete. Datasheet. *Cement Concrete & Aggregates Australia*, April 2006, 7 p.

CCAA (2006b) Compaction of Concrete. Datasheet. *Cement Concrete & Aggregates Australia*, June 2006, 7 p.

Malhotra, V. M., Carino, N. J. (2004) Handbook on Nondestructive Testing of Concrete. *CRC Press LLC*, 364 p.

Mehta, P.K., Monteiro, P. J. M. (2006) Concrete: microstructure, properties, and materials, *McGraw-Hill*, New York, 659 p.

Mindess, S., Young, J. F. (1981) Concrete. *Prentice-Hall*, Englewood Cliffs, 671 p.

Neville, A. M. (2000) Properties of Concrete. *Prentice Hall*, Essex, 844 p.

Szilágyi, K., Borosnyói, A., Mikó, T. (2013) “Comparison of the inherent variability in rebound hammer tests performed with different testing instruments”, *Építőanyag - Journal of Silicate Based and Composite Materials,* 2013/3, Vol. 65, No. 3, pp. 68-75. http://dx.doi.org/10.14382/epitoanyag-jsbcm.2013.14

Szilágyi, K., Borosnyói, A., Zsigovics, I. (2011a) “Rebound surface hardness of concrete: Introduction of an empirical constitutive model”, *Construction and Building Materials*, Vol. 25, No. 5, May 2011, pp. 2480-2487. http://dx.doi.org/10.1016/j.conbuildmat.2010.11.070

Szilágyi, K., Borosnyói A., Zsigovics I. (2011b) „Surface hardness and related properties of concrete”, *Concrete Structures*, Vol. 12, 2010, pp. 51-57. ISSN 2062 7904

Szilágyi, K., Borosnyói, A., Zsigovics, I. (2014) “Extensive statistical analysis of the variability of concrete rebound hardness based on a large database of 60 years experience”, *Construction and Building Materials*, Vol. 53, 28 February 2014, pp. 333-347. http://dx.doi.org/10.1016/j.conbuildmat.2013.11.113

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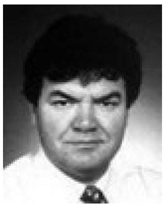
EXPERIMENTAL ANALYSIS OF THE SHEAR CAPACITY OF PRECAST CONCRETE BEAM-AND-BLOCK FLOOR SYSTEM



Kálmán Koris – István Bódi

The company Wienerberger Ltd. has recently started the production of new clay block types for their Porotherm beam-and-block floor system on the European market. In connection with the introduction of these new clay blocks in Hungary, the Department of Structural Engineering at Budapest University of Technology and Economics carried out a comprehensive examination of the blocks and the whole precast floor system according to Eurocode Standard. As we compared the calculated load carrying capacities to the values that can be found in other design tables for the same floor system, we found that our values are tendentiously lower. The primary reason for this difference was the different consideration of the shear capacity. For the investigation of realistic shear behaviour of the Porotherm floor system laboratory experiments were carried out. Results of the experiments showed that the shear capacity of the floor system is higher than the value calculated according to EC2. This improved shear capacity can also be justified by calculation using an appropriate calculation method.

Keywords: prefabrication, beam-and-block floor system, prestressed concrete, shear capacity, experimental approach

1. INTRODUCTION

Prefabricated concrete beam-and-block floor systems are widely used in Hungary and other European countries for the purposes of building construction because of their advantageous properties, such as fast and easy construction, economical operation, excellent thermal insulation and heat storage due to the ceramic surface. The company Wienerberger Ltd. – as one of the largest brick manufacturers in the world – has recently introduced new type of clay blocks for their Porotherm floor system *(Fig. 1)*. The new blocks have significantly different perforation shape and pattern than the previous ones, providing better thermal insulation and improved acoustic reduction to the floor structure (Wienerberger, 2013). The Department of Structural Engineering at Budapest University of Technology and Economics was assigned with the comprehensive examination of the new clay blocks, as well as the whole floor system, to prove its structural applicability (Bódi, Koris, 2012). Within this examination process, the following tasks were performed: laboratory testing and qualification of the new clay blocks; determination of the load carrying capacity of the Porotherm floor system according to the specification of Eurocode Standard; investigation of the force distribution effect of the cross-ribs formed by the new reduced height clay blocks (PTH 45/10 and PTH 60/10) as well as the analysis of the durability and fire resistance of the floor system (ÉMI, 2009).

The calculated load carrying capacity values were compared to values that can be found in other design tables (IBS, 2009, Linder, 2004) for the same floor system, such as:

– former Hungarian design tables using the specifications of old Hungarian Standard (MSZ),

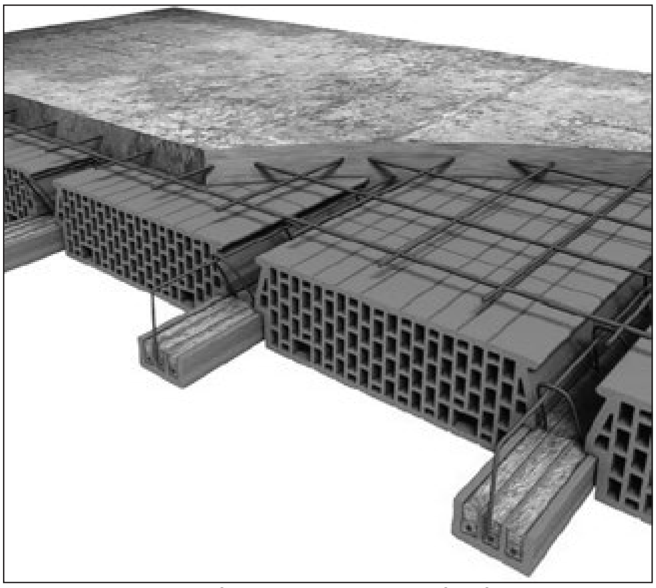


Fig. 1: The Porotherm floor system consisting of prefabricated, prestressed concrete beams, clay blocks and an in situ reinforced concrete layer

– design tables from Wienerberger Austria according to Eurocode (ÖN EN) Standard, and
– design tables from Wienerberger Slovakia according to Eurocode (STN EN) Standard.

The comparison of the results showed that our load carrying capacity values are tendentiously lower than other values (an example to this comparison can be seen in *Fig. 2*), which of course raised the question: what can be the reason of this

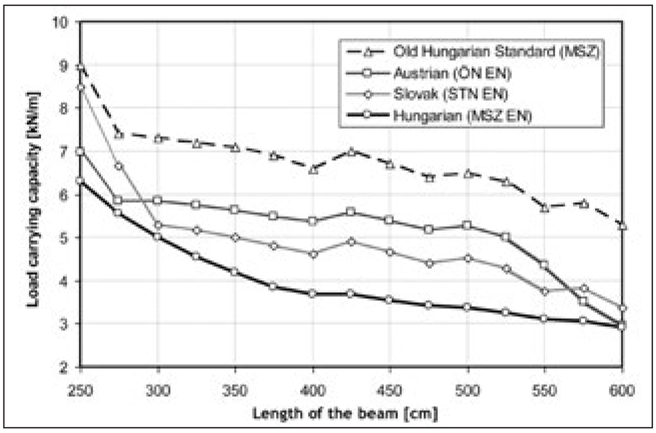


Fig. 2: *The load carrying capacity of the Porotherm floor system according to different design tables (single beam layout, 45 cm axis distance, 4 cm in-situ reinforced concrete layer on the top of clay bricks)*

difference? We separately compared the different components of the load carrying capacity, such as bending resistance and shear resistance, and we found that the results are almost the same in case of bending, while there is a significant difference in case of shear.

We have neglected the shear reinforcement during the determination of shear resistance, since the amount of the stirrups was less than the necessary minimum, as well as the stirrup spacing was not satisfactory according to the detailing rules of Eurocode 2. The shear capacity of the Porotherm floor was determined from the shear capacity of the concrete without shear reinforcement ($V_{Rd,c}$), and this value resulted a relatively lower load carrying capacity in the assembled design table. The Porotherm floor system has, however, been in use since a long time without any particular problems, so we set ourselves the goal to find out the realistic shear capacity of the system using an experimental approach. The company Wienerberger in cooperation with the Department of Structural Engineering assembled a test program for the investigation of shear resistance of the Porotherm floor system (Bódi, Koris, 2013). The test procedure and the most important results of the experiments are introduced in the followings.

2. INTRODUCTION OF THE EXPERIMENTS

2.1 DESIGN AND MANUFACTURING OF THE SPECIMENS

The laboratory tests were performed on floor specimens that were manufactured from Porotherm prestressed concrete beams and clay blocks. Specimens were designed by the Department of Structural Engineering (Bódi, Koris, 2013). To match the

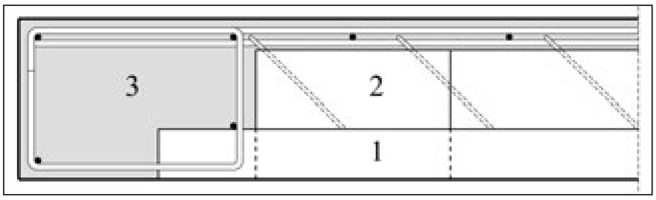


Fig. 3: *Configuration of the end of the floor specimen (1 – prefabricated Porotherm beam; 2 – Porotherm 60/17 clay block; 3 – monolithic reinforced concrete cornice)*

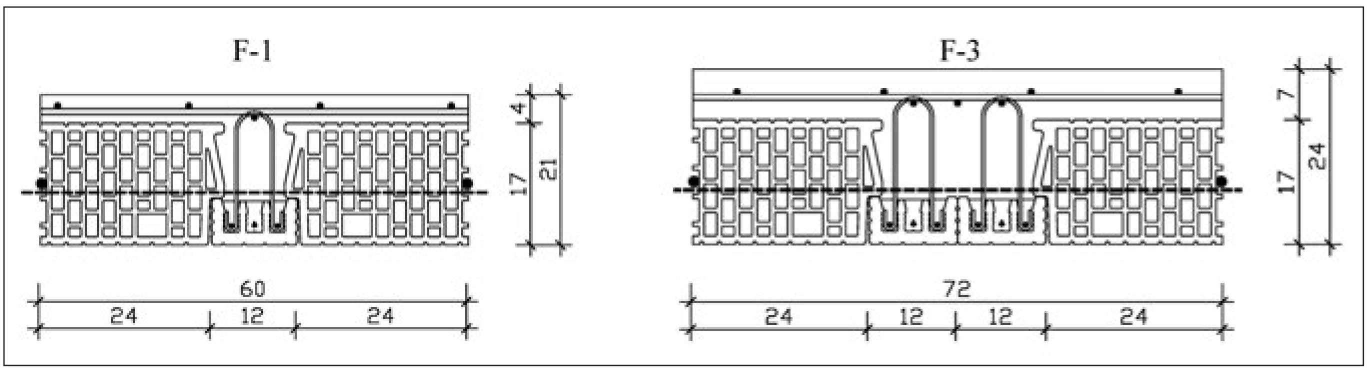
typical beam installation that is applied during construction, the reinforced concrete cornices – which are usually made monolithically on site – were also designed to the ends of the prefabricated beams *(Fig. 3)*.

Some of the specimens were constructed from prefabricated beams with modified reinforcement, which means that on one side of the beams a denser link distribution was applied compared to the original arrangement. These steel bars are usually bent up in 45º during the construction and they hold the upper reinforcement at the support before concreting the cornice. However, the modified links were designed to stay in horizontal position after the construction, too. The modified concrete beams were manufactured by the Wienerberger's factory in Sopron.

For the determination of the load carrying capacity of the floor 4 different types of specimens were designed. Specimen type F-1 was constructed according to the recommendations of the manufacturer, using 1 piece of 4.50 m long F-450 Porotherm beam (single beam layout). Specimen type F-2 was constructed from 1 piece of F-450 beam with modified reinforcement. On the opposite side of the beam an additional horizontal steel bar was also placed to take the additional tensile forces caused by the arch-effect at the end of the beam. The additional steel bar was tied into the cornice to ensure their proper anchorage. The specimen type F-3 was made from 2 pieces of 6.50 m long F-650 Porotherm beams (double beam layout), while specimen F-4 was constructed from 2 pieces of modified F-650 beams, using the same additional horizontal steel bar as in case of F-2 specimens. The clay blocks were cut in half and they were placed to the prefabricated beams. The upper mesh reinforcement was arranged according to the recommendation of the manufacturer, and after that the placing of the in-situ concrete layer was performed. Additional longitudinal and cross reinforcement was applied in the specimens to ensure their integrity during the transportation. Lifting hooks were also placed inside the specimens to help their lifting and moving. Cross-sections of specimens F-1 and F-3 are presented in *Fig. 3*.

The specimens were manufactured by the own mason team of the Wienerberger Ltd. according to the plans of the Department of Structural Engineering *(Fig. 4)*. After the construction of the formwork and assembly of the reinforcement, the concreting

Fig. 4: *Designed cross-sections of F-1 and F-3 floor specimens*



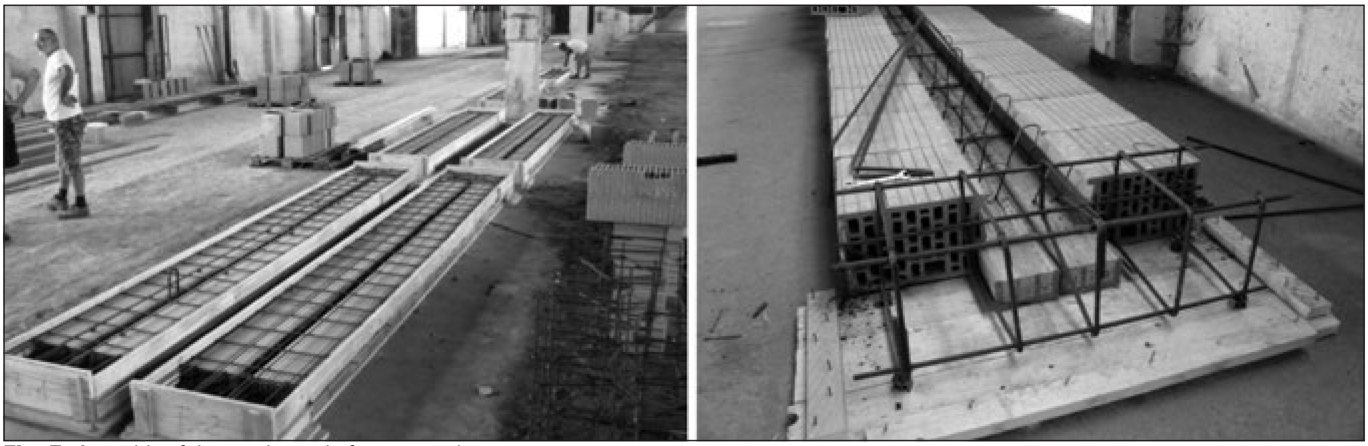


Fig. 5: Assembly of the specimens before concreting

was done using ready-mix concrete with quality certification. During the hardening period of the concrete a regular surface moistening was applied. The specimens were transported to the testing laboratory 28 days after concreting.

2.2 GEOMETRICAL SIZES AND MATERIAL PROPERTIES OF THE SPECIMENS

From each specimen type mentioned before, 3 pieces were manufactured. The most important geometrical sizes of the specimens are listed in Table 1.

For the prefabricated beams ceramic shells of f_k = 40 N/mm^2 strength were used. The clay shells were filled with C30/37-XC3-8-F6 grade concrete. The longitudinal reinforcement of the beams consists of St 180/200 type cold drawn wires with Ø2.5 mm diameter. There are 13 pieces of wires in F-450 beams and 19 pieces of wires in F-650 beams. According to the quality certificate of the wires, their Young's modulus was E_p = 184 kN/mm^2, their characteristic yield strength was $f_{p0,1k}$ = 1220 N/mm^2, and their ultimate strain was ε_{puk} = 40‰. The applied initial prestress of the wires was σ_{p0} = 1359 N/mm^2. The links inside the prefabricated beams were made of BHS 55.50 type smooth steel wires with Ø4.2 mm diameter. The ultimate strength of the links was $f_{u,min}$ = 654 N/mm^2, the Young's modulus was E_s = 200 kN/mm^2, and the characteristic yield strength was $f_{yk,min}$ = 500 N/mm^2. The reinforcement inside the in-situ concrete layer, as well as the supplementary reinforcement was assembled from S500B ribbed steel bars considering the instructions of the manufacturer for the execution. The in-situ concreting was done using C20/25-16-kk-X0 grade ready-mixed concrete. The strength of the concrete was measured by Schmidt-hammer 28 days after concreting. According to the non-destructive measurements, the characteristic concrete strength of the specimens was 25.4 N/mm^2 which corresponds to the planned C20/25 concrete grade.

2.3 THE TEST PROCEDURE

The experiments were carried out at the Structural Laboratory of the Department of Structural Engineering, Budapest University of Technology and Economics (Bódi, Koris, 2013). A loading frame was assembled for the examination of the floor specimens (see *Fig. 6* and *Fig. 7*). Specimens were placed to line supports that allowed the rotation of the beams. The recommended 12 cm minimum bearing length (measured on the prefabricated beams) was used at both ends of the structure, so the span of the floor was 4.375 m in case of the shorter F-1 and F-2 specimens, and 6.375 m in case of the longer F-3 and F-4 specimens. The floor specimens were loaded by uniformly distributed load using an 8.00 m long and 1.00 m wide fibre reinforced pressure blanket. The pressure blanket was supported by 90 cm wide framed formwork boards on the upper side, which were fixed to the cantilever beams of the loading frame (see *Fig. 7*). The lifting hooks were cut and removed so the pressure blanket could be properly positioned. The blanket was filled by air using a compressor, and the air pressure was measured by a pressure gauge. The deflection of the specimens was measured by inductive displacement pick-ups, which were placed in the middle and in the quarters of the span.

The pressure and displacement values were measured at 5 Hz sampling frequency using a 16-channel HBM Spider-8 electronic measuring system. The air pressure was gradually increased until failure of the specimens with a speed of ~$5\cdot10^{-4}$ bar/sec (0.05 kN/m^2/sec). After the failure of the floor specimen it was unloaded by slowly releasing the air from the pressure blanket. The places and external signs of damages were recorded after removing the blanket from the top of the specimen. The failure of the floors typically occurred at the pressure range of 0.09÷0.15 bar.

Some pictures showing the testing of an F-1 type floor specimen are presented in *Fig. 8.*

Table 1: Geometrical sizes of the specimens

Type of specimen	Number of manufactured specimens [piece]	Used Porotherm members		Geometrical sizes			
		Prefabricated beam	Clay block	Length [m]	Width [cm]	Height [cm]	In-situ concrete [cm]
F-1	3	1 piece of F-450	PTH 60/17	4.86	60	21	4
F-2	3	1 piece of F-450	PTH 60/17	4.86	60	21	4
F-3	3	2 pieces of F-650	PTH 60/17	6.86	72	24	7
F-4	3	2 pieces of F-650	PTH 60/17	6.86	72	24	7

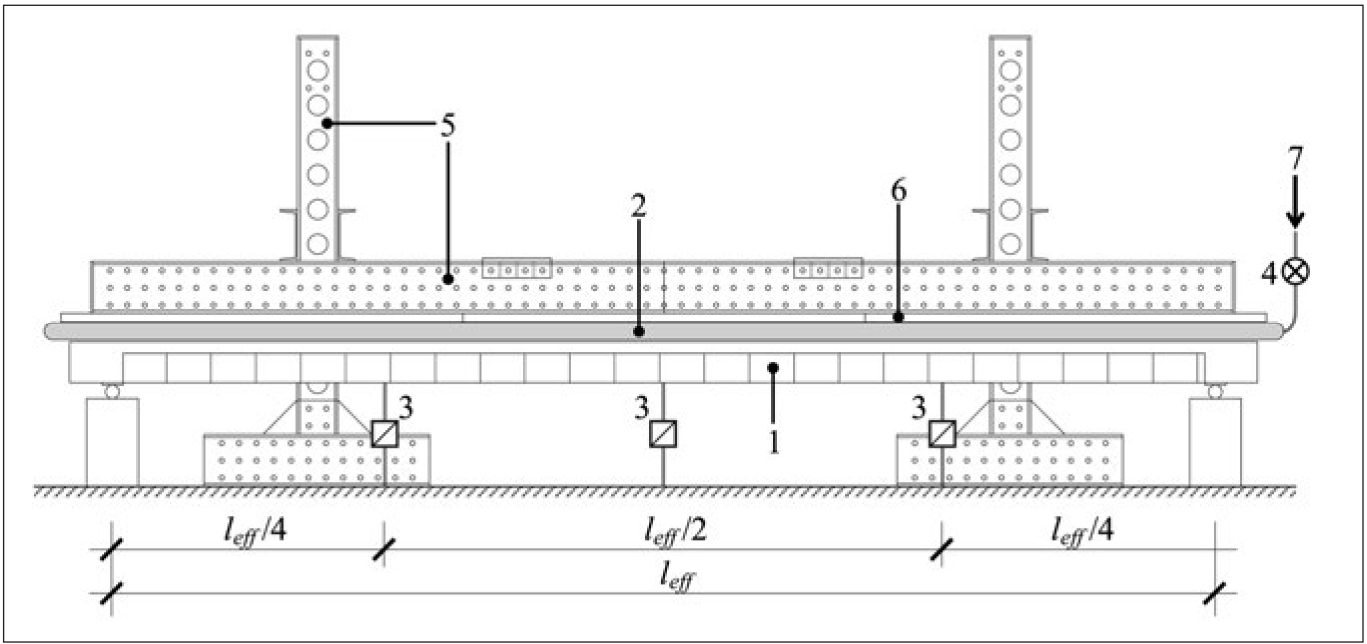


Fig. 6: Front view of the test arrangement (1 – Porotherm floor specimen; 2 – pressure blanket for the loading of the specimen; 3 – inductive displacement pick-up; 4 – pressure gauge; 5 – loading frame; 6 – counter formwork; 7 – compressed air supply)

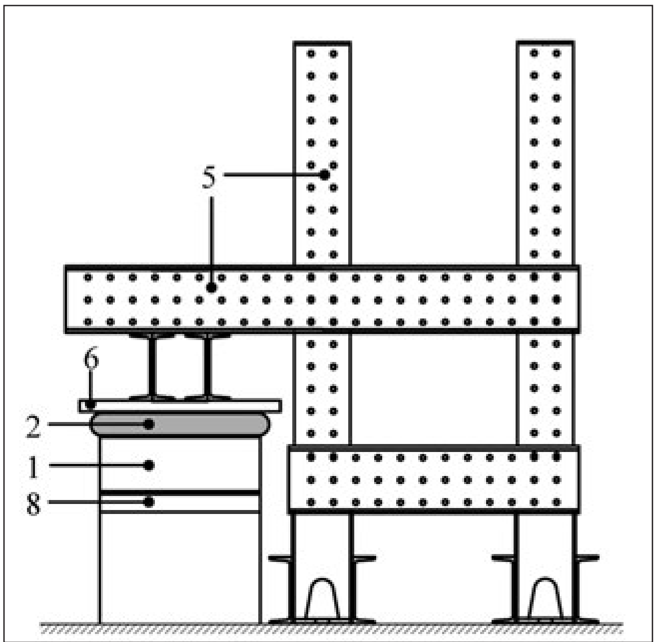


Fig. 7: Side view of the test arrangement (1 – Porotherm floor specimen; 2 – pressure blanket; 5 – loading frame; 6 – counter formwork; 8 – support)

3. RESULTS OF THE EXPERIMENTS

The most important results of the experiments are presented in *Table 2.* The mean value and standard deviation of load carrying capacity was calculated for each specimen type separately. Using the calculated stochastic parameters the 1‰ threshold value of the capacity was also determined, since it matches the level of safety that belongs in general to the design value of the structural resistance defined by Eurocode. The 1‰ threshold value could be compared to the calculated load carrying capacity values.

The load-deflection curves of the shorter (F-1 and F-2 type) specimens are displayed in *Fig. 9.* The plotted deflection values refer to the results measured in the middle of the span.

In *Table 3.* the most important test results on the shear capacity are compared to the results of calculation. In case of each specimen type the 1‰ threshold value of the shear resistance was calculated from the measured maximum loads. These threshold values represent the "measured" shear capacity in the 4th column of the table. The shear capacities that were calculated according to EC2 from the shear resistance of the concrete without shear reinforcement ($V_{Rd,c}$) are displayed in the 5th column. The prefabricated Porotherm beams contain some links – typically near to the end of the beam – which must be bent up in 45° during construction according to the recommendation of the manufacturer. The amount of this shear reinforcement is, however, smaller than the amount of necessary minimum shear reinforcement defined by EC2 so they cannot be considered during the design. The inadequacy of the shear reinforcement was also proven by the experiments, since the shear capacity of the reinforcement ($V_{Rd,cs}$) calculated according to EC2 was higher than the measured capacity. The calculated $V_{Rd,cs}$ values are presented in the 6th column of the table. In case of the F-2 and F-4 type specimens there was a modified reinforcement on one side of the beam, and an additional horizontal steel bar on the other side of it. By the application of this extra longitudinal reinforcement, we were able to consider the additional shear capacity of the concrete that can be calculated from the arch-effect (Draskóczy, 2009, Kollár, Dulácska, 2009, Kollár, Ther, 2011, *fib* Bulletin 65-66, 2012). These shear resistance values are displayed in the 7th column of the table. It can be seen, that the consideration of the arch-effect delivers higher shear capacities than the standard EC2 calculation procedure, however, they still remain under the measured values with an appropriate level of safety. The modified shear reinforcement inside F-2 and F-4 type specimens was not bent up into the desired position, but they remain horizontal, so the measured carrying capacities really reflect the shear resistance of concrete.

The $V_{Rd,cs}$ values in the table were presented just for illustrational purposes, but as it was mentioned before, they were not considered during the calculation and assembly of the design tables for the Porotherm floor beams. In the new design tables the increased shear capacity of the concrete due to arch-effect was considered, but this implies the application of the additional horizontal reinforcement at the ends of the prefabricated beam. The consideration of improved shear capacity resulted of course in increased load carrying capacity values in the design tables. The calculated values were justified by the results of the carried out experiments. In *Fig. 10* the

Fig. 8: *Testing of a shorter (F-1 type) specimen in the Structural Laboratory*

carrying capacities from different design tables are compared again for the same Porotherm floor structure (single layout beams, 45 cm axis distance and 4 cm in-situ reinforced concrete layer) that was presented in *Fig. 2.* It can be seen that the calculated – and experimentally verified – new carrying capacities are significantly higher than the previous values, providing appropriate applicability and competitiveness to the floor system on the market.

4. CONCLUSIONS

The Department of Structural Engineering at the Budapest University of Technology and Economics was collaborating with the industrial partner Wienerberger Ltd. to perform a series of tests on Porotherm floor specimens to determine the realistic shear capacity of the system. The amount of shear reinforcement inside the prefabricated Porotherm beams is smaller than the minimum amount defined by EC2 so it cannot be considered during the design procedure. The experimental results justified this neglect, since the consideration of the available shear reinforcement would lead to shear resistance values, which are not actually available under realistic conditions. On the other hand, the tests showed that the shear resistance of the concrete cross-section is higher than the $V_{Rd,c}$ value calculated according to EC2. An appropriate method for the justification of this improved shear behaviour is the consideration of the arch-effect at the ends of the floor beams. We have compared the measured shear capacity values to the

Table 2: *Results of the experiments of Porotherm floor specimens*

Specimen	Test results		Load carrying capacity				
	Max. load p_{max} [kN/m^2]	Max. deflection e_{max} [mm]	Mean value p_m [kN/m^2]	Standard deviation s_p [kN/m^2]	Standard deviation s_p [%]	5% threshold p_k [kN/m^2]	1‰ threshold p_d [kN/m^2]
F-1/1	10.10	67.4	10.4	1.0	10.1	8.70	7.19
F-1/2	9.58	97.7					
F-1/3	11.60	78.5					
F-2/1	11.86	52.3	11.9	1.2	10.2	9.93	8.18
F-2/2	10.74	69.1					
F-2/3	13.16	66.5					
F-3/1	14.70	91.1	13.3	1.4	10.8	10.90	8.83
F-3/2	13.26	84.3					
F-3/3	11.83	97.1					
F-4/1	11.76	77.3	13.0	1.2	9.5	10.96	9.17
F-4/2	13.01	122.9					
F-4/3	14.24	134.4					

Table 3: *Measured and calculated shear resistances of Porotherm floor specimens*

Type of specimen	Porotherm beam	Reinforcement	Shear capacity [kN]			
			Result of experiment (1‰ threshold)	Resistance of the concrete ($V_{Rd,c}$)	Resistance of the shear reinforcement ($V_{Rd,cs}$)	Resistance of the concrete considering the arch-effect (where possible)
F-1	1 piece of F-450	factory arrangement	13.10	7.10	14.55	7.10
F-2	1 piece of F-450	modified link arrangement and additional horizontal steel bars	14.29		-	11.28
F-3	2 pieces of F-650	factory arrangement	29.98	17.75	33.94	17.75
F-4	2 pieces of F-650	modified link arrangement and additional horizontal steel bars	30.71		-	26.96

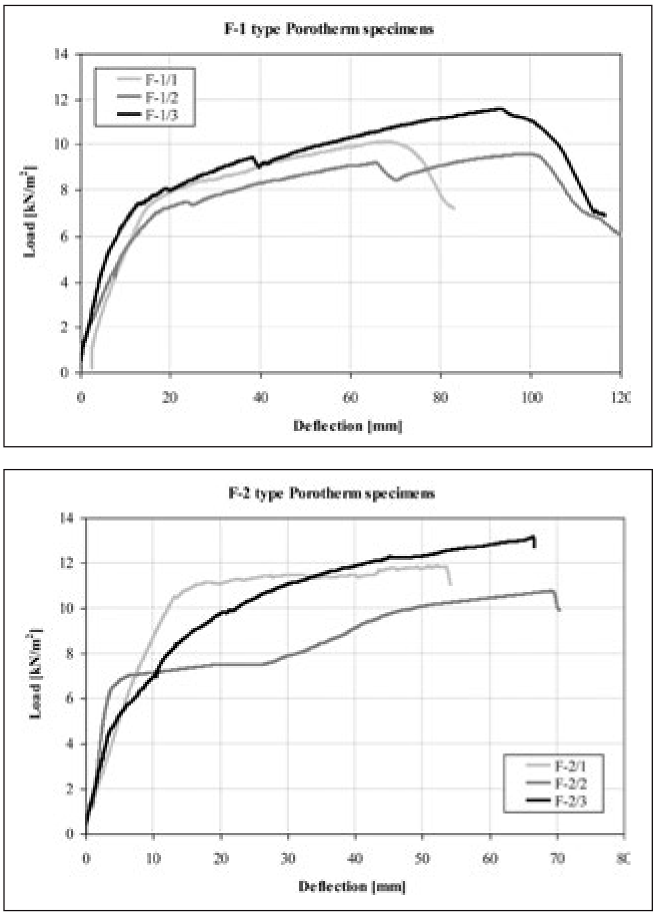


Fig. 9: *Measured load-deflection curves of F-1 and F-2 type specimens*

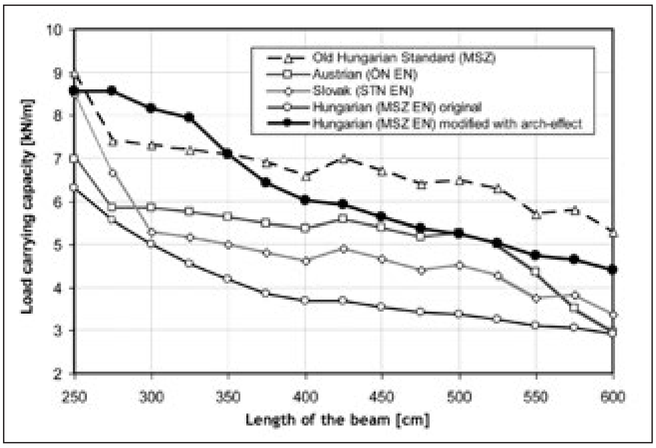


Fig. 10: *The load carrying capacity of the Porotherm floor system according to different design tables (single beam layout, 45 cm axis distance, 4 cm in-situ reinforced concrete layer on the top of clay bricks)*

results of calculation. The consideration of arch-effect delivers results that are closer to the test experiences with an appropriate margin of safety. According to the results of the experiments and calculations new design tables were assembled for the Porotherm floor system. The new load carrying capacities given in these tables reflect the consideration of improved shear resistance of the floor beams, providing more than 50% increase compared to the previous carrying capacity values.

5. REFERENCES

Bódi, I., Koris, K. (2012) „Load carrying capacity of Porotherm floor beams according to MSZ EN 1992-1-1" (in Hungarian) Research report of the Budapest University of Technology and Economics, Department of Structural Engineering.

Bódi, I., Koris, K. (2012) „Load distribution effect of the Porotherm floor ribs using PTH 45/10 and PTH 60/10 clay blocks" (in Hungarian) Research report of the Budapest University of Technology and Economics, Department of Structural Engineering.

Bódi, I., Koris, K. (2013) „Experimental analysis of the shear capacity of Wienerberger Porotherm floor system" (in Hungarian) Research report of the Budapest University of Technology and Economics, Department of Structural Engineering.

Draskóczy, A. (2009), „Is it possible to increase the shear capacity during the design of reinforced concrete beams?" (in Hungarian) *Vasbetonépítés* 2009/3, pp. 70-77.

Draskóczy, A. (2009), „Shear tests evaluation and numerical modelling of shear behaviour of reinforced concrete beams" *Periodica Polytechnica* 40/1, pp. 9-17.

ÉMI (2009) „Fire resistance analysis of Porotherm floor" (in Hungarian), ÉMi K-1/2009 Research Report.

fib Bulletin 65 (2012 April) „Model Code 2010" Final Draft Vol. I.

fib Bulletin 66 (2012 March) „Model Code 2010" Final Draft Vol. II.

IBS (2009) „Tragende Ziegeldecke mit einer an der Unterseite versehenen Putzoberflache und Aufbeton der Oberseite", Prüfbericht IBS Brandschutztechnik und Sicherheitsforschung GmbH, Linz.

Kollár, L., Dulácska, E. (2009) „Considering the arch-effect in reinforced concrete beams according to Eurocode 2" (in Hungarian) *Vasbetonépítés* 2009/3, pp. 78-82.

Kollár, L., Ther, T. (2011) „The arch-effect in reinforced concrete beams" (in Hungarian) *Vasbetonépítés* 2011/3, pp. 69-71.

Linder, J. (2004) „Schub- und Biegeversuche an Porotherm-Deckenelementen – Dokumentation der Versuchsergebnisse" *Konstruktive Versuchsanstalt*, Bautechnikzentrum der TU-Graz, Graz.

Wienerberger (2013) „Application and Design Guide" (in Hungarian).

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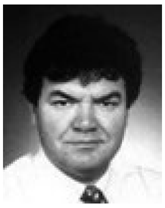
LATERAL TORSIONAL STABILITY ANALYSIS OF PRECAST CONCRETE HANGING BEAMS



Zsolt Antal Kovács – István Bódi

The condition of hanging from ropes is the most critical state of the precast concrete beams considering the lateral torsional buckling phenomenon. Calculation methods, handling this problem, can be found in publications and design guides (see References). The methods differ from each other in the number of the considered stiffnesses and rope arrangement parameters. These calculation modes mainly take perfect initial geometry and linear elastic material properties into account. A design formula is not recommended directly by the European standard (Eurocode), but some basic rules are determined for the analysis. Differences were found between the results of the methods considering various number of stiffnesses. In addition to the former, difference also appears with the results of the recommended design formula used in the practice, notably for the harm of safety.

Keywords: Critical loads, hanging beams, lateral stability, lateral torsional buckling, stability failure

1. INTRODUCTION

The lateral torsional buckling phenomenon typically appears in the behaviour of steel elements due to their slenderness. With the development of the bridgeable span, the cross-section of the concrete beams also becomes slender comparing to their length. This results that the stability analysis of the precast concrete girder beams takes more importance in addition to the strength resistance.

The position of hanging from ropes is one of the most critical states of the beams, because the end sections can rotate freely around the axis determined by the points of the rope connections. After the buckling of the compressed flange of the beam in the horizontal plane, the centre of gravity of the whole element moves sidewalls, which causes the rotation of the beam as a rigid body (*Fig. 1*). As a consequence, strength failure can occur due to the bending moment around the minor axis of the cross-section.

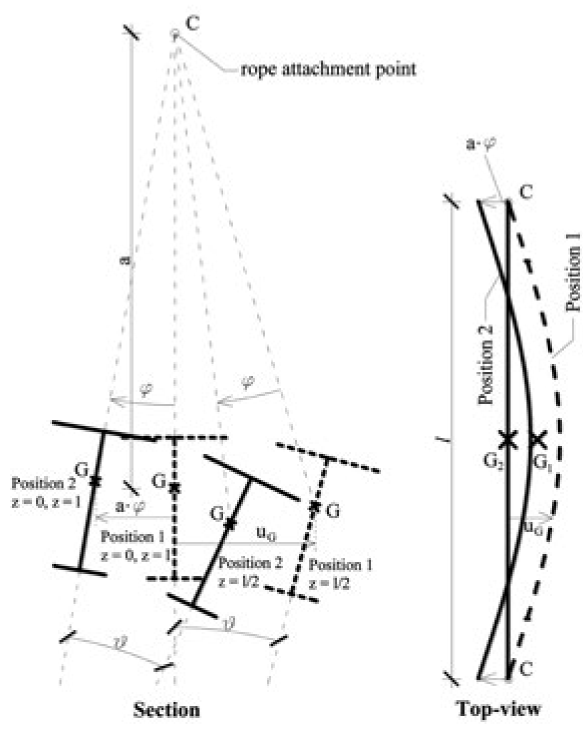
The main goal of this paper is to give a summary about the calculation methods found in publications and design guides dealing with this mentioned phenomenon and to make comparisons with their results by the calculation of a possible large span cross-section.

The methods can be compared according to the number of stiffnesses and rope arrangement parameters taken into account. Differences can be made between the formulas on the basis of their complexity and usability.

It is also important to determine the assumptions that are complied through the examinations. The cross-section of the beam is symmetrical around the minor-axis, thin walled and constant through the length. The rope can be in a vertical or inclined position and supports the element at the end points or at a distance from the ends. The lifting structure is rigid without any elastic deformations and the ropes bear only axial tension forces.

In the equilibrium and energy equations of the beam we use the *Theory of Small Deflections*. According to (Rafla, 1968) the prestressing tendons are equivalent with the reinforcement in the point of this lateral torsional buckling phenomenon, so the prestressing tendons are calculated in the same way as the reinforcement in the cross-sectional properties.

Fig. 1: The deformations of the beam during the lateral torsional stability failure



2. THE BASIC EQUATIONS OF THE STABILITY PHENOMENON

The basic equations can be composed according to the equilibrium and the energy method. *Fig. 2.* shows the basic geometrical notations for the case of the beam when hanging from ropes. Before the composition of the equations some neglectations and simplifications should be made. The deformation of the beam in the vertical plane from the self-weight load is negligible, so the two considered deformation components are the displacement in the horizontal plane *u(z)* and the torsion of the cross-section around the fore-axis *ϑ(z)*. Besides the vertical forces acting at the rope attachment points a normal force is also available whether the ropes are inclined. The only load causing these forces is the self-weight of the beam.

First the equilibrium equations are presented. Equation (1) describes the equilibrium of the forces parallel with the x axis. In Equation (2) the basic condition is that the torsion moment along the fore-axis of the beam should be zero. These equations are written according to (Petersen, 1982).

$$EI_y u'''' + Pu'' + P(a - y_T)\vartheta'' + \left(M_q\vartheta\right)'' = 0 \qquad (1)$$

$$EI_w\vartheta'''' - GI_t\vartheta'' + P(a - y_T)u'' + M_q u'' + P[i_P^2 + y_T^2 + a(r_x - 2y_T)]\vartheta'' + (r_x - 2y_T)\left(M_q\vartheta'\right)' + qy_T\vartheta = 0 \qquad (2)$$

In *Fig. 2 G* is the center of gravity and *T* is the shear center of the cross-section. The point of rope attachment is denoted by *C*. The geometrical measurements can be read from *Fig. 2*. The uniformly distributing load q equals to the self-weight of the beam. The normal force that acts parallel with the fore-axis is the consequence of the self-weight, so $P=q\cdot(1/2)\cdot\cot(\alpha)$. M_q is the bending moment in the *y-z* plane only from q.

The unknown parameters in these equations are the *u(z)* displacement in the horizontal plane and the *ϑ(z)* rotation.

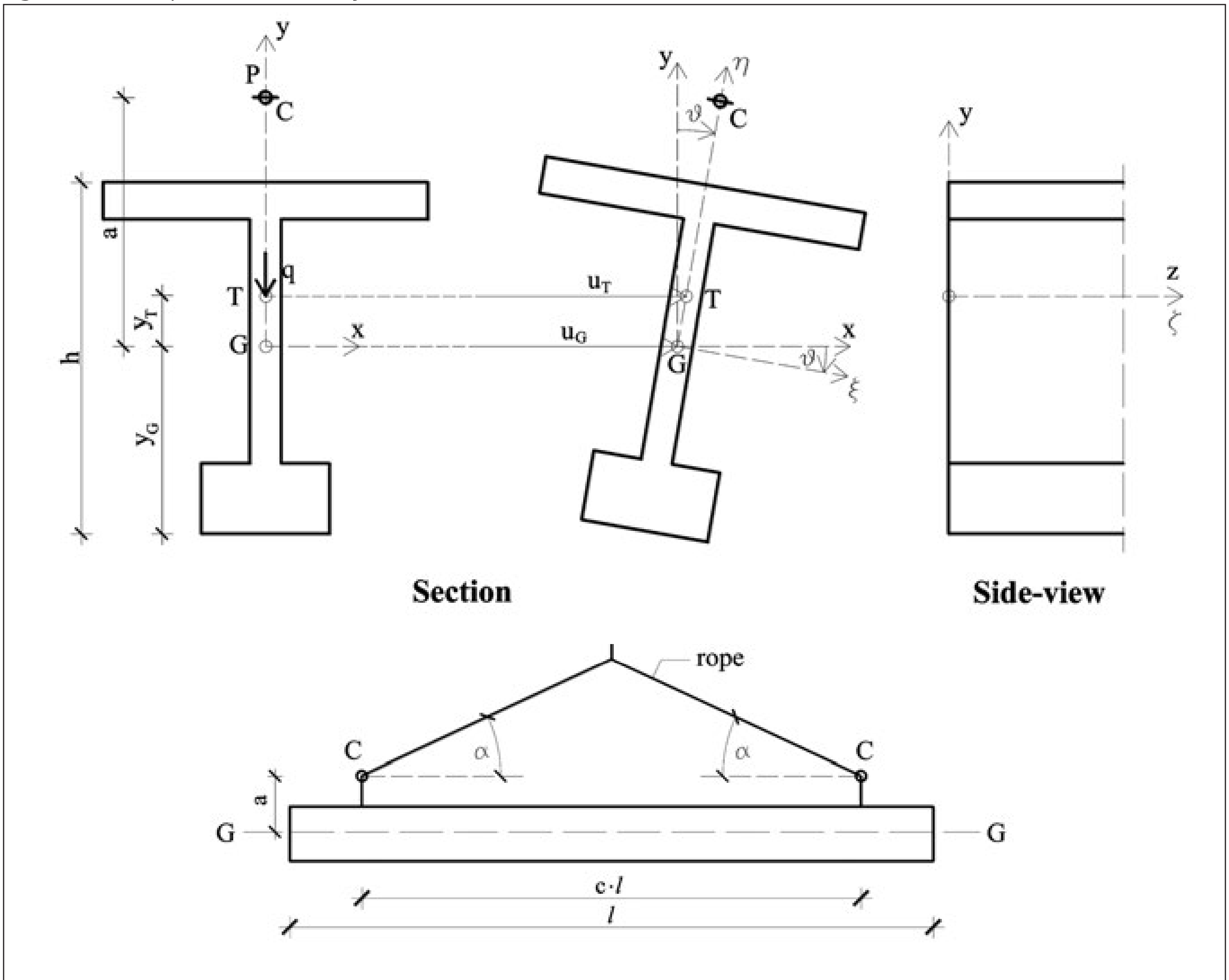
The energy equation for the total potential energy (3) of the system can be formed in the following way according to (Rafla, 1968) and (Petersen, 1982):

$$\Pi = \int_0^l \frac{1}{2}\{ EI_w\vartheta''^2 + \left[GI_t - P(i_P^2 + y_T^2) - Pa(r_x - 2y_T) - (r_x - 2y_T)M_q\right]\vartheta'^2 + 2P(a - y_T)u''\vartheta + 2M_q u''\vartheta + EI_y u''^2 - Pu'^2 - qy_T\vartheta^2\}dz - \frac{1}{2}qlf\varphi^2 \qquad (3)$$

The solution for the u and ϑ unknowns from the equilibrium equations can be found by substituting approximate functions to the place of the *u(z)* and *ϑ(z)* parameters and ordering suitable boundary conditions to be able to solve the equation system.

Solution for the *u(z)* and *ϑ(z)* parameters from the potential energy equation can be found by using the theory of the stacionarity of potential energy. To simplify the calculation

Fig. 2: Geometrical parameters of the analysed beam



the approximate functions for the *u(z)* displacement and *ϑ(z)* rotation can be used here as well.

The approximate functions for *u(z)* and *ϑ(z)* can be trigonometrical or polinomial formulas.

3. CALCULATION METHODS

In this paragraph five different calculation methods are described which are originating from publications and design guides. They mainly use trigonometrical approximate *u(z)* and *ϑ(z)* functions. For the solution of the p_{cr} critical force causing the stability failure the Rafla (1968) method uses for example the theory of the stacionarity of the potential energy, and the Dulácska (1985) method uses for example the equilibrium equations.

For the analysis of the usability of the methods four different lifting cases are taken into account (see *Fig. 2.*):

1st lifting case: Vertical ropes at the ends of the beam. Inconstant parameter: *a*

2nd lifting case: Inclined ropes at the ends of the beam. Inconstant parameters: *a*, α

3rd lifting case: Vertical ropes connected at a distance from the ends of the beam. Inconstant parameters: *a*, *c*

4th lifting case: Inclined ropes connected at a distance from the ends of the beam. Inconstant parameters: *a*, α, *c*

3.1 RAFLA (1968) METHOD

The calculation method written by Rafla (1968) in (Rafla, 1968) is suitable for cross-sections symmetrical about the minor axis. This method takes the EI_y, EI_x bending stiffness, the GI_t Saint-Venant torsional stiffness and the EI_w warping stiffness into account. The 1st and the 2nd lifting cases can be considered by this formula.

The basis of this method is to use the following approximate functions for the *u(z)* and ϑ(z) unknowns:

$$u(z) = la_1 \sin\frac{\pi z}{l} + y_T c_1 \sin\frac{\pi z}{l} \tag{4}$$

$$\vartheta(z) = c_1 \sin\frac{\pi z}{l} \tag{5}$$

With the substitution of Equation (4) and (5) into the formula of the total potential energy and by using the theory of the stacionarity of the potential energy, solution can be found for the a_1 and c_1 unknowns. For the use of the theory we should solve the following equations:

$$\frac{\partial \Pi}{\partial u} = 0 \tag{6}$$

$$\frac{\partial \Pi}{\partial \vartheta} = 0 \tag{7}$$

Finally Rafla (1968) gave a closed second order equation (8) in (Rafla, 1968) for the p_{cr} critical uniformly distributing force:

$$-24{,}352 \cdot P_{cr}^2\left(i_p^2 + r_x a - a^2\right) - P_{cr} p_{cr} l^2 \left(1{,}412 \cdot r_x \cdot 5{,}292 \cdot a + 2\frac{i_p^2}{a} + 2r_x\right) + p_{cr}^2 l^4 \left(0{,}287 - 0{,}232\frac{r_x}{2a}\right) + 24{,}352 \cdot P_{cr}\left(GI_t + \frac{9{,}870}{l^2}\left[EI_w + EI_y\left(y_T^2 + i_p^2 + r_x a - 2y_T a\right)\right]\right) + p_{cr} EI_y \left(13{,}940 \cdot r_x - 52{,}231 \cdot y_T + \frac{19{,}739}{a} y_T^2 + \frac{19{,}739}{a}\frac{EI_w}{EI_y} + \frac{2l^2}{a}\frac{GI_t}{EI_y}\right) - \left(2372{,}13\frac{EI_w EI_y}{l^4} + 240{,}35\frac{GI_t EI_y}{l^2}\right) = 0 \tag{8}$$

where:

$$P_{cr} = p_{cr}\frac{l}{2}\cot\alpha \tag{9}$$

3.2 KORDA (1965) METHOD

Korda (1965) has written a design guide (Korda, 1965) for the calculation of the stability of precast concrete beams during their lifting process. From the formulas found in this guide two were chosen which are suitable for the 3rd and 4th lifting cases, and consider rectangular cross-sections primarily. The solutions take only the EI_y bending stiffness into account.

The formula for the calculation of the 3rd lifting case:

$$p_{cr} = \frac{960}{3 - 3c^2 + 40c^5 - 5c^4} \cdot \frac{EI_y}{l^4} a \tag{10}$$

The formula for the calculation of the 4th lifting case:

$$p_{cr} = \frac{16EI_y a}{l^4} \cdot \frac{(1+2c)u^5}{(1 - 4c^2u^2 + 4c^4u^4)\tan u - u + (4c^2 - 0{,}333)u^3 + 1{,}6c^5u^5} \tag{11}$$

where:

$$N = p_{cr}\frac{l}{2}\cot\alpha \tag{12}$$

$$u = \frac{l}{2}\sqrt{\frac{N}{EI_y}} \tag{13}$$

3.3 DULÁCSKA (1985) METHOD

The Dulácska (1985) calculation method in (Dulácska, 1985) is suitable primarily for rectangular cross-sections and takes only the EI_y bending stiffness into account. The formula can be used for the calculation of the 1st and the 2nd lifting cases.

The equation for the calculation of the p_{cr} critical uniformly distributing force:

$$p_{cr} = g_E \cdot \frac{1}{1 + \frac{6a}{l\tan\alpha}} \tag{14}$$

where the g_E critical self-weight by Euler is:

$$g_E = 120\frac{EI_y}{l^4} a \tag{15}$$

3.4 SBT. DUNK (1999) METHOD

The basis of this calculation method is found in (Stratford, Burgoyne, Taylor, 1999), where the written formula is usable for the 1st and the 3rd lifting cases. The method is suitable primarily for rectangular cross-sections and takes only the EI_y bending stiffness into account.

A method suitable for all the four lifting cases can be composed by using the *Dunkerley's formula* (Kollár, 1991). In the followings this modified formula is denoted by SBT. Dunk (1999).

The equation found in (Stratford, Burgoyne, Taylor, 1999) is the following for the calculation of the 1st and the 3rd lifting cases:

$$p_{cr.SBT} = \frac{12EI_y a}{\frac{l^4}{10} - \frac{l^4(1-c)}{2} + 3\frac{l^4(1-c)^2}{4} - \frac{l^4(1-c)^3}{4} - \frac{l^4(1-c)^4}{16}} \quad (16)$$

The term considering the normal force from the effect of the inclined ropes is:

$$p_{cr.N} = \frac{2N_E}{l \cot\alpha} \quad (17)$$

where the N_E critical normal force by Euler is:

$$N_E = \frac{\pi^2 EI_y}{(c \cdot l)^2} \quad (18)$$

The value of the p_{cr} critical uniformly distributing force for the whole structure with the use of the *Dunkerley's formula* is the following:

$$p_{cr} = \frac{1}{\frac{1}{p_{cr.SBT}} + \frac{1}{p_{cr.N}}} \quad (19)$$

3.5 RECOMMENDED (1989) METHOD

The calculation method used in the Hungarian practice can be found in (Massányi, Dulácska, 1989). This method is suitable primarily for solid rectangular cross-sections where the EI_w warping stiffness is zero. The formula takes only the EI_y bending stiffness into account. All the four lifting cases can be calculated by this method whether the a distance of the rope attachment point and the centre of gravity of the cross-section is less than or equal to the h height of the cross-section ($a \leq h$). The beam can rotate freely around the axis determined by the rope attachment points.

In the MSZ EN 1992-1-1 the paragraph 5.9 is about the stability analysis of slender precast beams. In this section no exact calculation formula is composed, only the importance of this analysis and the consideration of an *l/300* (*l* is the total length of the beam) initial geometrical imperfection is mentioned. Mainly this is the reason why the Hungarian engineers use the formula in (Massányi, Dulácska, 1989). In the followings this method is denoted by Recommended (1989).

The equation for the p_{cr} critical uniformly distributing force by the Recommended (1989) method is:

$$p_{cr} = k_{31} k_{32} \frac{120 EI_y a}{l^4} \quad (20)$$

where k_{31} can be get from *Table 1* by linear interpolation.

Table 1: k_{31} values

(1-c)/2	0	0.1	0.2	0.225	0.3	0,4	0,5
$\mathbf{k_{31}}$	1.0	3.58	38.5	66.6	13.7	7.35	2.66

The k_{32} parameter can be calculated after the determination of k_{31}, by Equation (21):

$$k_{32} = \frac{1}{1 + \frac{k_{31} a (c \cdot l)^2}{2l^3 \tan\alpha}} \quad (21)$$

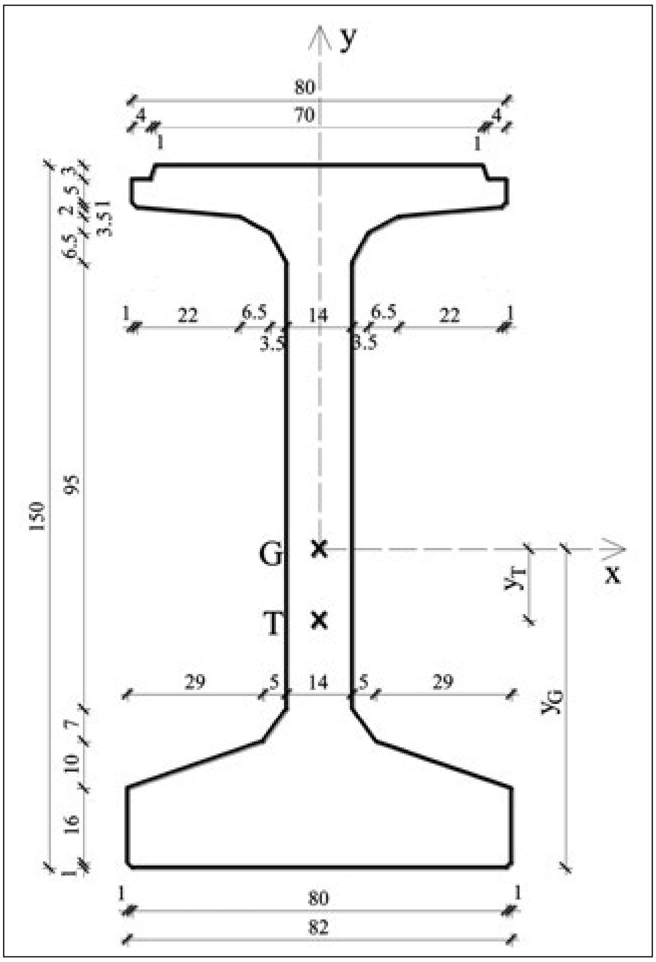


Fig. 3: The geometrical properties of the analysed cross-section

4. COMPARISON ON THE BASIS OF THE CONSIDERED PARAMETERS

The following calculations (*Figs. 4-10*) show the results of the ν_{cr} safety level for the cross-section shown on *Fig. 3.* This beam is a *44.8 m* long bridge girder for the construction of prefabricated orthotropic bridges mainly above highways. The geometrical properties are shown on *Fig. 3.*

The calculations are based on the characteristic values of the material properties and independent from any standard. The beam is assumed to be geometrical perfect without any initial imperfections and its material follows linear-elastic properties. The ν_{cr} = *1.00* is the critical case, if the safety level decreases below *1.00*, the stability failure occurs according to the calculation.

Table 2 shows results for the ν_{cr} safety level values belonging to the examined cross-section (*Fig. 3.*) in the four different lifting cases.

In the 1st and 2nd lifting cases the difference between the results from all of the considered methods is only 41%. In the 3rd lifting case the Korda (1965) method approximates for the favour of safety with 18%, the SBT.Dunk (1999) method for the harm of safety with 432% and the Recommended (1989) method also for the harm of safety with 1060% compared to the Rafla (1968) calculation method. In the 4th lifting case the approximation of the Korda (1965) method differs with 64% for the favour of safety, the SBT. Dunk (1999) method with 296% for the harm of safety and the Recommended (1989) method with 1173% also for the harm of safety compared to the Rafla (1968) calculation method.

Table 2: Results for the ν_{cr} safety level values belonging to the examined cross-section (Fig. 3)

Safety level considering the stability failure (ν_{cr} [-])				
Parameters	Basic values			
a =	h-y_G	h-y_G	h-y_G	h-y_G
α =	90°	30°	90°	30°
c =	0.99	0.99	0.75	0.75
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	1.14	0.95	1.26	1.03
Korda (1965)	-	-	1.03	0.37
Dulácska (1985)	1.19	0.99	-	-
SBT.Dunk (1999)	1.25	1.03	6.70	4.08
Recommended (1989)	1.34	1.32	14.62	13.11

Fig. 4 presents the relation between the ν_{cr} safety level and the *a* distance of the rope attachment point from the centroid of the beam. The main conclusion of the graphs is that the ν_{cr} safety level increases with the larger *a* distances. The *a* distance is presented in its relation to the distance between the centre of gravity and the top of the cross-section [$a/(h-y_G)$]. The relation between the two parameters in many cases is linear, except for the Rafla (1968) method. The methods give closely equal safety level results for the different *a* values.

According to the graphs the ν_{cr} value is less than or equal to *1.00* in the case when the $a/(h-y_G)$ relation is around and less than *0.80*.

Fig. 5 shows the relation between the ν_{cr} safety level and the α angle closed by the rope and a horizontal line. With the increase of the α angle the ν_{cr} safety level is strictly monotonously increasing. This gain of ν_{cr} is notable at the smaller α angles (*0°-20°*). After *45°* the shape of the graphs get nearly horizontal, so the change in the α angle has less effect on the value of the safety level. The relation between the two parameters is non-linear.

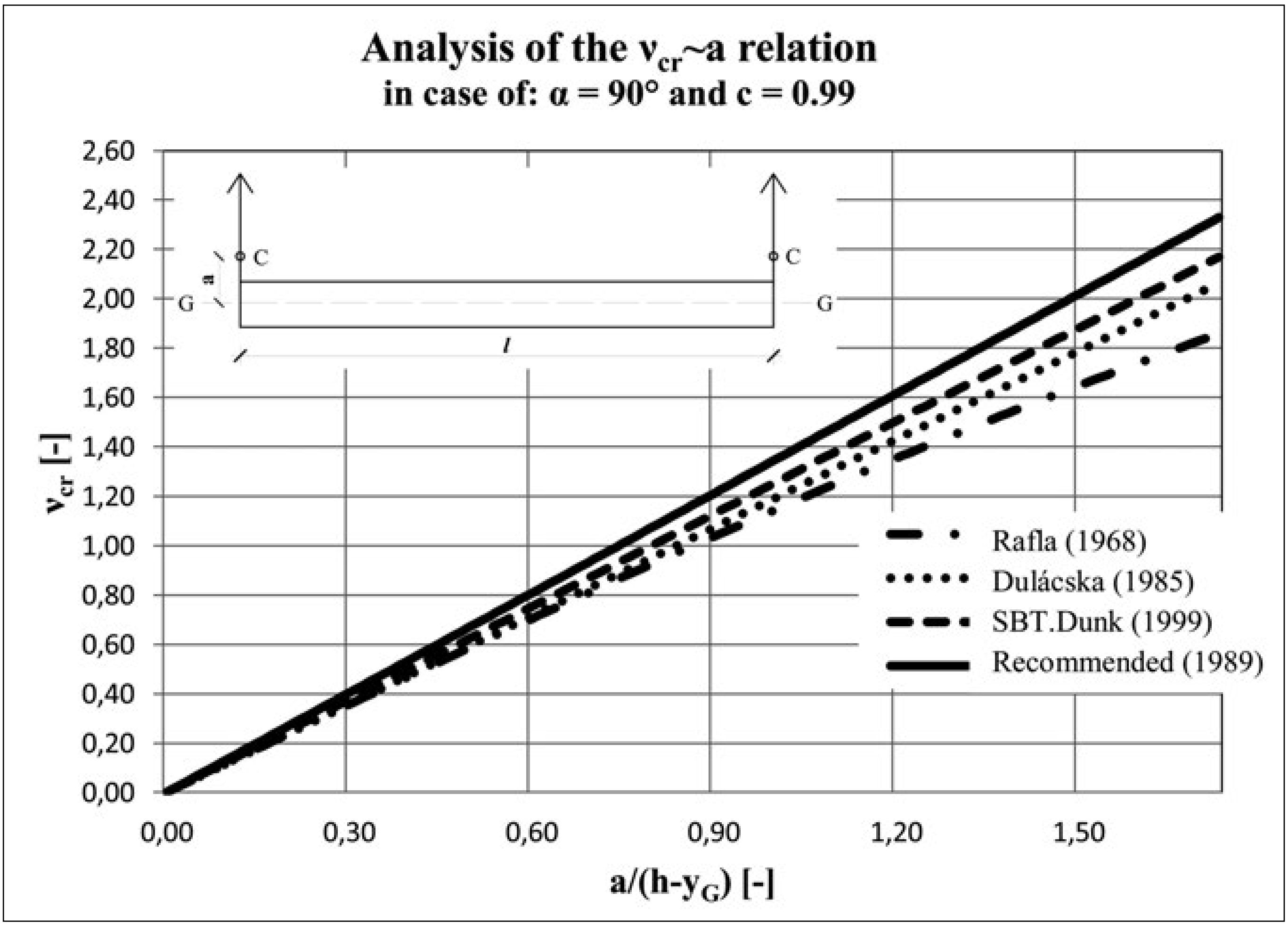
Excluding the Recommended (1989) method the other graphs are presenting nearly the same safety level values for the different α angles. The Recommended (1989) method calculates for the harm of safety.

Around α = 30° the methods give around *1.00* safety level value, except for the Recommended (1989) calculation method which result is around $\nu_{cr} = 1.30$.

Fig. 6 presents the relation between the ν_{cr} safety level and the c parameter, which determines the length of the consoles at the ends of the beam in the case when the angle closed by the rope and a horizontal line is α = *90°*. Around *c = 0.55* the ν_{cr} value reaches its maximum according to the SBT.Dunk (1999) and the Recommended (1989) methods. At most of the *c* values the Recommended (1989) method approximates for the harm of safety comparing to the SBT.Dunk (1999) graph. The Korda (1965) formula is for the favour of safety on the whole *c* region.

Fig. 7 shows the relation between the ν_{cr} safety level and the *c* parameter, which determines the length of the consoles at the ends of the beam, but now in the case when the angle of the rope closed by a horizontal line is α = *30°*. All the three graphs show the same characteristic, they present maximum ν_{cr} values around the *c = 0.55* parameter. Comparing to

Fig. 4: Analysis of the ν_{cr}~*a* relation in case of: **α = 90°** and *c = 0.99*



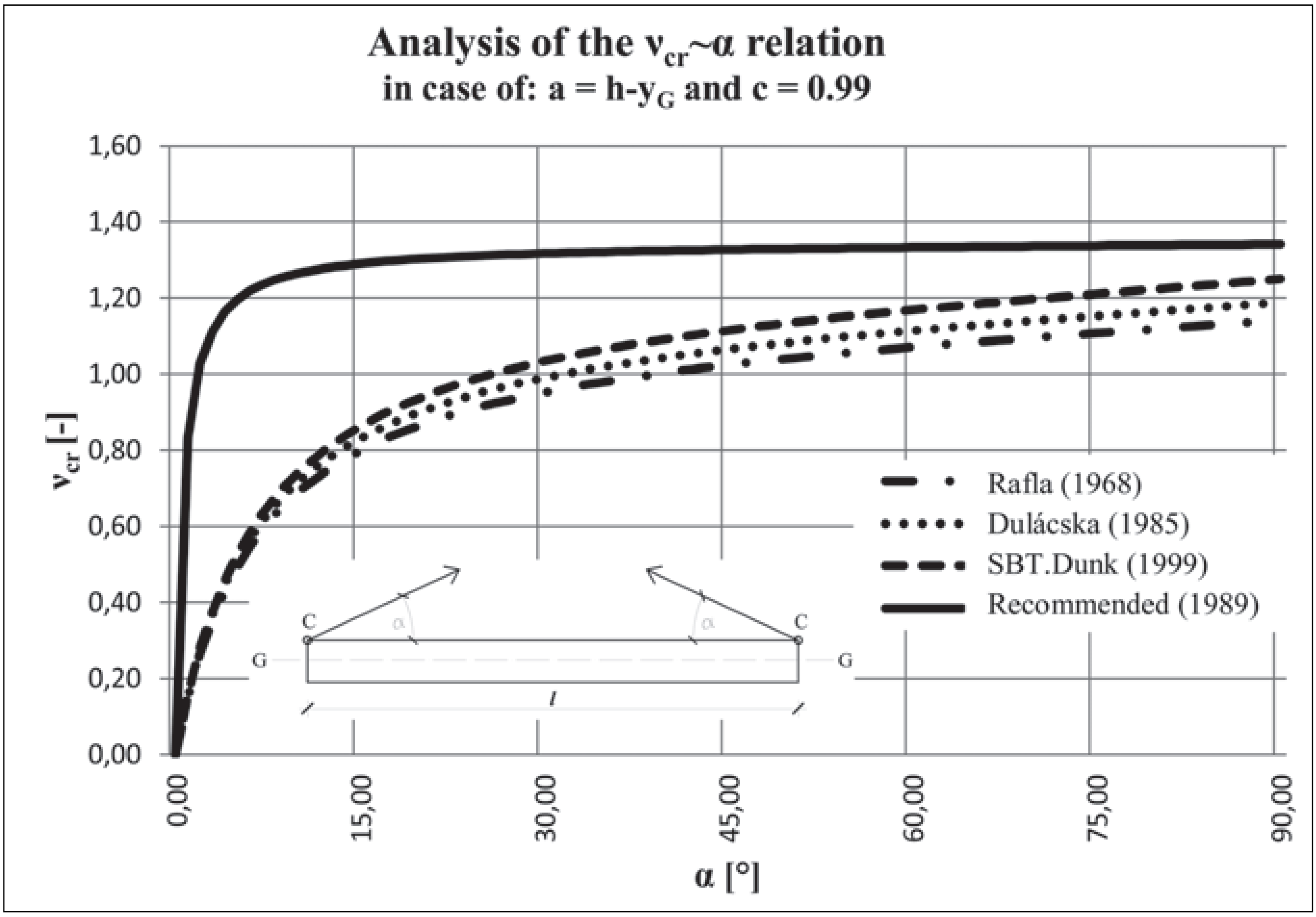


Fig. 5: Analysis of the ν_{cr}~α relation in case of: $a = h\text{-}y_G$ and $c = 0.99$

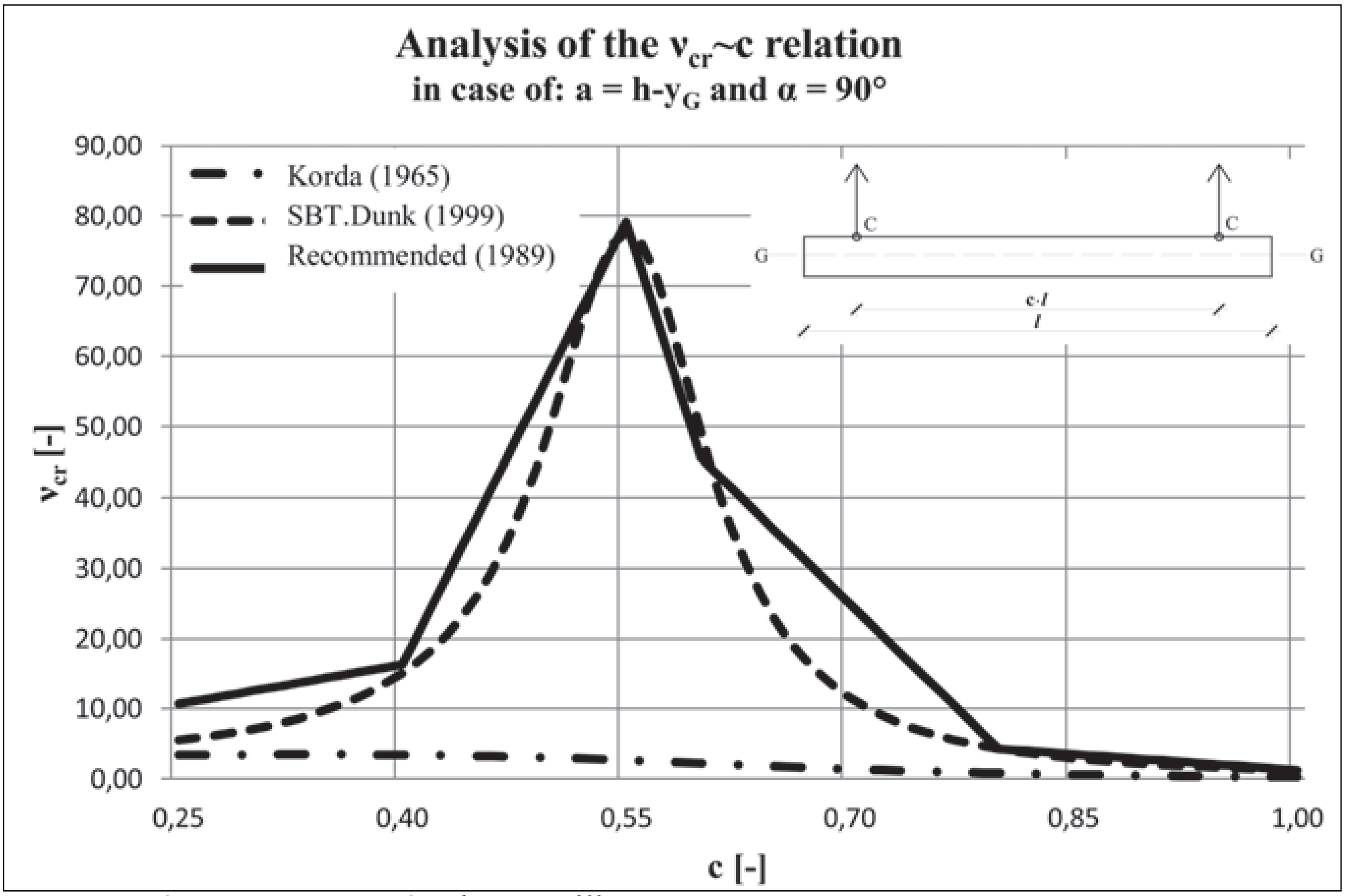


Fig. 6: Analysis of the ν_{cr}~c relation in case of: $a = h\text{-}y_G$ and $\alpha = 90°$.

the SBT.Dunk (1999) method the Korda (1965) formula approximates for the favour, and the Recommended (1989) method for the harm of safety on the whole *c* region.

Table 3 shows results of the safety level considering the stability failure for typical beam cross-sections used in the practice in Hungary.

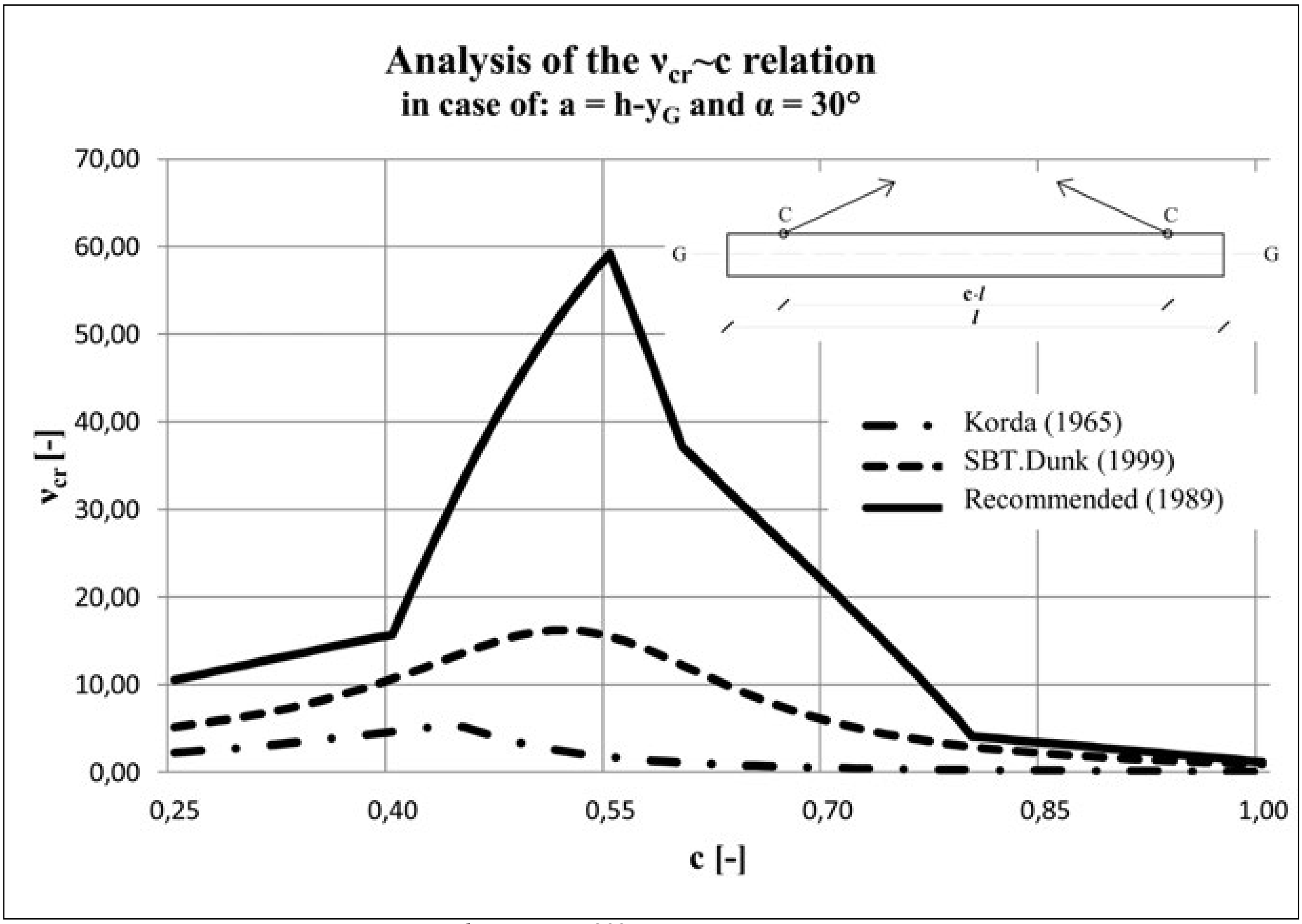


Fig. 7: Analysis of the ν_{cr}~c relation in case of: $a = h\text{-}y_G$ and $\alpha = 30°$.

5. RELATIONSHIP BETWEEN THE ROPE ARRANGEMENT PARAMETERS

In this chapter the graphs show the relation between the *a* distance of the rope connection point from the centre of gravity of the cross-section, α angle closed by the rope and a horizontal line and the *c* parameter, which determines the length of the consoles at the ends of the beam. Among these three parameters (*a*, α and *c*) two is chosen to draw a graph and one is set to be fixed. The relation between the two chosen parameters is determined for the case when the ν_{cr} safety level equals to *1.00*.

Fig. 8 presents the relation between the α rope angle and the *c* parameter. For the practice this is the most useable diagram, because it assumes that the distance of the rope attachment point (*a*) from the center of gravity of the cross-section is fixed, which is often true in the construction routine. With the increase of the *c* value the α angle is also increasing to satisfy the ν_{cr} = *1.00* condition. The relation between the two examined parameters is non-linear.

When the ropes support the beam at its endpoints (*c = 0.99*), the α angle should be around *30°* according to the SBT.Dunk (1999) method, and around *3°* according to the Recommended (1989) calculation method. The conclusion here is also the fact, that the Recommended (1989) method approximates for the harm of safety comparing to the SBT. Dunk (1999) method.

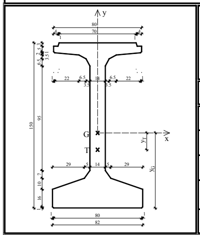
Fig. 9 shows the relation between the α angle and the *a* distance of the rope connection point from the center of gravity of the cross-section. The *a* distance is presented in its relation to the distance of the center of gravity and the top of the cross-section [$a/(h\text{-}y_G)$]. With the increase of the *a* value the α angle is decreasing to fulfil the ν_{cr} = *1.00* assumption. The *c* parameter is fixed to be *0.99* in this analysis. The relation between the two examined parameters is non-linear.

At the $a/(h\text{-}y_G)$ = *1.50* value the methods excluding the Recommended (1989) give values for the α angle around *15°*. The Recommended (1989) calculation method approximates for the harm of safety in this case as well, because it presents that only around α = 2° is enough to satisfy the ν_{cr} = *1.00* condition.

Fig. 10 shows the relation between the *a* height of the rope attachment point and the *c* parameter, which determines the length of the consoles at the ends of the beam. Around the *c = 0.55* value the graphs of the SBT.Dunk (1999) and the Recommended (1989) methods show minimum *a* heights. The graph of the Korda (1965) calculation formula has its minimum *a* values around the *c = 0.40* parameter. The *a~c* relation is non-linear and determined to satisfy the ν_{cr} = *1.00* condition when the α angle closed by the rope and a horizontal line is *30°*. The *a* distance is presented in its relation to the distance of the center of gravity and the top of the cross-section [$a/(h\text{-}y_G)$].

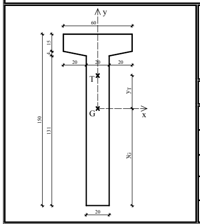
According to the SBT.Dunk (1999) and the Recommended (1989) methods the height of the rope attachment point should be close to the top of the cross-section of the beam ($a/(h\text{-}y_G)$ = *1.00*) to fulfil the ν_{cr} = *1.00* assumption when the rope supports the beam at its endpoints (*c = 0.99*). The curve of the Korda (1965) method is above the other graphs

Examination beam Nr. 1. *(l = 44.8m)*



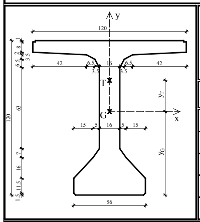
Safety level considering the stability failure (ν_{cr} [-])				
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	1.14	0.95	1.26	1.03
Korda (1965)	-	-	1.03	0.37
Dulácska (1985)	1.19	0.99	-	-
SBT.Dunk (1999)	1.25	1.03	6.70	4.08
Recommended (1989)	1.34	1.32	14.62	13.11

Examination beam Nr. 2. *(l = 32.0m)*



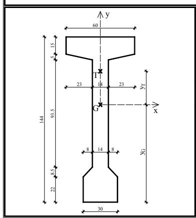
Safety level considering the stability failure (ν_{cr} [-])				
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	1.02	0.84	1.12	0.91
Korda (1965)	-	-	0.89	0.32
Dulácska (1985)	1.02	0.85	-	-
SBT.Dunk (1999)	1.07	0.88	5.76	3.46
Recommended (1989)	1.15	1.13	12.56	11.22

Examination beam Nr. 3. *(l = 32.8m)*



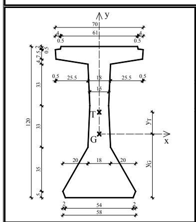
Safety level considering the stability failure (ν_{cr} [-])				
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	3.50	2.97	3.86	3.23
Korda (1965)	-	-	3.07	1.12
Dulácska (1985)	3.54	3.01	-	-
SBT.Dunk (1999)	3.72	3.14	19.97	12.72
Recommended (1989)	4.00	3.93	43.57	39.53

Examination beam Nr. 4. *(l = 35.0m)*



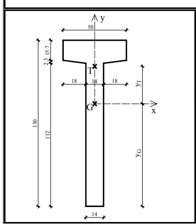
Safety level considering the stability failure (ν_{cr} [-])				
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	0.83	0.70	0.92	0.76
Korda (1965)	-	-	0.73	0.26
Dulácska (1985)	0.84	0.70	-	-
SBT.Dunk (1999)	0.88	0.73	4.72	2.93
Recommended (1989)	0.94	0.93	10.29	9.27

Examination beam Nr. 5. *(l = 32.8m)*



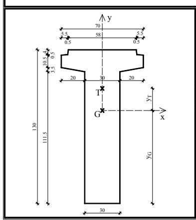
Safety level considering the stability failure (ν_{cr} [-])				
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	1.47	1.22	1.62	1.33
Korda (1965)	-	-	1.29	0.47
Dulácska (1985)	1.48	1.24	-	-
SBT.Dunk (1999)	1.56	1.29	8.37	5.10
Recommended (1989)	1.68	1.65	18.27	16.38

Examination beam Nr. 6. *(l = 25.0m)*



Safety level considering the stability failure (ν_{cr} [-])				
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	1.77	1.46	1.96	1.59
Korda (1965)	-	-	1.54	0.56
Dulácska (1985)	1.78	1.47	-	-
SBT.Dunk (1999)	1.87	1.53	10.05	5.98
Recommended (1989)	2.01	1.97	21.92	19.54

Examination beam Nr. 7. *(l = 34.8m)*



Safety level considering the stability failure (ν_{cr} [-])				
Method\Lifting case	1st	2nd	3rd	4th
Rafla (1968)	0.99	0.84	1.09	0.91
Korda (1965)	-	-	0.86	0.31
Dulácska (1985)	0.99	0.84	-	-
SBT.Dunk (1999)	1.04	0.88	5.58	3.60
Recommended (1989)	1.12	1.10	12.17	11.07

Table 3: *Results of the safety level considering the stability failure for typical beam cross-sections used in the practice in Hungary*

so it orders larger *a* values for the same *c* parameters which is an approximation for the favour of safety. The graph of the Recommended (1989) calculation method is mainly below the curve of the SBT.Dunk (1999) method, which is an approximation for the harm of safety.

6. EXPERIMENTAL ANALYSIS

After the comparison of the calculation methods found in different publications, experiments were made in order to proof the following facts:

- the safety level considering the stability failure depends on the rope arrangement parameters, and
- the modification of the parameters cause a change in the safety level in the direction that the calculation methods show.

Ideas about the experiments were got form (Liska, 1986), where the lifting arrangement and process is described in details.

For the experiments slender reinforced concrete beams were fabricated with a cross-section shown in *Fig. 13* (*Fig. 11*). The length of the beam is *8.50 m* (*l = 8.50 m*) and the arrangement of the lifting-hooks is presented on *Fig. 14.* With this build-up the variable parameters of the rope position are: α (angle closed by the rope and a horizontal line) and *c* (the parameter determining the length of the consoles at the ends of the beam).

The material properties were determined by the compression tests of the testing cubes in a laboratory. With the tests the compression strength of the material could be directly determined, and after that with the use of the equation found in the standard (Eurocode 2) the elastic modulus could be calculated.

$$E_{cm}[GPa] = 22 \cdot [(f_{cm}[MPa])/10]^{0,3} \tag{22}$$

The results for the material properties of the examined beam are presented in *Table 4.*

Table 4: The material properties of the experimental beam

Proper-ties	f_{cm} [N/mm²]	E_{cm} [N/mm²]	γ_c [kN/m³]	γ_s [kN/m³]	γ_{rc} [kN/m³]
Experi-mental beam	12.3	23400	18.2	1.9	20.1

In the followings a lifting series is described and compared to the results of the calculation methods. The rope was set to be vertical (α = 90°) and the beam was put down on the floor. The cable was attached to the lifting-hook with sign *1* (*Fig. 14.*) (the *c* value belonging to this console length is *0.818*). The beam remained stable after arising from the floor. The same happened when the lifting-hook with sign *2* (*c = 0.841*) was used. Finally when the same beam was elevated with the lifting-hook with sign *3* (*c = 0.865*) the lateral torsional stability failure occurred (*Fig. 12.*). The experiences of the experiments and the results of the calculation methods for the ν_{cr} safety levels in the tested rope arrangements of the experimental beam are presented in *Table 5.*

Table 5. shows that the SBT.Dunk (1999) method gives results closer to the experimental experiences than the Recommended (1989) calculation formula. The Korda (1965) method approximates for the favour of safety.

Fig. 15 presents the results for the ν_{cr} safety level considering the stability failure by the methods mentioned in *Table 5.* when the *c* parameter varies from *0.70* to *1.00* (the *c* parameter determines the ratio of the distance between the rope attachment points and the length of the beam). The main conclusion here (as in *Table 5*) is that the method which graph is the closest to the experimental results is the SBT.Dunk

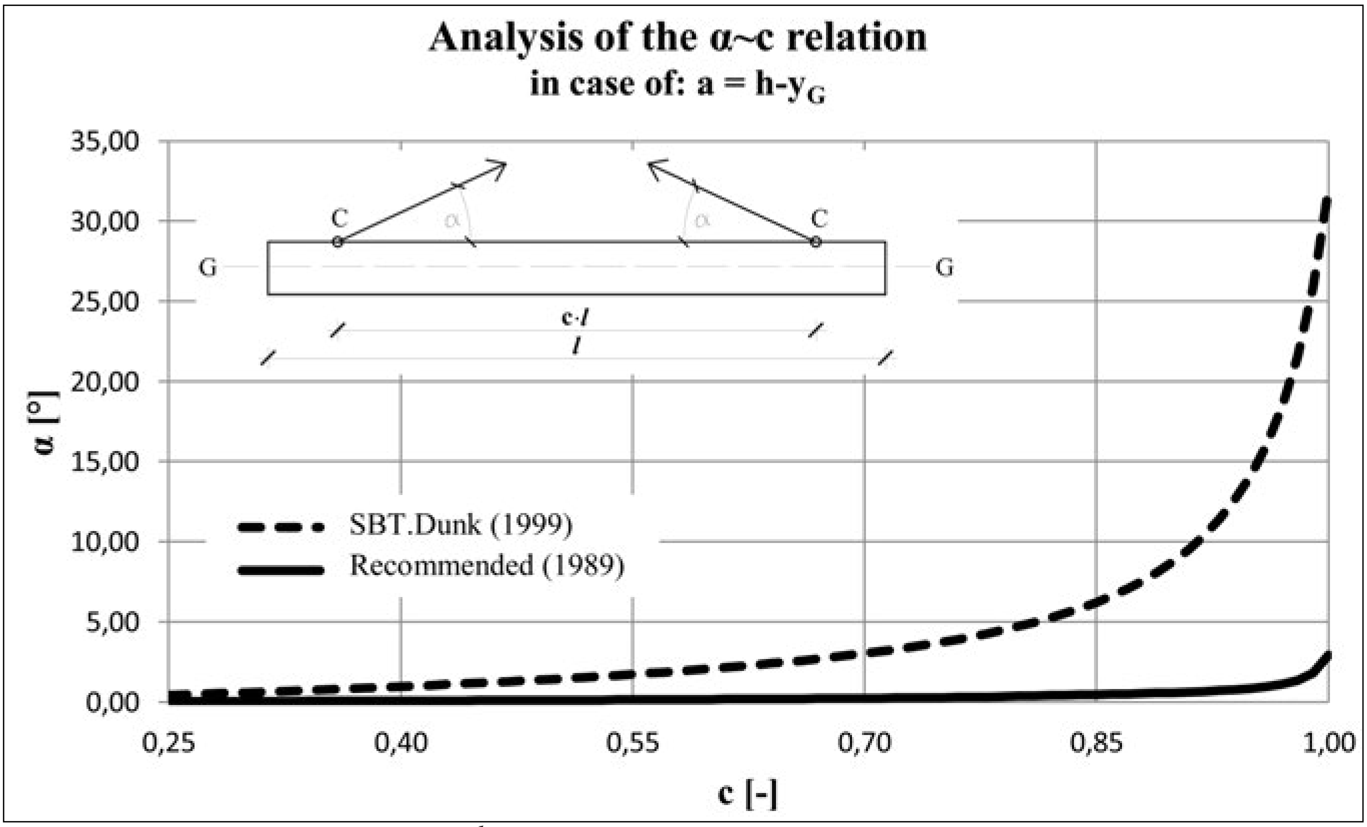


Fig. 8: Analysis of the **α~c** relation in case of: $a = h\text{-}y_G$

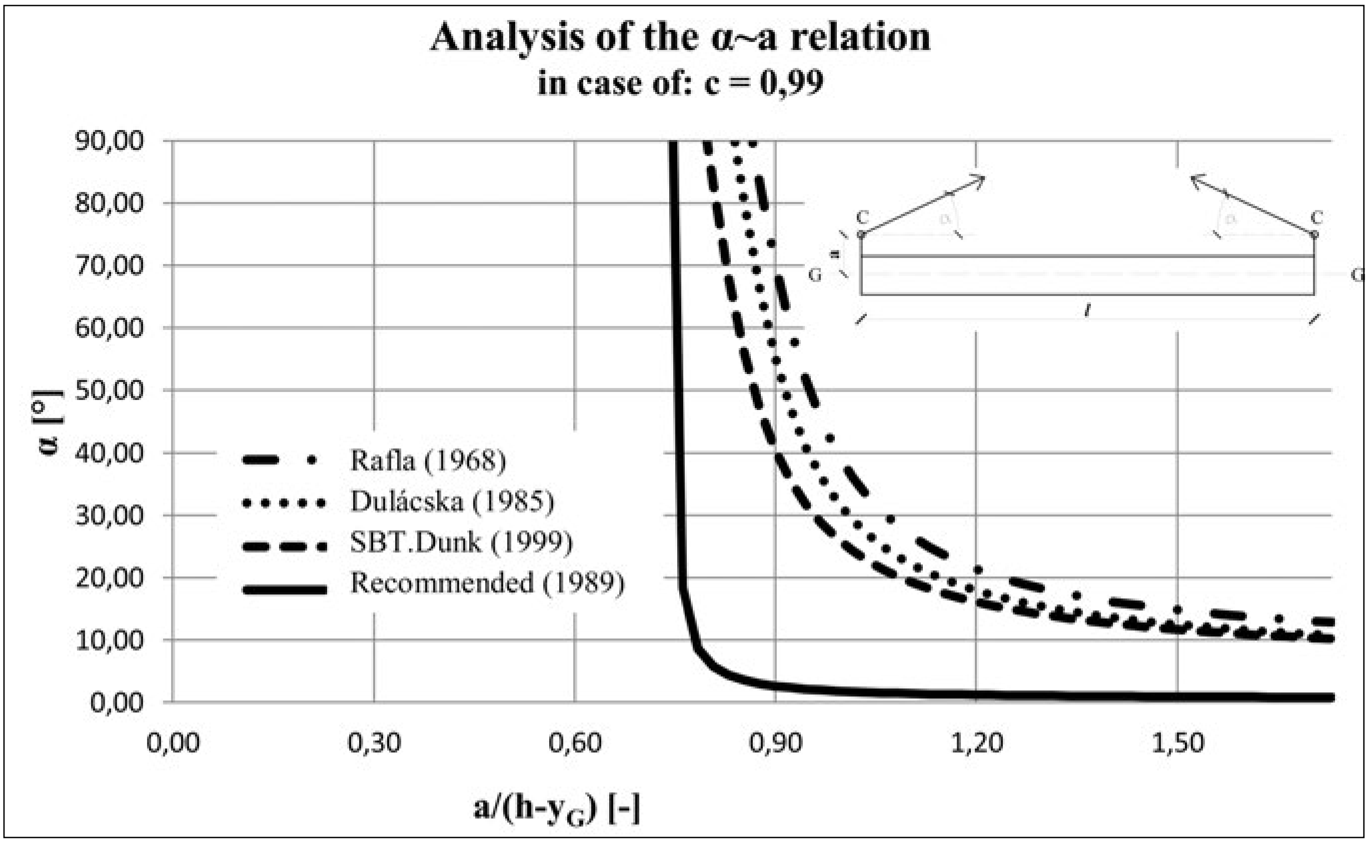


Fig. 9: Analysis of the **α~a** relation in case of: $c = 0{,}99$.

(1999). The Recommended (1989) method approximates for the harm, and the Korda (1965) method for the favour of safety. The tendencies on the graphs follow the experimental experiences: with the increase of the *c* value (at small console lengths) the safety level decreases.

The main conclusion of the experiments is that the safety level considering the lateral torsional buckling phenomenon changes in the direction that the calculation methods show by varying the rope arrangement parameters.

7. CONCLUSIONS

The general conclusion of this paper is that the lateral torsional stability analysis of precast concrete hanging beams is not a negligible calculation. As the study shows there can be critical rope arrangements for the slender beams when the safety level considering the stability failure reduces below *1.00*, which means that the stability failure occurs. With the correct choice of the height of the rope connection point, the

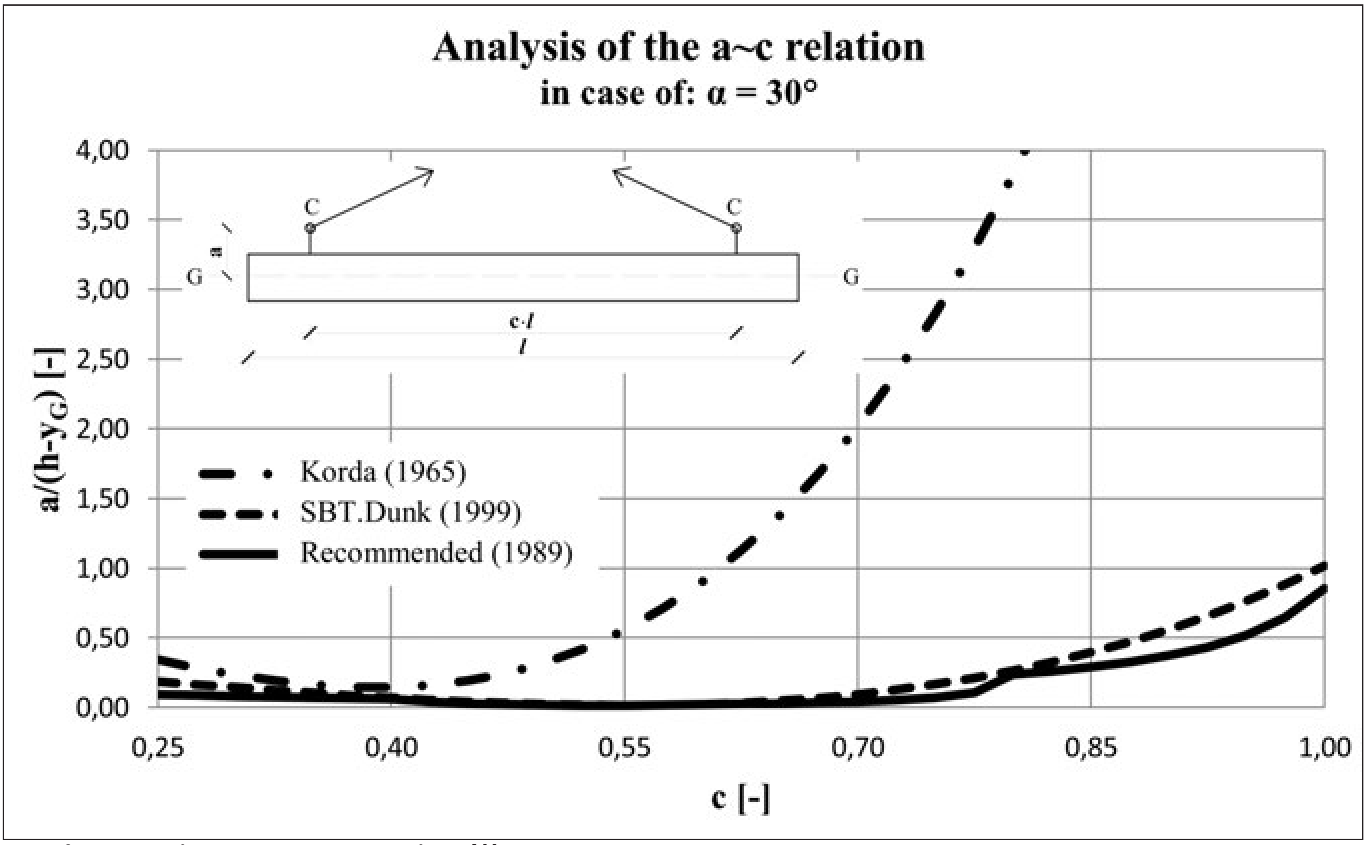


Fig. 10: Analysis of the *a~c* relation in case of: $\alpha = 30°$.

Table 5: *The safety level results of the calculation methods and the experimental experiences considering the stability failure*

Safety level considering the stability failure (ν_{cr} [-]) ($\alpha = 90°$, $a = 18.3cm$)			
Method\Position of the rope attachment (c [-])	0.818	0.841	0.865
Korda (1965)	0.37	0.33	0.29
SBT.Dunk (1999)	1.92	1.62	1.36
Recommended (1989)	2.08	1.89	1.70
EXPERIMENTAL BEAM	**stable**	**stable**	**failure**

angle of the rope and the length of the consoles at the ends of the beam this stability phenomenon can be avoided.

In the followings some basic conclusions are collected from the study.

- The difference between the results of the calculation methods considering the EI_w warping stiffness, the GI_t Saint Venant torsional stiffness and the EI_x, EI_y bending stiffnesses, and the ones taking only the EI_y bending stiffness into account (*Fig. 4.*) is negligible.
- In the practice lifting-hooks are used for the elevation of the beams so the height of the rope connection point is fixed (it is at the top of the cross-section). Due to this fact the most practical graph is on *Fig. 8.*, which determines the smallest rope angles for the given console lengths (*c* parameters) in order to keep the safety level above *1.00* (so the beam remains stable considering the stability failure).
- It can be generally stated that the Recommended (1989) calculation method, which is mainly used in the practice in Hungary, approximates for the harm of safety in most of the lifting cases. Eye-catching difference can be seen on *Fig. 7.*, comparing to the results of the other calculation methods.

Fig. 11: The experimental beam

Fig. 12: The deformed shape of the experimental beam after the stability failure

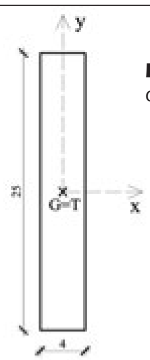


Fig. 13: The cross-section of the experimental beam

- The experiments confirmed that the safety level considering the lateral torsional buckling phenomenon depends on the change of the rope arrangement parameters, and this variation of the rope arrangement affects the safety level in the direction that the calculation methods show.

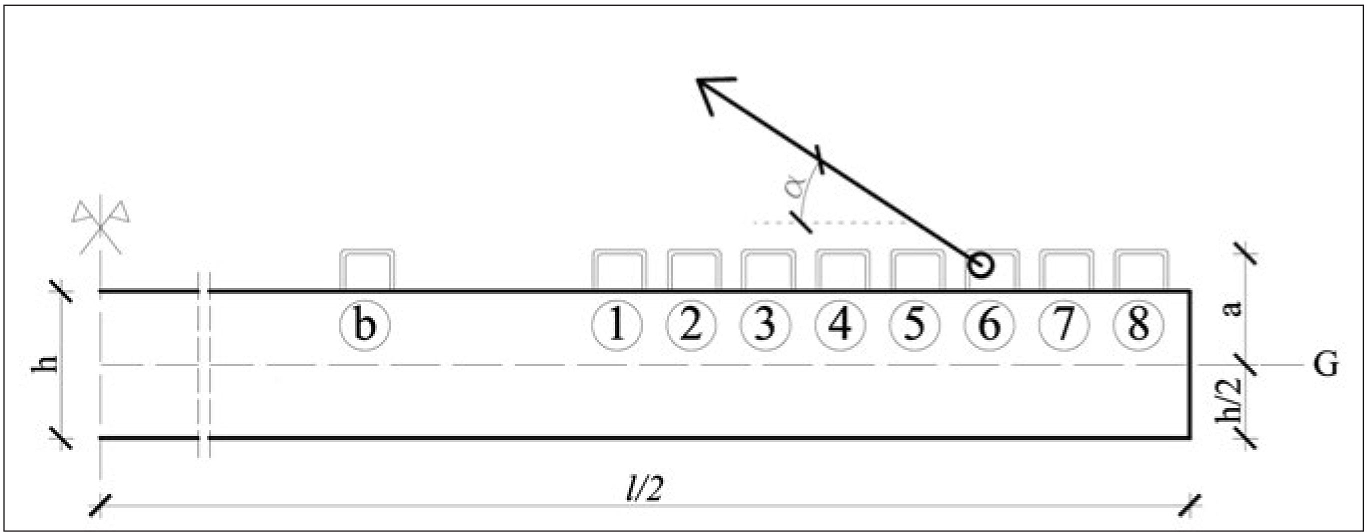


Fig. 14: The arrangement of the lifting-hooks at the ends of the experimental beam

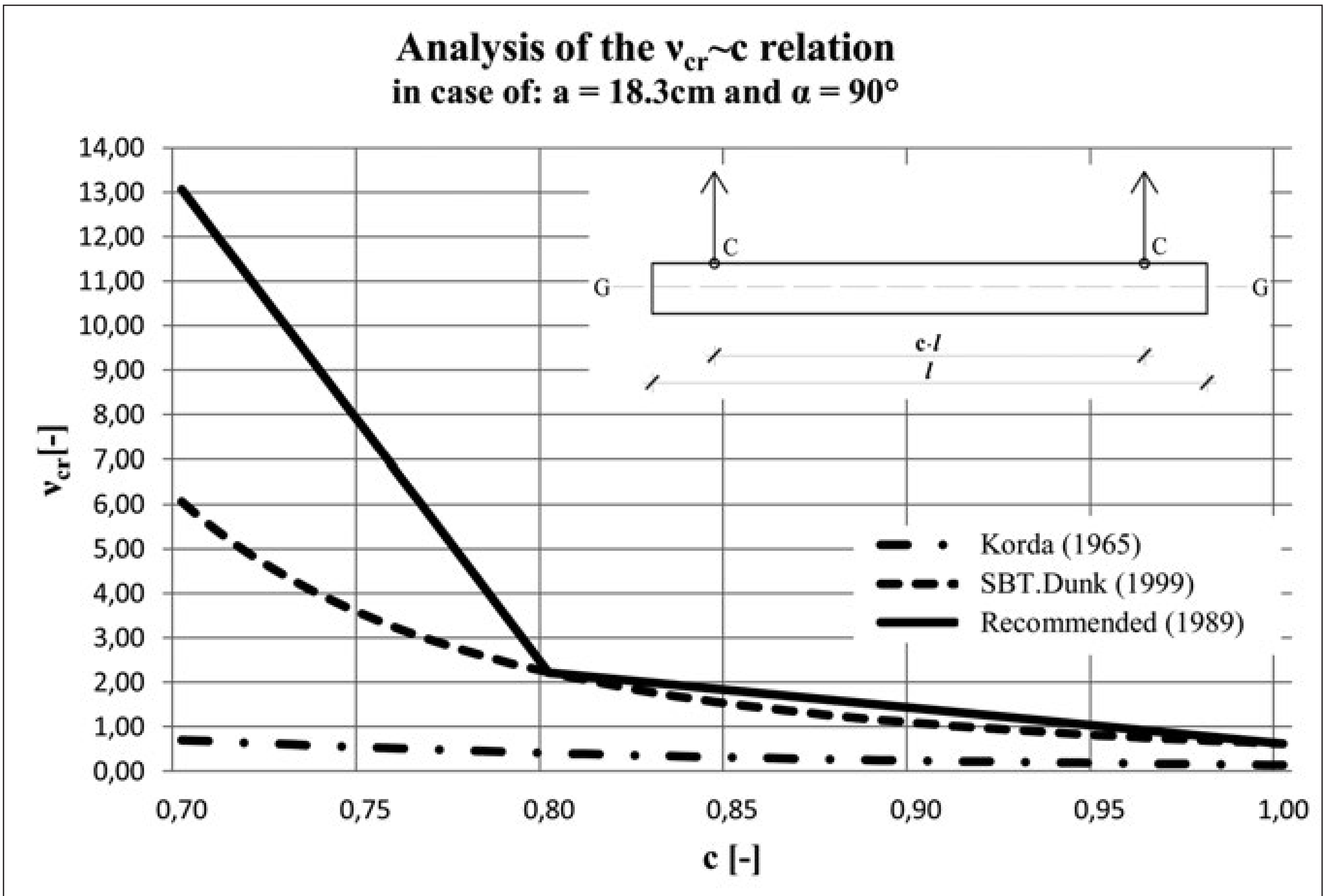


Fig. 15: Analysis of the ν_{cr}~c relation in case of: *a = 18.3 cm* and **α = 90°**

There is not exact calculation method determined in the Eurocode standard now, so it could be the subject of a further study to give a calculation formula suitable for the analysis of long span (*35-40m*) slender precast concrete beams with cross-sections symmetrical about the minor axis.

8. NOTATIONS

l	total length of the beam
h	height of the cross-section
A	area of the cross-section
y_T	distance between the shear center and the center of gravity of the cross section
y_G	distance between the center of gravity and the bottom of the cross-section in the direction of the *y* axis
$h-y_G$	distance between the center of gravity and the top of the cross-section in the direction of the *y* axis
i_p	polar radius of gyration
E; G	elastic and shear modulus of concrete
EI_x; EI_y	bending stiffnesses around the *x* and *y* axes
GI_t	*Saint-Venant* torsional stiffness
EI_w	warping stiffness
f_{cm}	mean value of the compression strength of concrete
E_{cm}	mean value of the elastic modulus of concrete
γ_c	volume weight of concrete
γ_s	volume weight of reinforcement
γ_{rc}	volume weight of reinforced concrete
a	distance between the rope attachment point and the center of gravity of the cross-section in the direction of the *y* axis
$a/(h-y_G)$	ratio of the height of the rope connection point and the distance between the center of gravity and the top of the cross-section in the direction of the *y* axis

α	angle of the rope to the horizontal
c	ratio of the distance between the rope connection points and the *l* length of the beam
ν_{cr}	safety level considering the stability failure $\nu_{cr}=p_{cr}/q$
P	normal force acting parallel with the fore-axis of the beam
q	uniformly distributing force acting parallel with the *y* axis along the length of the beam
M_q	bending moment around the *x* axis from the *q* uniformly distributing force
p_{cr}	critical uniformly distributing force considering the stability failure acting parallel with the *y* axis along the length of the beam
P_{cr}	critical normal force considering the stability failure acting parallel with the fore-axis of the beam
u	displacement in the direction of the *y* axis
ϑ	torsion of the cross-section around the shear center
φ	angle of the rotation around the rope attachment point as a rigid body
r_x	cross-sectional quantity:

9. REFERENCES

Dulácska, E. (1985): Lateral torsional plastic buckling of slender reinforced concrete beams (In Hungarian). Építési *Kutatás Fejlesztés*, (1985)/2, pp. 60-64.

Kollár, L. (1991): *The special problems of the stability theory on engineering fields* (In Hungarian). Akadémiai Kiadó, Budapest, pp. 17-40., 40-76., 613-634.

Korda, J. (1965): Stability of precast reinforced concrete structures during lifting process (In Hungarian). É. M. *IPARTERV Műszaki Osztály*, Budapest.

Liska, A. (1986): The effect of cracking on the lateral torsional critical load of the reinforced concrete girders lifted by ropes (In Hungarian). *Dissertation, Budapest University of Technology and Economics*.

Massányi, T. and Dulácska, E. (1989): *Handbook of static engineers* (In Hungarian). Műszaki könyvkiadó, Budapest, pp. 616-618.

Petersen, C. (1982): Statik und Stabilität der Baukonstruktionen. Friedr. Vieweg & Sohn, Braunschweig/Wiesbaden, pp. 665-677.

Rafla, K. (1968): Beitrag zur Frage der Kippstabilität aufgehängter Träger. *Dissertation, TU Braunschweig*

Stratford, T. J., Burgoyne, C. J. and Taylor, H. P. J. (1999): Stability design of long precast concrete beams. *Proc. Inst. Civil Engrs,* 134, pp. 159-168.

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