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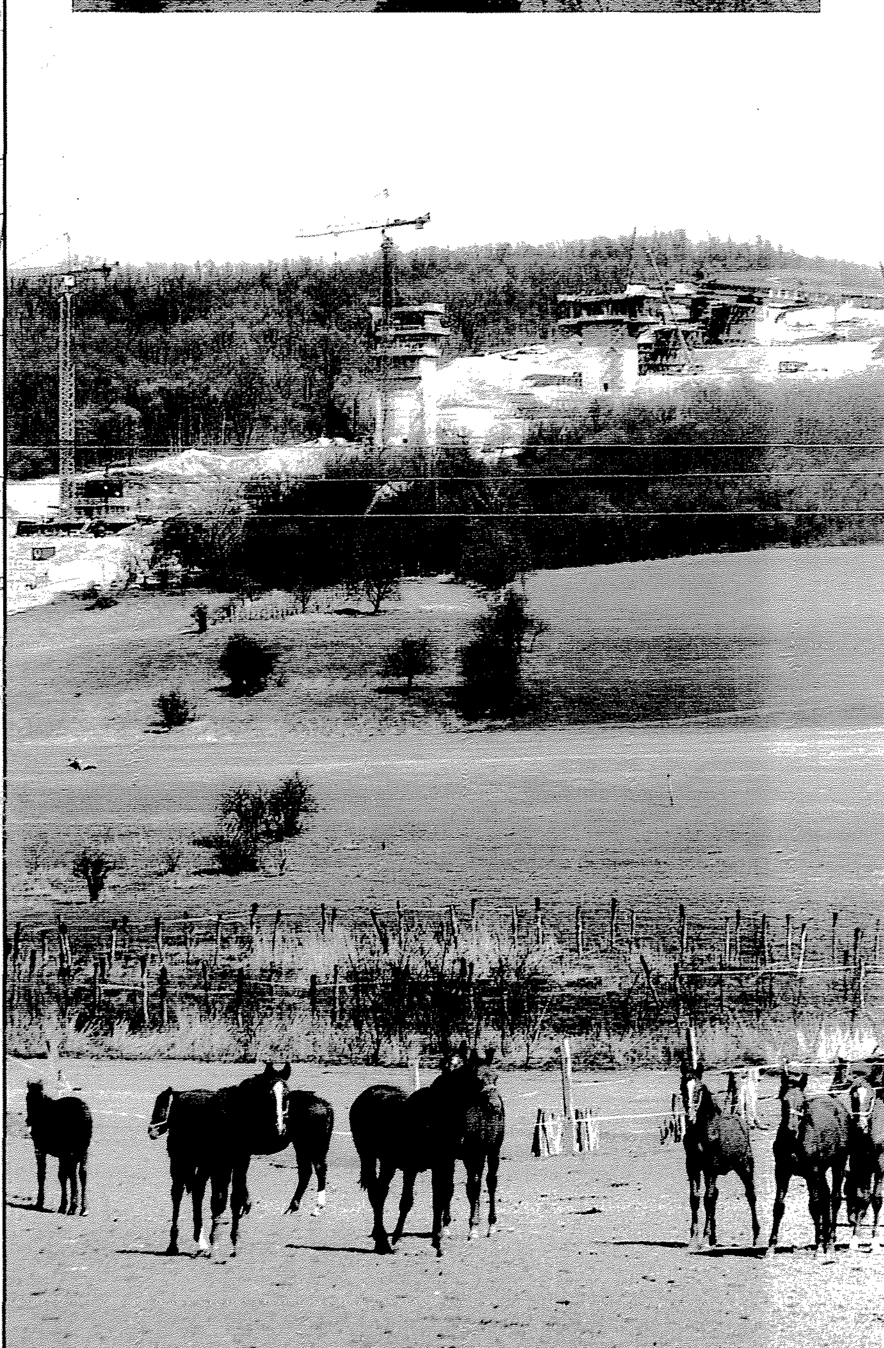

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Kőröshegy viaduct during construction on the Hungarian motorway M7
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BENEFIT OF TECHNICAL/SCIENTIFIC SYMPOSIA

– KEEP CONCRETE ATTRACTIVE –

fib Symposium 23-25 May 2005, Budapest, Hungary

Prof. Géza Tassi – Prof. György L. Balázs – Dr. Adorján Borosnyói

The ***fib*** Symposium to be held in Budapest in 2005 will be a highly stimulating meeting with themes relevant and appealing to an international audiente.

The organisers are very pleased and honoured by the interest shown by international specialists who have accepted invitations as keynote speakers to this Symposium. Additionally the response to a call for papers was overwhelming in its numbers, quality and diversity, and we have been able to choose papers which will generate lively discussions during technical sessions as well as stimulating debate during the breaks. The workshop will complete the message of the symposium.

Displaying the work of junior colleagues who have been awarded the *diploma for young engineers* confirms our belief that not only the structural concrete has a future, but that there is a talented generation to follow the work of the current generation of ***fib*** engineers and of the entire society of structural concrete engineers. The exhibition is always an informative and stimulating part of the meeting at which the displays will not only be depicted as pictures but also three-dimensional models of new achievements of concrete technology. The proceedings and other printed material distributed at the symposium will provide an enduring chronicle for attending delegates and also provide technical and other information to those who are unable to attend. Technical excursions will provide an impression of special technical solutions, methods and tools that will enhance the experience of visitors. Added to all of these technical-scientific benefits will be those exchanges of ideas which many delegates consider to be the most profitable aspect of the symposium. Such discussions may take place during coffee breaks and luncheons, (and we note that the word "symposium" refers to common meals during which there are exchanges of opinions and ideas). We anticipate that the 2005 ***fib*** Symposium will be rich in all of these professional benefits.

The task of finding interesting and stimulating themes for this symposium, given that ***fib*** holds six symposia and a congress every four years, has been a challenge, however the long history of structural concrete has proven that its development is enduring. The requirement for new research in materials, fundamental theory, design, production, construction techniques, maintenance and refurbishment is on the increase. The challenges facing many fields of application of structural concrete are increasingly more acute. Discussion needs to continue on both old and new problems being faced by this industry. We believe that the topics of the 2005 ***fib*** Symposium will draw the attention of delegates.

The following topics will be presented:

- Aesthetics of concrete structures
- Innovative materials and technologies for concrete structures
- Modelling of structural concrete
- Sustainable concrete structures
- Prefabrication
- Design of concrete structures with respect to fire resistance

Delegates to this symposium will remark the high number of papers which have been submitted to the organisers, showing that choice enabled a selection of high quality papers for presentation. We trust that delegates will benefit greatly from the presentations and discussions as well as the workshop. To the honourable readers of this journal who regrettably are unable to attend the 2005 *fib* Symposium in Budapest, we offer the proceedings and other reports on this event as an alternative.

The organisers and the members of the Hungarian Group of ***fib*** hope that delegates and accompanying persons will enjoy the atmosphere of the symposium, the venue – both the Hungarian Academy of Sciences building as well as the ship "Európa", anchored near to the Academy – and also the picturesque surroundings of the city of Budapest, the social events, the excursions and all aspects of the meeting. And lastly, we hope that the visitors to this Symposium will become better acquainted with our homeland, our historic capital city, with Hungarian culture and history, with the unique spirit of the inhabitants and we offer the hospitality of our people towards all nations of the world.

It is anticipated that the principal professional benefit of the meeting will be to encourage the structural engineers and architects of the world to

KEEP CONCRETE ATTRACTIVE.

THEORETICAL BACKGROUND OF ACOUSTIC EMISSION ANALYSIS


Lea Zamfira Forgó – Florian Finck

Acoustic emission analysis is a non-destructive test method, by which the damage of materials can be monitored. Damage processes can be detected in time and space by localisation of acoustic events. By means of moment tensor inversion and graphical presentation, we can give a fracture mechanical characterisation of crack propagation. In the next paper in this issue of the Concrete Structures, a practical example will follow the theoretical review by providing insight into the presentation and evaluation of a compressive test of a concrete specimen.

Keywords: acoustic emission, fracture mechanics, localisation, moment tensor inversion

1. INTRODUCTION

If the distance of an atom or ion pair exceeds the critical atomic distance, due to any loading, the adhesive cohesion energy or a part of it is deliberated and dissipated as heat or vibration. The vibration is the acoustic emission generated by this elastic, irreversible inner deformation processes (or acoustic emission) that can be in the audible and imperceptible acoustic range as well (*Fig. 1.*).

The quantitative examination of acoustic emissions (e.g. by a localisation) enables us to follow the damage in time and space. Furthermore, the registered wave forms can be evaluated by inversion of the moment tensor. It provides information on the type of the fracture, orientation of failure surface and the radiated seismic energy (*Grosse, 1996; Finck, 2001*).

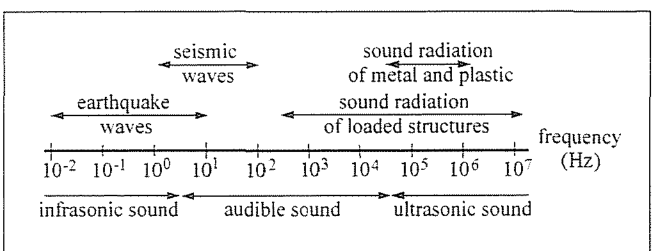


Fig. 1: Sound wave frequency ranges (*Czigány, 1998*)

2. OVERVIEW OF THE METHODS

The detection of sub-audible sounds was successful for the first time in the 50's by making special instruments. At the beginning of the 70's the first acoustic emission instrument was created that could have been used in practice, too. In Hungary, the research in this field has begun in 1976 at the Central Research Institute for Physics. One application of the acoustic emission analysis is the leakage control in the monitoring system of the nuclear power station of Paks. These tests have been performed since 1989 (*Czigány, 1999; Klimaj, 2001*).

Besides active ultrasound methods, where acoustical energy is transmitted by using ultrasonic sound generator (e.g. piesoelectric), ballimpactor or a hammer we can measure structural elements, detect cracks, voids on difficult accessible or inaccessible areas. There are also passive methods and one of them is discussed in this article. The acoustic energy or vibration of total atomic element is generated due to internal processes in consequence of external effects (e.g. earthquakes, wind).

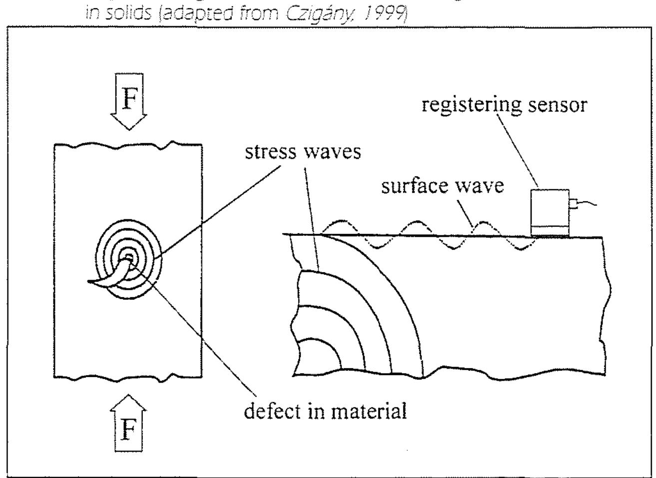
At parameter-based acoustic emission analysis signals are analysed online by a computer and they are stored (*Fig. 3.*). This method supports fast, simple localisation of acoustic emissions. It is useful when only a rough, but overall image of the damage is sufficient. In case of signal-based acoustic emission analysis full wave phases of acoustic events are stored, thus detailed examination of fracture processes can be managed, interferences can be filtered and reasons of deformations, damage procedures can be analysed. This technique is necessary for other methods such as tomography, residuum-technique or coherence analysis (*Grosse, 1996*).

3. ACOUSTIC EMISSION ANALYSIS

3.1 Localisation of acoustic emission

Acoustic emission means appearance or emission of elastic, transient tension waves during structure change of a tension bearing material. When the material is isotropic and homogeneous the waves travel from a defect location as spherical symmetric waves, and this can be registered with suitable instruments (*Fig. 2.*).

Fig. 2: Origin, propagation and detection of loading-related stress waves in solids (adapted from *Czigány, 1999*)



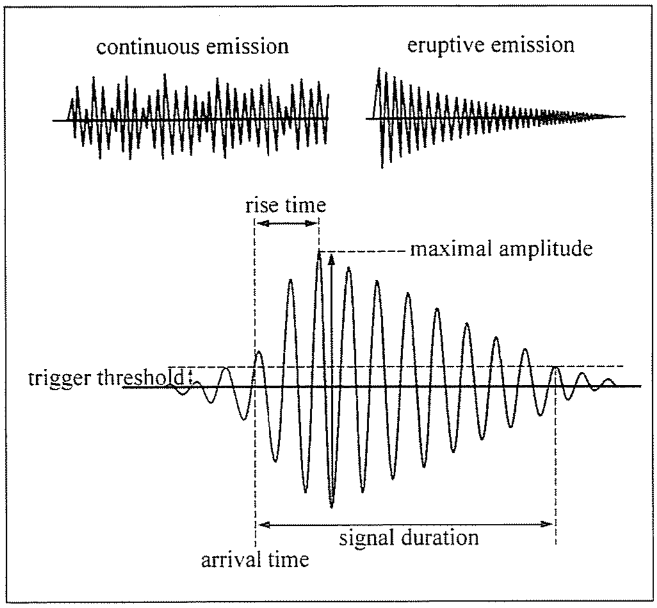


Fig. 3: Top: continuous and eruptive emission (*Klimaj, 2001*). Bottom: description of an acoustic emission signal

Acoustic emission analysis is related to compressive waves, which are the fastest nevertheless the weakest ones. The detection of such waves is our main goal. Because this wave arrives first, it is called primary wave (P wave). Transversal or secondary waves (S waves) and other waves follow these.

During the registration, the so-called trigger plays a significant role. The trigger registers the overcome of a given signal level. Background noises give a rise to a continuous acoustic emission (*Fig. 3.*). The trigger starts the registration when an eruptive wave is generated and it exceeds the level of background noise. *Figure 3.* shows some parameters that can be registered by parameter-based acoustic emission equipments. Such parameters are: arrival time, maximal amplitude, rise time, signal duration, etc.

The air pores in the concrete, the aggregates and crack generation exert a big influence on the expansion of waves (absorption, break-up, impingement from a side-wall, etc.), and occasionally inhibits the detection of P waves. Equipment and background noises cause difficulties in the evaluation, therefore it is important to have an acceptable signal-noise ratio during the registration of the acoustic emission events.

When a 3D solid is monitored, at least four sensors are necessary for the localisation of an acoustic emission event, because of the four unknown parameters (the three spatial coordinates and the source time). In case of more sensors an overdetermined equation system will be obtained as it is shown in the equation:

$$v_P (t_S - t_0) - (x_S - x_0) = 0 \qquad (1),$$

which leads to a more successful and reliable localisation (v_p: P wave velocity in the material, t_s and x_s: appreciated time and distance between the source and the given sensor, t_0 and x_0: beginning time and the place of the sensor in the coordinate system).

Localisation of origin of acoustic emission events means the identification of hypocentre of acoustic emission (namely that space will be defined, from where the fracture begins or from where the displacement of the particles of the material compared to one another begins). By means of localisation, it is possible to follow the damage processes in space and time. This is the first and most important step in the data interpretation besides the distinction between noises and acoustic emission events.

In case of regular test-samples, there are not any acoustic emission events under a given load. Testing experiences show that at such time intervals the Kaiser-effect is documented, namely the material 'remembers' to the maximal load of the previous stress condition. According by the acoustic emission analysis is suitable to determine the primer stresses. Csorba and Huszár (*1988*) have examined the phenomenon for stones.

3.2 Fracture mechanical background

Fracture mechanics studies the conditions, the origin and extend of fractures. Broek (*1982, 1989*), Blumenauer and Push (*1987*) have an engineering approach, while Bažant and Planas (*1998*) as well as Carpinteri and Aliabadi (*1999*) applied fracture mechanics for concrete.

To control the inversion methods of fracture mechanics on natural data, well-known and pure fracture types are needed. While by means of tension tests pure tension fracture (mode I – opening mode) can be induced easily, to produce pure shear fracture in plane (mode II – sliding or shearing mode) is more difficult. A non planar fracture is also exist (mode III – tearing mode). Reinhardt and Xu (*1998*) have shown that such compression tests, when the specimens are loaded on half side length with vertically distributed load, can produce shear fracture in plane, but suitable geometry and load is necessary.

3.3 Moment tensor inversion

The description of acoustic emission events is based on mathematical description of point sources and they will be applied in acoustic emission experiments (*Czigány, 1999; Grosse, 1999*).

Concrete with aggregates of 10 mm in diameter or less can be considered as homogeneous from measuring technical point of view, because in the experiments the wavelengths exceed 20 mm (*Finck et al., 2003*). Consequently, we suppose that the material is homogeneous and isotropic.

The moment tensor, which fully describes the characters in place and time of the acoustic emission source, can be decomposed into the following (from each other independent) fracture types:

$$M = M^{ISO} + M^{DC} + M^{CLVD}, \qquad (2)$$

$$M^{DC} = M^{II} + M^{III}. \qquad (3)$$

M^{ISO} indicates the isotropic part of the source (expansion or constriction). M^{DC} and M^{CLVD} are deviatoric, non volume variable. As the equation above shows, the shear part can be further divided. In case of M^{II}-mode the rupture is spreading parallel to the particle movement. In case of M^{III}-mode these two motions are perpendicular to each other. A shear crack follows the double couple mechanism M^{DC}. M^{CLVD} hides compensated vector dipole, namely it describes a physical model, when besides the tension a perpendicular contraction is also generated.

Figure 4. shows the physical meaning of the symmetric 3×3-tensor. Dipoles point in all the three directions with a couple of tensors at the end. In the first diagonal the couples are parallel to the dipoles, in the secondary diagonal they are perpendicular to the dipoles. Supposing rotational symmetry around the z-axis, the tensor can be subdivided into the forms shown in *Figure 5.*

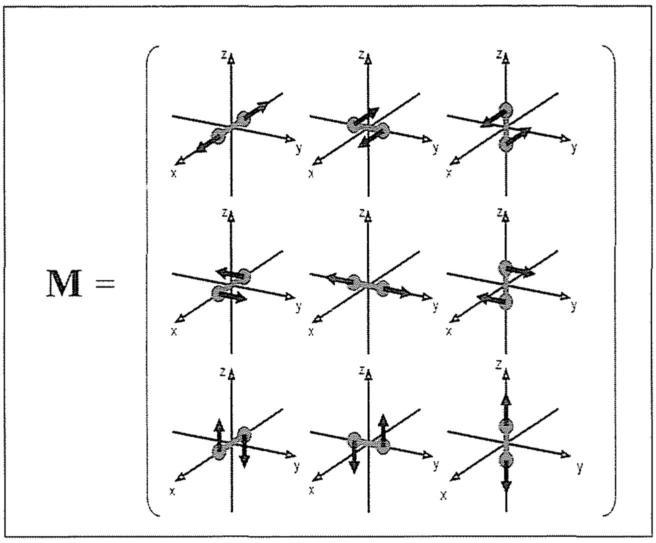


Fig. 4: Interpretation of tensor components as couples (*Finck, 2001*)

Because of the Green's functions are unknown (*Bojtár, 1988*), which characterise the influence of the material properties on the waveforms, the moment tensor can not be directly inverted. Dahm's relative moment tensor inversion (*1993, 1996*) gives a solution to the problem. To take the influence of the covered wave-length to the sensors to the same extent, we form groups from acoustic emission signals that are very close to each other (*Fig. 6.*). By means of detecting the amplitude, which can be measured at appearance of primary waves, the Green's functions can be finally eliminated (*Grosse, 1999*).

3.4 Graphical representation of the moment field

In case of fracture, the radiation of the seismic energy in the source region can be entirely represented as a projection on a hemisphere, for example with the Lambert's asimuth method that keeps constant surface area. In the case of a DC, in the projection of the moment field the strongest radiation is in the middle of the compressed site or of the tensile site and it can be characterised with the maximum (P, pressure) or minimum (T, tension) principal moment. The P-T-plot characterises the shearing of DC (*Fig. 7.*).

The graphic representation of the moment field is difficult. When the principal axis transformation is carried out, there

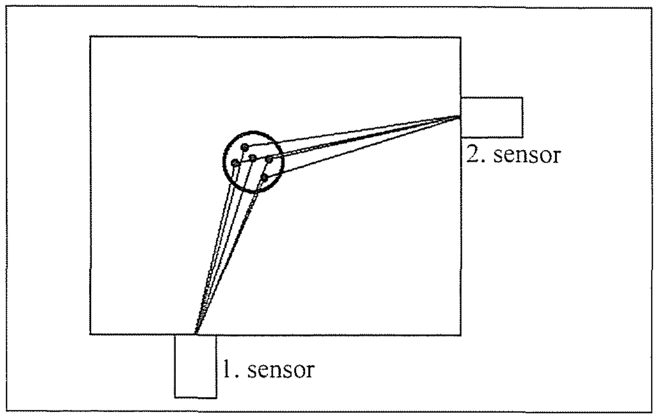


Fig. 6: Grouping of acoustic emission signals in order to eliminate the Green's functions

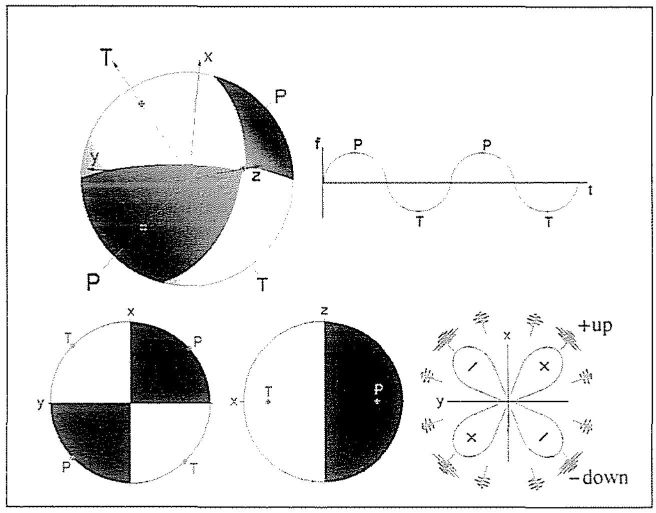
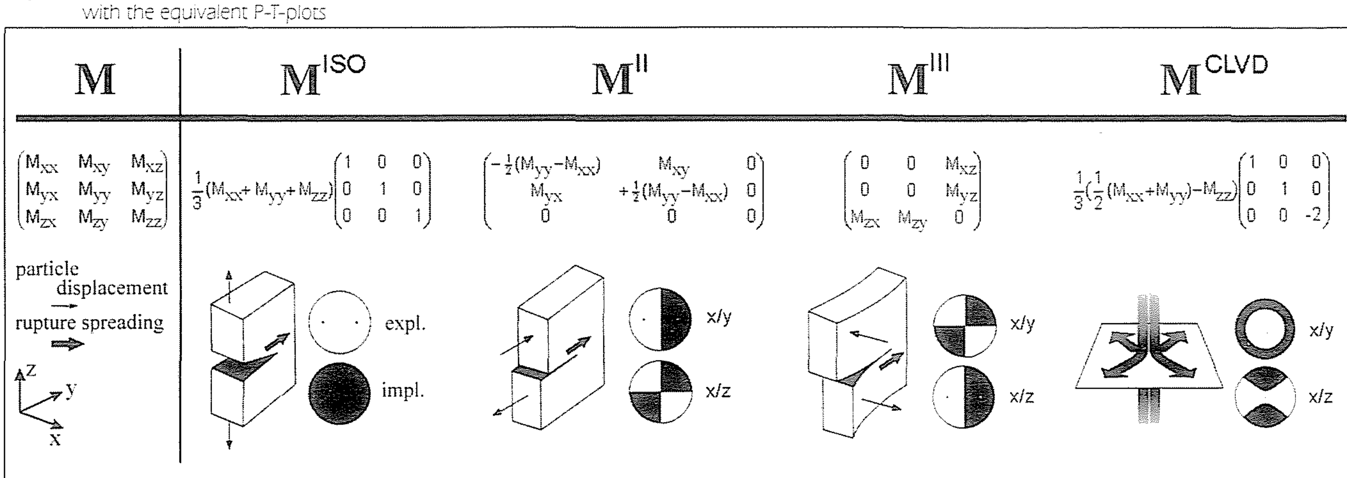


Fig. 7: A possible graphical interpretation of the moment field in the case of pure shear failure. Top left: moment field with the first moment diagonals, in left-handed rectangular coordinate system (pressure: black and positive; tension: white and negative). Top right: amplitude above the asimuth (two sinus waves: compared with the moment field – it is visible that the highest amplitude is at the first diagonals and it is zero at the boundary of tension and compression). Bottom left and middle: Lambert's projection to the x/y- and x/z-planes. Bottom right: radiation characteristics on the x/y-plane (the waves propagate upwards under pressure and downwards at tension; the signals are the strongest at the first diagonals, they are weaker in any another direction and they are not observable at the boundary of compression and tension).

will be three eigenvector with the related eigenvalues. In case of expansion, all of the three eigenvalues are equal and positive. By tension failure one eigenvalue (in the tensile direction) exceeds the other two. In case of pure shear failure one

Fig. 5: Top: A possible distribution of moment tensor components (*Grosse, 1999*). Bottom: displacement models (*Broek, 1982*; adapted from *Finck, 2001*) with the equivalent P-T-plots



eigenvalue is zero, while the remaining two are equal, but have the opposite sign (*Weiler, 2000*).

The graphic representation of tension failure is similar to the representation of shear failure. The T-axis corresponds to the tensile direction, for P-axis we choose one from the two remaining axis with identical eigenvalue. To distinguish the tensile failure from the shear failure the eigenvalues of the calculations have to be taken into consideration, the P-T-plot oneself is not enough to decide (*Weiler, 2000*).

4. CONCLUSIONS

Acoustic emission analysis is a non-destructive test method to monitor the internal damage of materials, based on the detection of oscillations generated by internal movements of the material under tension.

The method can be applied for the monitoring of machines and equipments during their operation, since the probable destruction time can be forecasted from the physical parameters of the generated sound waves (*Czigány, 2000*).

The advantage of the acoustic emission analysis is that the damage progresses can be entirely followed and furthermore this method gives a more profound and a more detailed information compared to methods, where the failure itself is studied. The knowledge of the processes that leads to the failure is absolutely important both for the research of the available materials and for the development of new materials.

It is necessary to remark that nowadays the evaluation of signal-based acoustic emission data is difficult and time consuming. The effort for the automatisation of the data treatment (*Finck et al., 2003*) makes the method promising in the field of testing, diagnostics and production-control.

5. ACKNOWLEDGEMENT

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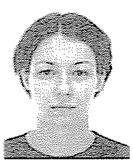
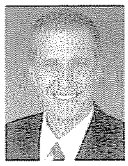
Florian Finck (1969) studied geophysics at Free University in Berlin and is research associate and PhD student at the Institute of Construction Materials at University Stuttgart. Main fields of research are acoustic emission analysis, fracture mechanics of brittle materials and non-destructive testing.

FRACTURE MECHANICAL TESTING BY MEANS OF THE ACOUSTIC EMISSION ANALYSIS

Lea Zamfira Forgó – Florian Finck

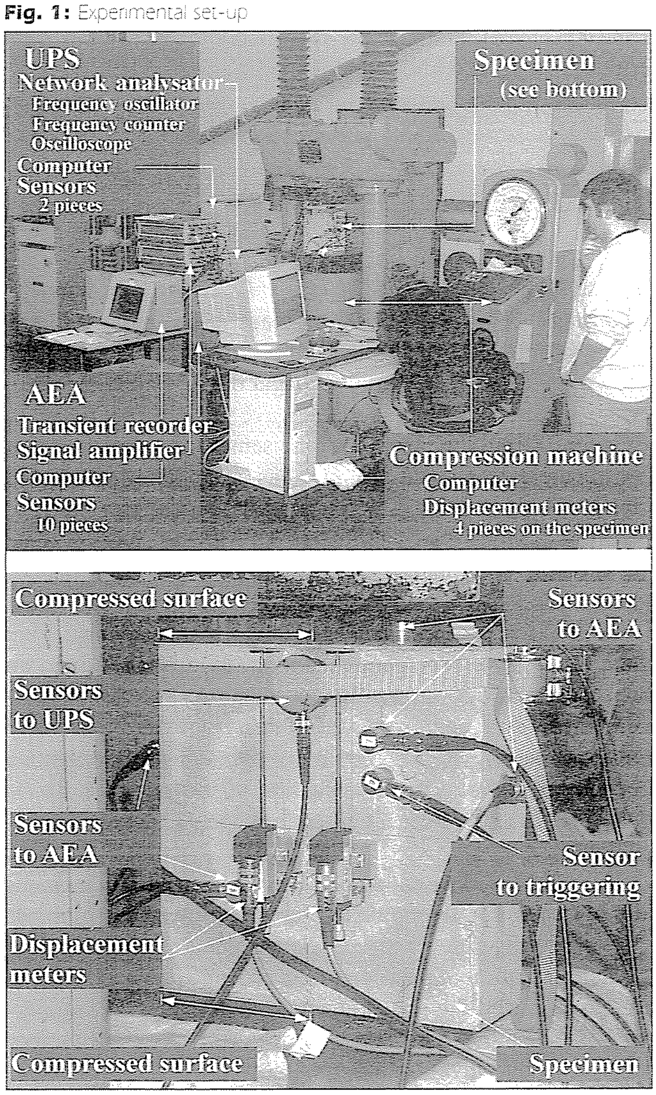
This paper describes a practical example of a pressure test monitored by acoustic emission technique. The experimental set-up and the geometry of the one-sided loaded specimen allow to analyse pure failure modes. Thus relative moment tensor inversion method could be tested. The localisation of the acoustic sources provided good result, while by the inversion of the moment tensor and the graphical presentation caused difficulties.

Keywords: compressive test, acoustic emission, localisation, fracture mechanics, moment tensor inversion

1. INTRODUCTION

The Materials Testing Institute University of Stuttgart was founded in 1884 and has always been a main research centre for materials of all kinds. Testing of construction materials began when Otto Graf became the director of this institute. In the past ten years acoustic emission analysis is one of the key research fields at the Institute of Construction Materials (*Grosse, 2001*). One of the first publications on acoustic emission analyses in Hungarian was written by Pellionisz (*1993*). The present paper follows the previous theoretical one (*Forgó & Finck, 2005*, in this issue) by describing the application of the method for a compressive test.

Fig. 1: Experimental set-up



2. EXPERIMENTAL SET-UP

While pure tension fracture can be induced easily, to produce pure shear fracture is problematical. Reinhardt et al. (*1997*) and Reinhardt & Xu (*1998*) attempted to produce the latter failure mode with several experiments by changing the geometry and the loading conditions. The experimental set-up of a compressive test on a half side length loaded specimen with vertical load distribution is shown in *Fig. 1*.

A 300 mm × 300 mm × 100 mm concrete specimen (C40/50 according to EN 206-1, see also *Table 1*) has been prepared and according to the experiences of Reinhardt & Xu (*1998*) a 100 mm × 150 mm surface was compressed to generate pure mode II failure. The dimensions were also chosen for practical reasons: suitable surface dimensions for placing the sensors, mobility etc. Between the specimen and the metal compression surface a teflon layer was applied to reduce the friction (*Balázs, 1994*) and to eliminate the noises related to local stresses coming from the compression machine (*Csorba*

Table 1. Characters of the produced concrete

Sample	Size [mm^3]	Volume [dm^3]
3 pieces	150 · 150 · 150	3.375 / piece
2 pieces	300 · 300 · 100	9 / piece

Components	Type	kg/m^3	in Agitator of 40 l [kg]
Cement	CEM I 42.5 R	420	16.80
Water		210	8.40
Aggregate	0 / 0.6	381	15.24
	1.2 / 2.0	310	12.40
	2.0 / 4.0	494	19.76
	4.0 / 8.0	460	18.40
FMF flux		3.0	0.07

Storage
20 °C / 100 %: 7 days
20 °C / 65 %: min. 21 days

& *Huszár, 1988*). According to Balázs (*1994*) there is no friction, when the height is three times the width of the specimen and spatial stress does not appear at edges of the specimen and an uniaxial stress is generated in the middle part of the test sample. We have taken the specimen dimensions similar to the experiments of Csorba & Huszár (*1988*), considering the above listed conditions.

Hydraulic compression machines have a noise with uncertain extent and it significantly influences the data quality (the signal-noise ratio). Therefore, a manual hydraulic compression machine was used (max. load: 3000 kN, Tonindustrie Prüftechnik, Berlin).

The signals were recorded by a transient recorder with eight channels based on two Elsys Trans PC data acquisition boards. The amplitude resolution was 12 bit and the samole rate 5 MHz. An external, so-called slewrate-trigger (type: 800 1 TRF, W+W Instruments AG, Basel) was used to identify the signals (*Forgó, 2004*).

The delay time of the transient recorder was 0.5 s while the data saving process was active, which means that two signals could be recorded per second.

Eight multi-resonant pieso-electrical sensors (type: UEAE, Geotron) were glued on the specimen for the acoustic emission analysis and additional two sensors of the same type for the triggering. The signals were preamplified by a Tektronix TM504.

An ultrasonic phase spectroscopy experiment was carried out parallel to the acoustic emission analysis (*Grosse et al., 2002*). Therefore, the loading process was stopped at certain load levels temporarily to perform these measurements.

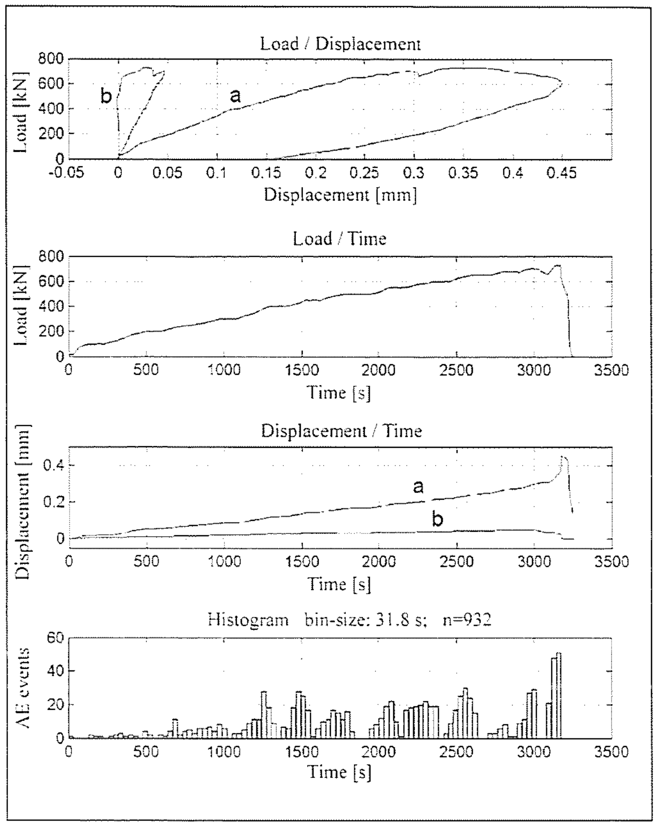


Fig. 2: Machine dates and statistics. Load / displacement, load / time and displacement / time relations diagram and origin of acoustic emission / time relation histogram; 2a2 graph: displacement in the compressed zone, 2b2 graph: displacement in the free zone

3. RESULTS

3.1 Analysis of ultrasonic sound wave propagation

For the evaluation of the acoustic emission analysis, we have to know the propagation speed of the ultrasonic P waves in the concrete specimen. This has been measured first by ultrasonic-impulse technique with 1-2 % discrepancy. The average of nine results has been calculated, giving a value of app. 4500 km/s. The anisotropy of the concrete specimen was proved to be neglectable.

3.2 Statistics: load-/time-/displacement-correlations

The load, time and four external displacement meter values were recorded by a computer. In the case of the displacement meters we have taken the average of data of the meters that were located on the opposite side of the specimen and these have been presented. On the histogram of *Fig. 2* the constant periods of the load are visible and in accordance the generated acoustic emission is minor during the ultrasonic phase spectroscopy measurements. The load/displacement diagram shows compression both in the free and in the loaded field – this can be attributed in the free area to the 'closeness' of the pairs of meter to the compressed field and thus it is related to the success of the compressive effect. When the diagrams are compared it is clear that in the compressed field the compression is more significant. Furthermore, an opening crack was in the loaded zone, which closed again nearly completely (*'a' curve*), while there was only elastic deformation in the free zone (*'b' curve*). The first two diagrams show that around the critical load (~ 700 kN) the specimen was unloaded when the cross shearing took place.

3.3 Localisation

A computer program WinPecker© (*Grosse, 2000a*) with subroutine Hypo^{AE} (*Onescu & Grosse, 1996*) was used for the localisation (*Forgó, 2004*) of the acoustic emissions.

The menu lines of WinPecker© 1.2. (*Grosse, 2000b*) are shown in *Fig. 3*. The input data (right down) is the geometry of the test specimen in mm, the P wave velocity mentioned in Chapter 3.1 and the position of the sensors in a right-handed coordinate system. The program has an on-set picker based on Hinckley's criterion (*Grosse, 2000b*). Vertical lines in *Fig. 4* point the estimated arrival time of the P waves, registered by eight sensors. During our experiment such wave-sequence of 8 has been stored 932 times.

Several results have been deleted from the data base. That data, which have been generated by us by turning off and on the equipment during the ultrasonic phase spectroscopy examination, have been removed. Other ones were extremely noisy and were useless some further erroneous data were also deleted not to cause the interruption of the software. So finally 710 data have been localised from the 932 acoustic emission events.

By using of semi automatic option we have modified manually the offers for determine the acoustic emission beginning.

Considering that 10 mm is the acceptable inaccuracy for the determination of position of the acoustic emission events, we have deleted the acoustic emission events with large errors. Because the WinPecker© showed the iteration number incorrectly and invalid iterations showed as valid, occasion-

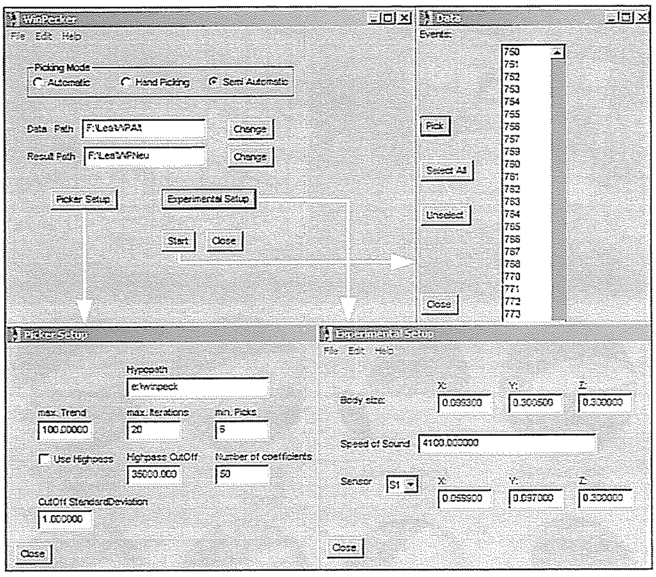


Fig. 3: Top left: main menu of the program WinPecker®, bottom left: menu of program settings, bottom right: menu of input data of the experiment, top right: serial numbers of acoustic emission events from the experiment and the 'Pick' program starter button

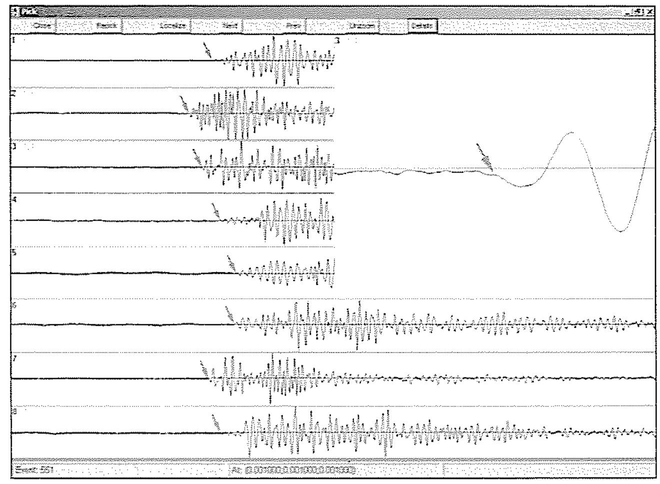


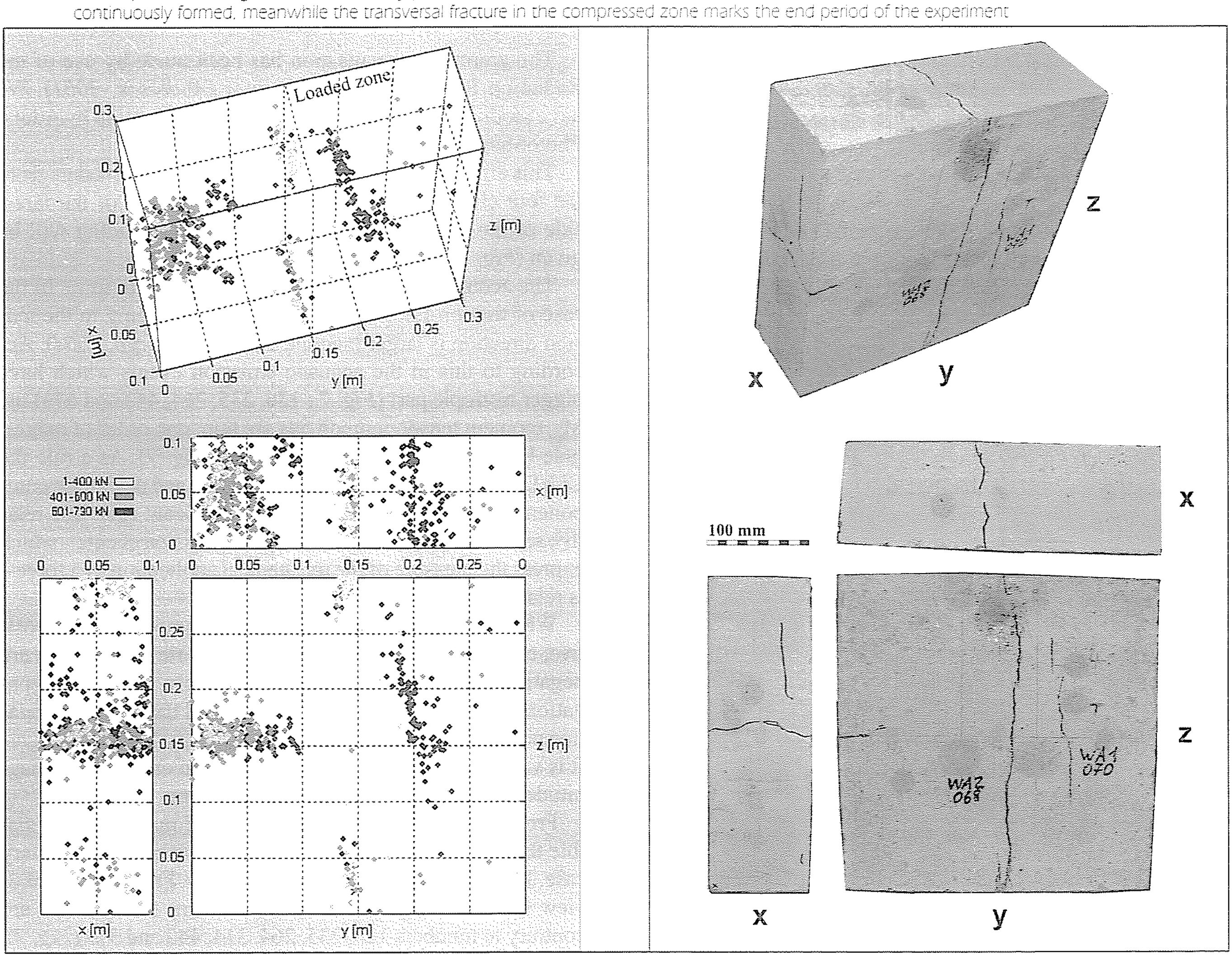
Fig. 4: The signals recorded by eight sensors. Evaluation of the beginning of primer waves (arrows) with the program WinPecker®. Top right: enlarged view of the cursor

ally also with acceptable accuracy, we worked with the aid of the Hypo 2-1 subroutine, too. After performing the iterations those acoustic emission events have been deleted, for which the inaccuracy exceeded the acceptable limit. Finally approximately half of the registrated events, 474 have been pronounced to be correct (*Table 2*). It can be regarded succesful because

Table 2: Number of the registered, localised and valid acoustic emission events in the function of load and time

Load [kN]	Time period [s]	Registered AE event	Localised AE event	Valid AE event	Code
0 - 400	0 - 1320	171	140	83	bright
401 - 600	1380 - 2360	408	291	203	medium
601 - 730	2460 - 3180	353	279	188	dark
		Σ: 932	Σ: 710	Σ: 474	

Fig. 5: The localised acoustic emission events (left) and position of macro ruptures in the specimen (right) are in good correspondence. The rupture in the top of the specimen was generated in the early period of the experiment, the rupture in the left side of the specimen and the crack below have been continuously formed, meanwhile the transversal fracture in the compressed zone marks the end period of the experiment



the acoustic emission experiments are acceptable when 30 % of the hits is correct.

It is important and remarkable that for the localisation an ultrasonic sound velocity value of 4100 m/s of the concrete was used which was obtained by ultrasonic phase spectroscopy and not the 4500 m/s value which was measured with ultrasonic impulse technique. Thus the results were slightly less exact – but still far within the acceptable error range –, but the number of valid acoustic emission hits were more significant.

The results were graphically represented by the use of MatLab 6.5 and they were more clearly described by use of Adobe PhotoShop 4.0.

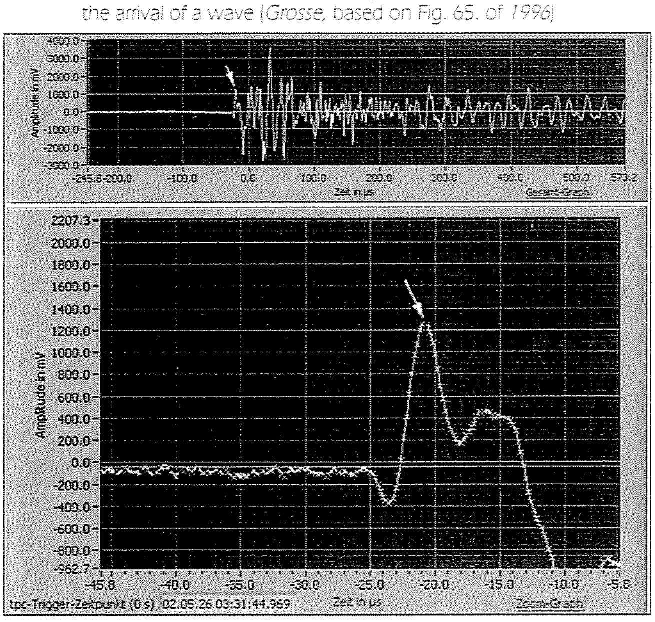
Comparing the pictures in the *Fig. 5* it is visible that the acoustic emission events coincide with the macro-crack pattern of the concrete specimen. Four fracture zones are discernible on the specimen. Acoustic emission hits are observable in the middle from the load period before obtaining the 400 kN, timely scattered ones are in the free zone down in the middle, while some are under the loaded zone from the load period over 600 kN.

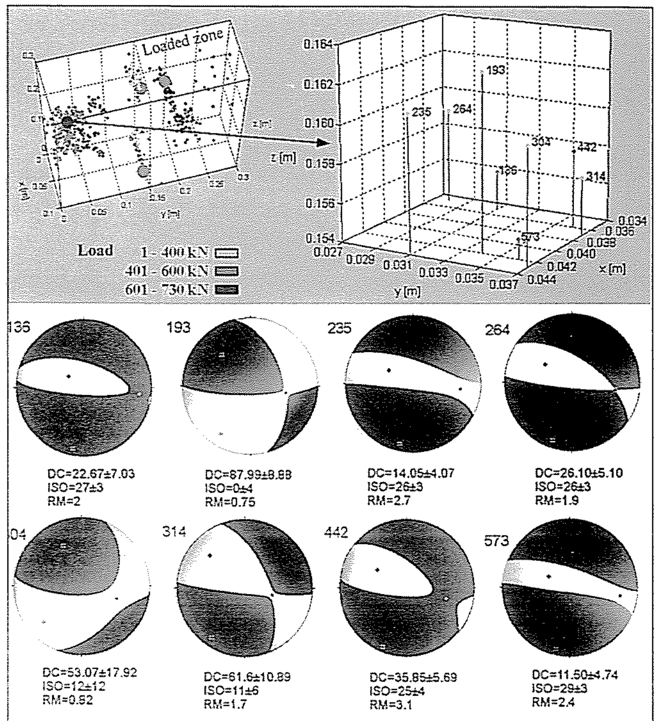
3.4 Fracture mechanical examinations: relative moment tensor inversion and graphical representation of the moment field

Within each of the four fracture fields a cube of roughly 10 mm edge-length was considered and the including 4-8 acoustic emission events were grouped. We supposed that such grouping with close standing acoustic emission events allow us to eliminate the Green's functions and provides a good image for the whole fracture characteristics (*Forgó, 2004*).

Only the first half periods of the waves give information about the fracture, the next set of waves is already influenced by the geometry and the material properties. The phases of the P waves of the acoustic emission events are characterised by the polarity and volume values of the first considerable peak. Determination of the input data of the adopted relative moment tensor inversion (*Dahm, 1993*) is frequently unambiguous (*Fig. 6*) that is why it needs experience.

Fig. 6: Reading of the first significant amplitude by means of the program ShowPick 7.7.a. Top: the whole signal, below: help window with the arrival of a wave (*Grosse*, based on Fig. 65. of *1996*)





Event No.	M_{xx} (h)	M_{yy} (h)	M_{zz} (v)	M_{xy} (h)	M_{zx} (v)	M_{zy} (v)
136	1.3	0.043	-0.113	-0.258	0.29	0.012
193	-0.333	0.0238	0.268	0.707	-0.619	-0.162
235	1.28	-0.000847	-0.0762	-0.316	0.282	-0.0113
264	1.17	0.0714	-0.095	-0.446	0.326	0.0114
304	0.461	-0.148	0.28	0.559	-0.717	-0.132
314	0.592	0.13	-0.105	-0.792	0.427	0.0406
442	1.13	0.116	-0.129	-0.526	0.263	0.0779
573	1.3	0.0371	-0.0563	-0.319	0.221	-0.0191

Fig. 7: Cluster 1. Top, left: 3D position of clusters, top, right: position of the acoustic emission events within the cluster, middle: P-T-plots, bottom: moment tensor components

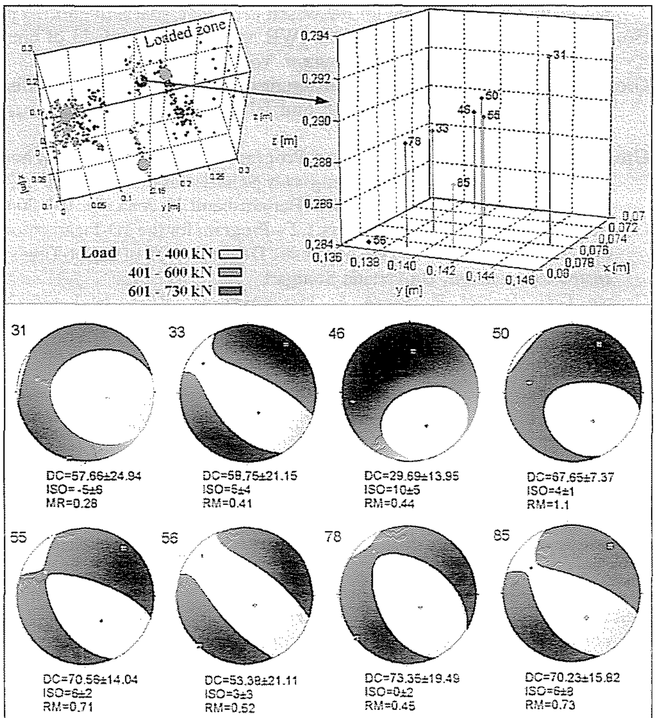
The graphical visualisation has been made by use of the Radiation Pattern Bitmap Generator (*Andersen, 2001*). We have modified these files for the better visualisation by Adobe PhotoShop 4.0.

This article presents detailed analyses of two clusters from the four clusters. First the group in the middle of the farest side of the specimen that is away from the loading will be given (*Fig. 7*).

The acoustic emission events are scattered in time. In the case of tension fracture the eigenvalue belonging to the tension direction is larger then the other two (*Forgó, 2004*). According to this in the acoustic emission events which have bigger isotropic part (*Fig. 7.*: 136, 235, 264, 442 and 573) the M_{xx} moment tensor components are min. one order of magnitude larger than the M_{yy} and M_{zz} ones (*Fig. 7*). As a rule the CLVD-part is also large, when the isotopic and deviatoric components in the x-x direction play a significant role. The relative seismic moment of these acoustic emission events – which express the measure of the mechanical application of a force – is relatively large.

When the horizontal M_{xy} deviatoric components are positive, than the vertical M_{zx} and M_{zy} deviatoric components are negative and it is also true contrary. Namely, the crack propagation parallel active M^{II} components and the crack propagation perpendicular active M^{III} components have opposite signs. It is visible that the M_{zy} components are in one order of magnitude larger than the two other components.

From the recumbent position of the P-T main axes it is visible that the moment fields stand in similar position in every case. Here the difference is in the polarity. From this point of view the acoustic emission event numbers 193 and 304 are contrary to numbers 136, 235, 264, 314, 442 and 573 (*Fig. 7*).



Event No.	M_{xx} (h)	M_{yy} (h)	M_{zz} (v)	M_{xy} (h)	M_{xz} (v)	M_{zy} (v)
31	0.567	-0.08	-0.952	-0.168	-0.132	-0.581
33	0.608	0.372	-0.534	-0.659	-0.409	0.0441
46	0.141	0.467	-0.0341	-0.318	-0.82	-0.327
50	0.425	0.26	-0.288	-0.517	-0.712	-0.244
55	0.537	0.386	-0.431	-0.676	-0.479	-0.0414
56	0.559	0.426	-0.665	-0.716	-0.087	-0.108
78	0.371	0.537	-0.85	-0.545	-0.22	-0.282
85	0.507	0.373	-0.462	-0.671	-0.494	-0.0154

Fig. 8: Cluster 2. Top, left: 3D position of clusters, top, right: position of the acoustic emission events within the cluster, middle: P-T-plots, bottom: moment tensor components

The results above mostly coincide with the expected ones, since the tension is parallel with the z axis.

According to this the main moments would have been in the z direction. Considering the fine lateral shift of the main axis of tension this corresponds to the small angle. The appearance of the P-T-plot presented in the middle of the picture has not include every important information since it serves for the graphic presentation of the shear components. Consequently our statements are to deal with reservations. The moment tensor is different and according to our calculations it is the strongest into the x-x direction, which is different from what we have expected before.

Fig. 8 shows the group near the loaded zone in the upper part of the specimen. Every acoustic emission events shown are from the first period of the experiment with a loading level under 400 kN. According to a shear fracture the DC-part plays the main role, ISO-part hardly appers. The CLVD-value can be difficulty estimated because of the uncertainity due to the huge error. The relative seismic moment is small in every case. The horizontal M_{xx} and M_{yy} values are positive, while the others are negative. The moment tensor components are equivalent in magnitude. The position of the moment fields are similar. The T axis stands similar in every case. The P axis also, and the only changes are whether, in which side it is visible.

In spite of the large error the P-T-plot is more accurate than previously. The fracture is next to the loading zone (*Fig. 5*). According to this it is possible that in the moment field of the selected acoustic emission events the tension direction is almost vertical and the compression direction is almost horizontal.

After the inversion of the moment tensor and preparation of P-T-plot (graphical representation of the results) we have seen that above in the middle part a shear crack is observable, while in the free zone mode I. fracture is related to the tensile stress. In the upper middle part assumingly an edge rupture is noticeable coming from the disorder of the specimen edge and airpores, thus here it was not successful to induce pure shear fracture. A transversal displacement is observable in the loaded zone. This fracture view (*Fig. 9*) does not allow us to determine the fracture mode, further experiments and statistics are needed to decide, whether it was a transversal slide-fracture or crumbling fracture originating from compression loading.

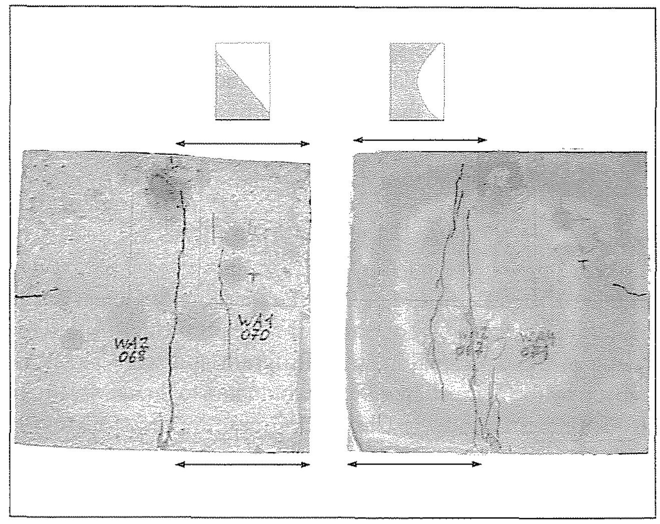


Fig. 9: Opposite sides of the same specimen. It is not possible to judge from the fracture pattern below the loaded zone (arrows) whether it was a transversal slide-fracture along the fracture layer or it is crumbing fracture related to the compression loading

There is a relationship between the isotropic components and the relative seismic stress, since when the ISO-part plays a significant role the relative seismic stress increase, too. The influence of the CLVD-part can be difficulty judged from this point of view since of the error factor is huge.

4. CONCLUSIONS

The experimental adjustment and the geometry of the specimen (*Reinhardt & Xu, 1998*) seem to be convenient to induce well-known and pure fracture modes and also to control the method of moment tensor inversion.

The evaluation of several hundred data mass is time-consuming, and with the development of the sign-register equipments the continuous registration of acoustic emission might be possible. There will be an increase of the data mass. The need for automatic localisation and problem solution becomes urgent.

The reading methods of amplitude value of the input data of the relative moment tensor inversion are controversial. With comprehensive and detailed tests these parameter could be more precise. The representation of the P-T-plot, which graphically shows the parameters of the moment tensor is not adequate yet. Its coordinate system differs the one used during the experiment employed and this makes the evaluation more difficult. Although, the results are quantitatively good, the moment directions do not correspond to the original ones.

To eliminate the above mentioned problems it is necessary to study, whether the huge error is related to these problems.

The data processing of the acoustic emission analysis is labouresome and lengthy nowadays, but the technical development of the method is encouraging. There are successes in the field of testing, diagnostics, produce controlling or operating supervision.

Whereas the method of acoustic emission analysis is not well-known and wide-spread in Hungary, there were some institutions where similar studies have been made, for example at departments of the Budapest University of Technology and Economics.

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INVESTIGATION OF PRECAST PRESTRESSED CONCRETE BRIDGE BEAMS

Ľudovít Naď

Utilization of precast elements in the construction industry is well observable in the fifties and sixties of the last century. Slovakia was not an exception. Several thousands of concrete bridges were built from precast prestressed concrete beams of different cross sections (I, Π, □, U) during the last 30 to 40 years. General requirements should be formulated in few points, e.g. high quality of concrete including precise dimensions and decreasing of self weight (because of transport and installation equipments). Last but not least, it is also important from the point of view of decreased dead load.

This slenderness tendency has also a negative part because of potential consequences in load carrying capacity, strength and eventually stability. Therefore, some complementary experimental investigations are often required to confirm the theoretical assumptions. This paper presents some results from theoretical as well as experimental investigations of bridge beams carried out by the Department of Concrete Structures and Bridges of the Faculty of Civil Engineering, Technical University of Košice, Slovak Republic.

Keywords: bridge, prestressed concrete, precast beam, load test

1. INTRODUCTION

After the past century "prefabrication period" of the sixties and seventies, as if a new "renaissance" period put in an appearance during last decade. Experienced bridge engineers developed several precast beams and many successful bridges were designed and built using them. Transversal beam cooperation, which directly relates to a beam shape, belongs to considerable points of innovation and development. Mainly open ribbed bridge cross sections are used. Individual beams cooperation is provided by cast in situ reinforced concrete plate. Actually I–shape and reverse T–shape pre-tensioned and I–shape post-tensioned beams are available for bridge construction.

But we often have to seriously treat several hundreds of currently 20 – 50 years old bridges. A role of bridge engineers then includes two parts – design of new bridges and investigation as well as evaluation of existing old bridges. Both of them are supported and validated by experimental test results. Two investigations, which are different in their theoretical approach as well as utilization time period, will be presented. The first example deals with up-to-date pre-tensioned concrete bridge beam series (IST 97), which Inžinierske stavby, a.s. Košice, Slovak construction company desires to present also to a Hungarian construction market. But the second one is almost 50 years old, so called VLOŠŠÁK post-tensioned concrete bridge beam. These beams were removed from one 35 years old bridge, while the near statically independent bridge span structure was submitted to strong load test *(Naď, 2002; Naď, Krištofovič, 2001)*. Two examples allow us to compare one completely new bridge beam with another one which is in the middle of its service life.

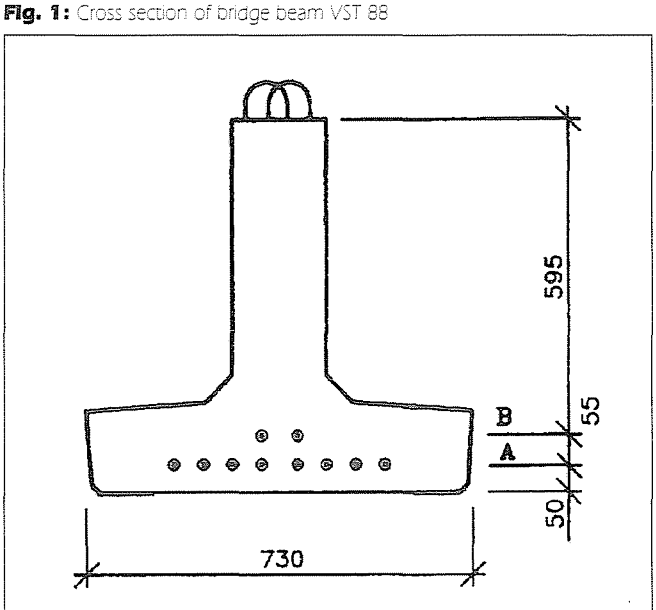


Fig. 1: Cross section of bridge beam VST 88

2. DESCRIPTION OF TESTED BEAMS

2.1 Precast pre-tensioned concrete bridge beam IST 97

Based on older, so called VST 88, prefabricates, this bridge beams were developed by Inžinierske stavby, a. s. Košice in collaboration with Dopravoprojekt, a. s. Bratislava. According to an original design, the VST 88 beams (*Fig. 1*) were placed side by side and mutually connected by one 120 mm thick reinforced concrete slab, which assured the beams cooperation and transversal load distribution.

Later, according to a Road Administration Office requirement, the bridge structure had to be visually totally checkable from below. That was possible only by beams transversal redistribution, by which minimal clear distance between two adjacent beams flanges could be 480 mm. The solution is so called IST 97 beam series (the cross section is shown in *Fig. 2*). The cast in situ reinforced concrete bridge deck thickness

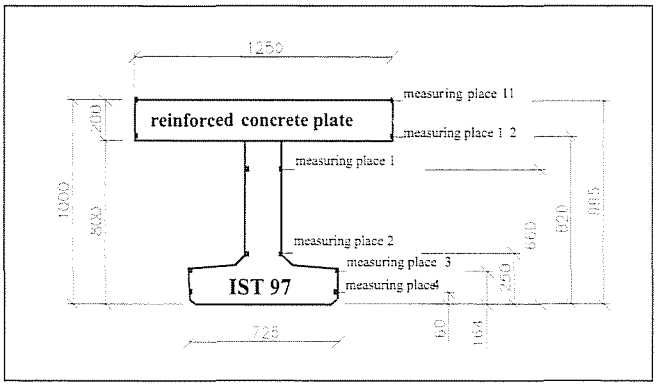


Fig. 2: Cross section of precast pre-tensioned concrete bridge beam IST 97 together with the fragment of cast in-situ reinforced concrete plate

then sufficiently ensures the beams transversal cooperation. Catalogue's typical beams length are : 9, 12, 15, 18, 21, 24, 27 and 30 m. The 27 and 30 m long beams are no more of inverted T–shape, but because of statically needed upper flange, they have I–shape cross section.

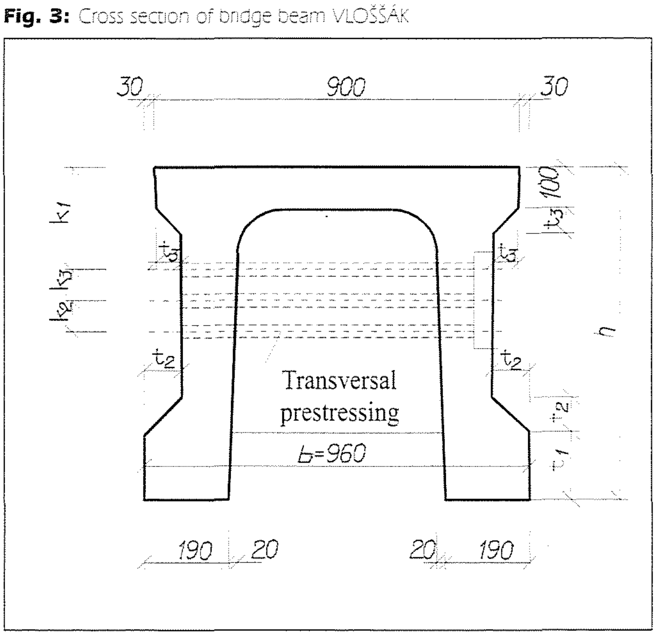
The beams are produced in two 60 m long prestressing stands. The precast beams are made of concrete C55 and the cast in situ bridge deck of C35. The strand nominal strength is 1800 MPa and initial prestress is 1430 MPa.

2.2 Precast post-tensioned concrete bridge beams VLOŠŠÁK

So called VLOŠŠÁK bridge beams were developed in 1956 and almost 800 bridges were built from them during next more than 25 years in Slovakia. They are 30 to 45 years old now and are getting to the middle of their designed service life. This shows success as well as popularity of these beams. The main reason for this was a statically very efficient structure. Optimal material distribution in the cross section can be seen in *Fig. 3*.

It is shown that both, walls and slab are very slender. The concrete cover is only 15 mm thick according to a design (in reality 12 to 13, or only 10 mm in some places). That is very serious problem influencing the structure load carrying capacity to a large extent. Originally, after the placement, the beams were transversally prestressed (every one meter). In this manner, a very stiff structure with excellent transversal load distribution was created. Few years later, because of technological problems (difficulties with placement of transversal prestressing cables), transversal prestressing had to be skipped. The beams connection worked in the manner of hinges. The beams were produced in three length groups: 10-11-12 m, 13-14-15 m and 16-17-18 m of structural span. In case of the liquidation of one 35 years old road bridge, five beams of 21.40 m length (in good technical condition) were gained for experimental purposes. The bridge cross section is presented in *Fig. 4*.

Fig. 3: Cross section of bridge beam VLOŠŠÁK



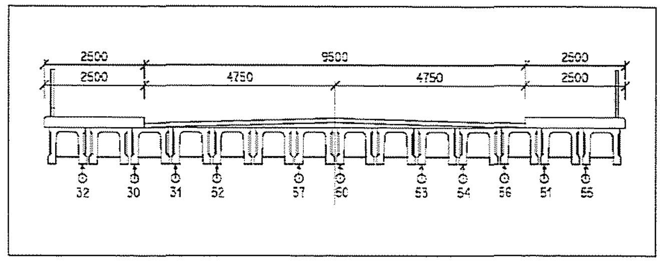


Fig. 4: Cross section of the bridge

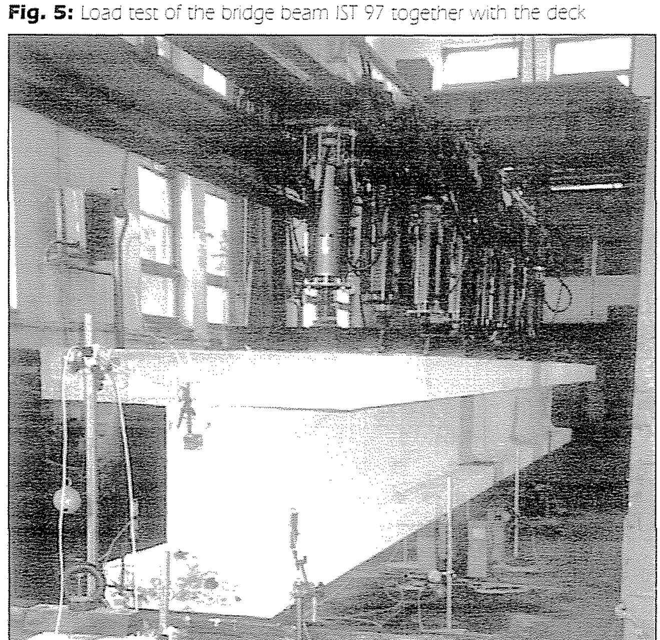
3. WORK APPROACH AND USED EQUIPMENT

3.1 Precast pre–tensioned concrete bridge beam IST 97

The load test was executed according to the plan prepared in the cooperation between TSÚS Prešov and Technical University of Košice. The main load done by IST 97 beams type design (STN 73 6203 "Loading tests of bridges") meant the 100% of bending moment. *Fig. 5* shows a general load set. Measurement of the stress was done in 4 strands by magneto - elastic method using gauges and measuring central.

The gauges had been placed on the strands before their fi-

Fig. 5: Load test of the bridge beam IST 97 together with the deck



nal positioning into the stand. The prestressing of the strands was performed step by step. The calibration diagram of the gauges was created or improved with comparison of data measured by a prestressing equipment. The strands' initial prestressing was 1430 MPa. The beams were cast straight after the prestressing of the strands. The strands stress was measured after the casting was finished and then during next four days. The goal of such measurements was to estimate the influence of a concrete shrinkage on the prestressing force change. The strain of concrete was measured in the mid span cross section by means of Huggenberger measuring device (sensitivity ± 0.001 mm). Target points were fixed on the surface of the concrete in 250 mm basic distance (in course of beam length).

3.2 Precast post-tensioned concrete bridge beams VLOŠŠÁK

The non damaged beams picked up from bridge were loaded in couples. All beams were visually inspected before experimental loading. The loading was performed according to an arrangement shown in *Figs. 6, 7 and 8* – beams were in aboard basic position, linked with rigid steel frames on both back–ends, they were gradually loaded by two hydraulic jacks in mid span. The span of 21.4 m was appropriate for the reality of beams built in the bridge. Deformation (deflection) of the beams was registered in the middle and in the first and third fourth of span by electronic inductive gauges on both sides of beam walls (the measurement places up and down). Target points were fixed on the surface of the concrete in 250 mm basic distance (in course of beam length). The strain of concrete was measured by means of Huggenberger measuring device (sensitivity ± 0.001 mm).

Fig. 6: Load test of beams VLOŠŠÁK – in couples

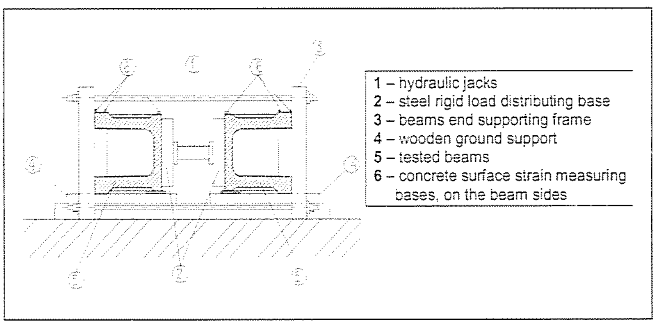


Fig. 7: Preparatory phases of the load test of beams VLOŠŠÁK – in couples

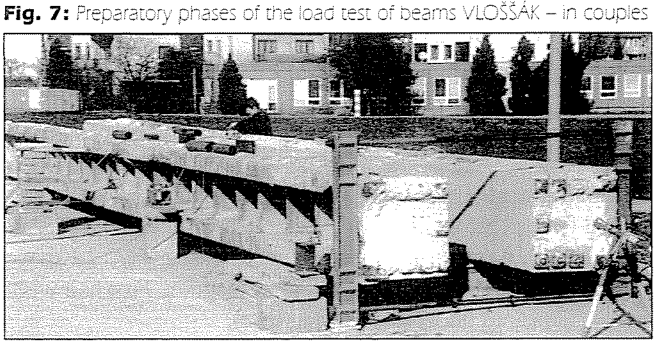
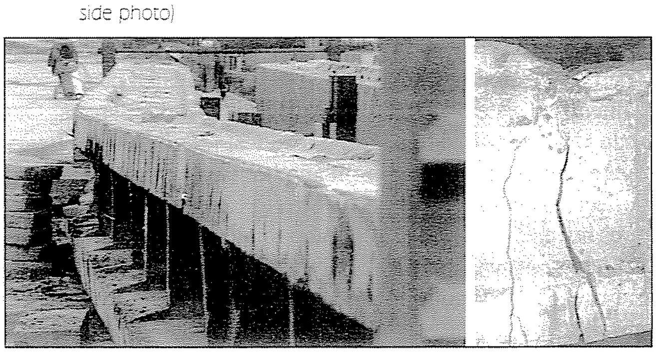


Fig. 8: Final phases of the load test of beams VLOŠŠÁK – in couples (left side photo) and rupture pattern mechanism of the beam (right side photo)



4. EXPERIMENTAL RESULTS

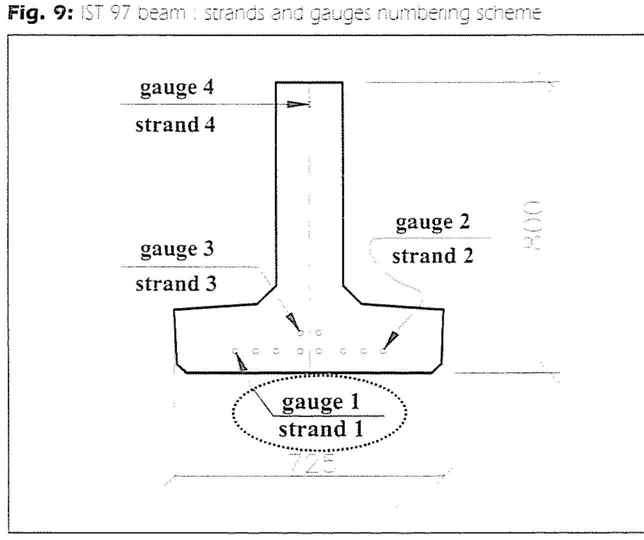
4.1 IST 97 – precast pre-tensioned concrete bridge beam (Nad', 2001)

The results are divided in two groups: before load test and during load test. Until the beginning of load test, the strand stress decreased from initial prestressing of 1430 MPa to 1349 MPa in a strand No. 1 (L1), to 1336.7 MPa in a strand No. 3 (L3) and to 1389.7 MPa in a strand No. 4 (L4). Strands numbering is shown in Fig. 9. State of stress in the beam mid span cross section (inverted T) was determined on the base of a measured strain before and after introduction of prestressing into the hardened concrete in prestressing stand. Just prior to a load test, stress in concrete bottom fibre was $\sigma_{c'bm} = 9.91$ MPa and $\sigma_{c,to} = 7.52$ MPa in a top fibre of reverse T section respectively.

The load test was carried out according to the loading plan prepared in collaboration with TSÚS Prešov. The bending moment corresponding to the main bridge load (Slovak standard STN 73 6203 "Loading tests of bridges") done in the IST 97 type design, represented 100 % of applied testing load. Taking the loading facilities into consideration, the load test itself was executed in three stages. First stage had to be stopped on the load level of 180 %, second stage was interrupted on a 280 % load level. The beam was totally broken under the load above 380 % in the third – final stage.

Fig. 10 shows the stress variation in the strand L1. We can see that under a 280 % loading the L1 stress is $\sigma_p = 1464$ MPa. Last measurement was done just prior to the beam rupture and obtained stress exceeded a nominal strength ($\sigma_p = 1832$ MPa $> f_y = 1800$ MPa). As it is seen in the figure, after a load test finish (concrete crashing in the compression zone), $\sigma_p \approx 1100$ MPa stress was measured. On the diagrams in *Figs. 10 to 13*, the vertical axis represents a relative moment M/M_d,

Fig. 9: IST 97 beam : strands and gauges numbering scheme



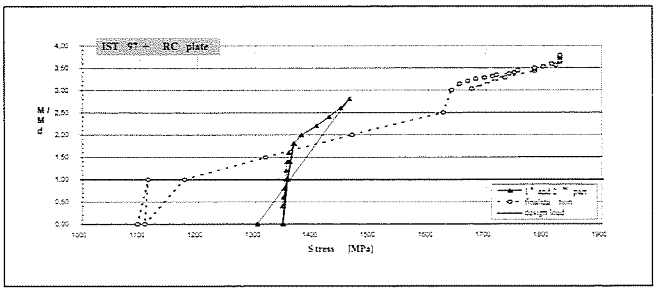


Fig. 10: Result of the test of IST 97 prestressed concrete beam together with RC plate : "relativ moment versus strand stress" diagram

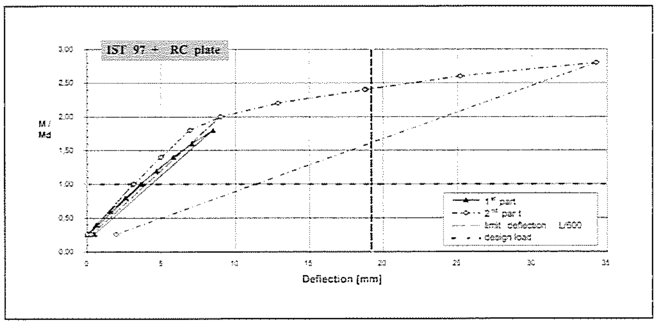


Fig.e 11: Result of 1. and 2. stage of the test of IST 97 prestressed beam together with RC plate: "relative moment versus deflection"

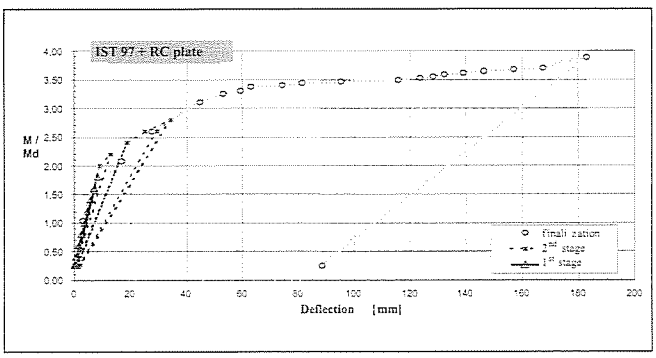
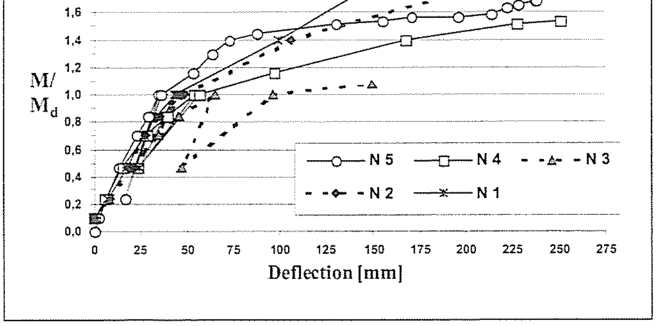


Fig. 12: Result of whole test of IST 97 prestressed concrete beam together with RC plate : "relative moment versus deflection"

where M is a bending moment corresponding to an applied load level and M_d is the design moment (type design real load is $M/M_d = 1.0$).

Next two figures show a beam deflection dependence on a relative moment : 1st and 2nd stage of loading (*Fig. 11*) and the whole load test (*Fig. 12*) respectively. Allowed deflection (according to a standard – L/600) was attained under 250 % of load.

Fig. 13: Result of the test of VLOŠŠÁK prestressed concrete beams : "relative moment versus deflection" (limit deflection l/600 = 35.4 mm)



Stress in the extreme bottom concrete fibre corresponding to a deformation under 100% load level is $(\sigma_{c.bm})_{100\%} = 8.62$ MPa and in the extreme top fibre is $(\sigma_{c.to})_{100\%} = 4.37$ MPa (compression over the whole cross section). Maximal concrete strain under 280% of load is $\varepsilon_c = 0.815‰ < \varepsilon_{c.lim} = 2.5‰$ and a corresponding stress in the concrete is $\sigma_{c.to} = 27.6$ MPa < 30.5 MPa (concrete strength according to STN 731251). Concrete crashing was observed under the 380% of loading.

First crack of 0.1 mm appeared under 200% of load and it remained hair width after de-loading.

A real compressive strength of concrete was 55.1 MPa, tensile strength (under bending) 5.49 MPa at the time of the load test. The real average strength of strands was 1840 MPa.

4.2 Precast post-tensioned concrete bridge beam VLOŠŠÁK (Nad', 2001)

The beams gained from the bridge were loaded in couples: N1+N2, N1+N3, N1+N4, N1+N5. The beam N1 was randomly used with others and all of them were broken. Beam N1 remained unbroken after finish of whole loading tests and there were no technical facilities to attain its ultimate limit state (fracture). *Fig. 13* shows the deflection versus relative moment diagram for all tested beams. Beam N3 had one totally broken prestressing cable and one other was 50 % damaged by corrosion. Only small corrosion traces were observed on other beams.

Maximal chloride ion content was 1.575 % to 4.55 % Cl^-/m_c (most damaged beam). Carbonated maximal depth was 35 mm. Intercrystalline corrosion of prestressing wires was not observed, not even during precise laboratory tests.

5. COMMENTS ON OBTAINED RESULTS

5.1 Precast pre-tensioned concrete bridge beam IST 97

Experimental results obtained during an investigation of a unique beam have shown its high bearing capacity – 3.82 multiple of design moment. These results were compared to the results obtained from older investigation of three beams VST 88 (TSÚS, 1991). Both groups of results are very similar. *Fig. 14* shows the "relative moment versus deflection" diagram. First hair width cracks appeared under 265 % load level.

5.2 Precast post-tensioned concrete bridge beam VLOŠŠÁK

The bridge structure, from which the investigated beams were gained, consisted of three statically independent spans. One span was submitted to a strong load test *(Nad', 2001)*. (Investigated beams originated from another bridge span.) Maximal deflection measured during a strong load test of the bridge does not exceed 5 mm (210 t was a total mass of applied heavy vehicles – it was not physically possible place more load on the bridge. According to a Slovak standard, the heaviest applicable bridge load is 196 t). During an individual beams test-

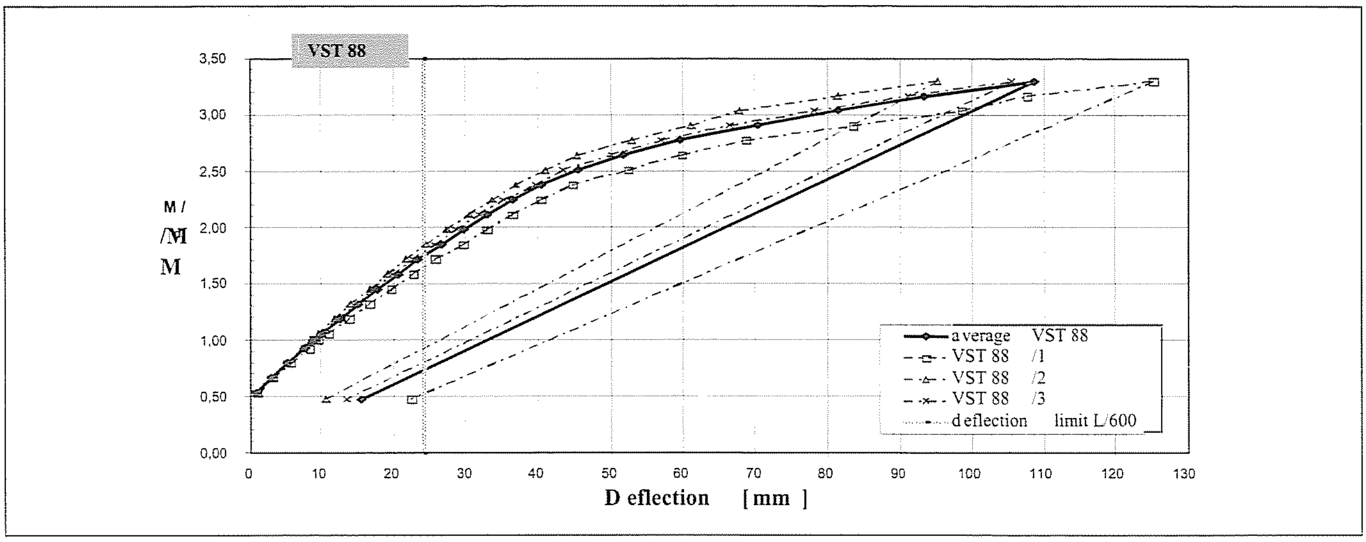


Fig. 14: Result of the test of VST 88 prestressed concrete beams : "relative moment versus deflection"

ing, the deformation of 33 – 50 mm was measured under the beam design moment load level. During the bridge load test no cracks were observed.

During beams load test, the first cracks of average width of 0.1 mm were observed under 80% of design moment. Load bearing capacity was reached under a load level of 185 % in case of the beam N2, 170 % beam N5, 160 % beam N4 and 120% beam N3 (where 2 prestressing cables were initially broken). Beam N1 was submitted to four serious loading programs and after the last one the beam retrieved its initial camber. Its load carrying capacity then largely exceeded 185 % of design moment. Each loading program contained also 10-times repeated loading-unloading cycles (between self weight and 100 % of design load).

5.3 Summary

Number of authors, who are dealing with this problem, can be found on the international level. Even if their opinions should differ, there is one common feature. It is a reliability and serviceability of bridges (*Astiz, 2002*). Above mentioned bridges are under service without damages during several decades, which leads to very promising conclusion. Past experiences implementing in new bridges design seems to be desirable.

6. CONCLUSION

Following the above described arguments we can state that the investigated bridge beams have a large reserve in load bearing capacity and then the bridges built from them have also large reserve. Potential failure found on several decades of years old bridges are sometimes overestimated.

Mainly the transversally prestressed bridges represent very stiff structure. Some individual seriously damaged beams also cannot cause statical problems, because upon transversal cooperation, the internal forces will be redistributed. So, when we consider the ideal road bridge heavy load, according to a standard, it is not physically possible to place it on the short span bridge in reality. From all that, we can conclude, that some re-evaluation or modification of existing evaluation of the bridges built of precast prestressed concrete beams would be useful.

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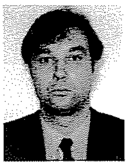
ROLE OF CONCRETE PIERS IN APPEARANCE OF SPECIFIC PIPE BRIDGES IN BUDAPEST



Dr. Béla Csíki

From point of view of structural engineering attractiveness of concrete structures or structural elements can strongly be related to the direct and noticeable relationship between their structural role (behaviour) and architectural appearance. The paper presents an interesting example in this context on the occasion of the fib Symposium "Keep Concrete Attractive" held in Budapest in May 2005.

Keywords: appearance, attractiveness of civil engineering structures, bridges, columns, structural engineering, piers, pipes, pipe bridges.

1. INTRODUCTION

Two specific pipe bridges with parallel axes, both of about 90 m length carrying a pair of waste water pipes over the Danube-bay at Népsziget in Budapest were built in 1997. Both hybrid – steel and concrete – structures were designed by Mélyépterv Complex Engineering Co. Ltd., constructed by Alterra Building Ltd. and Ganz Steelworks Co. Ltd. and have been operated by the Municipal Sewerage Installation Works of Budapest.

The two pipes spanning the Bay at Népsziget take important part in the waste water collecting and carrying system of Northern-Budapest (Bancsik, Nagy and Tóth, 1997). The special requirements and the general considerations of the design of the bay-spanning together with a detailed description of the pipe bridges of very specific structural arrangement are summarized in the paper of Csíki (1997). Aspects of the building technology due to the unusual arrangement to be considered during the whole design process are reached in studies of Csíki (1997 and 2001) and also outlined in a wider context in the paper of Szöllősy, Konkoly Thege, and Mihályi (1997). These specific pipe bridges were mentioned among the most significant civil engineering structures in Hungary at the time (Tassi, 1999).

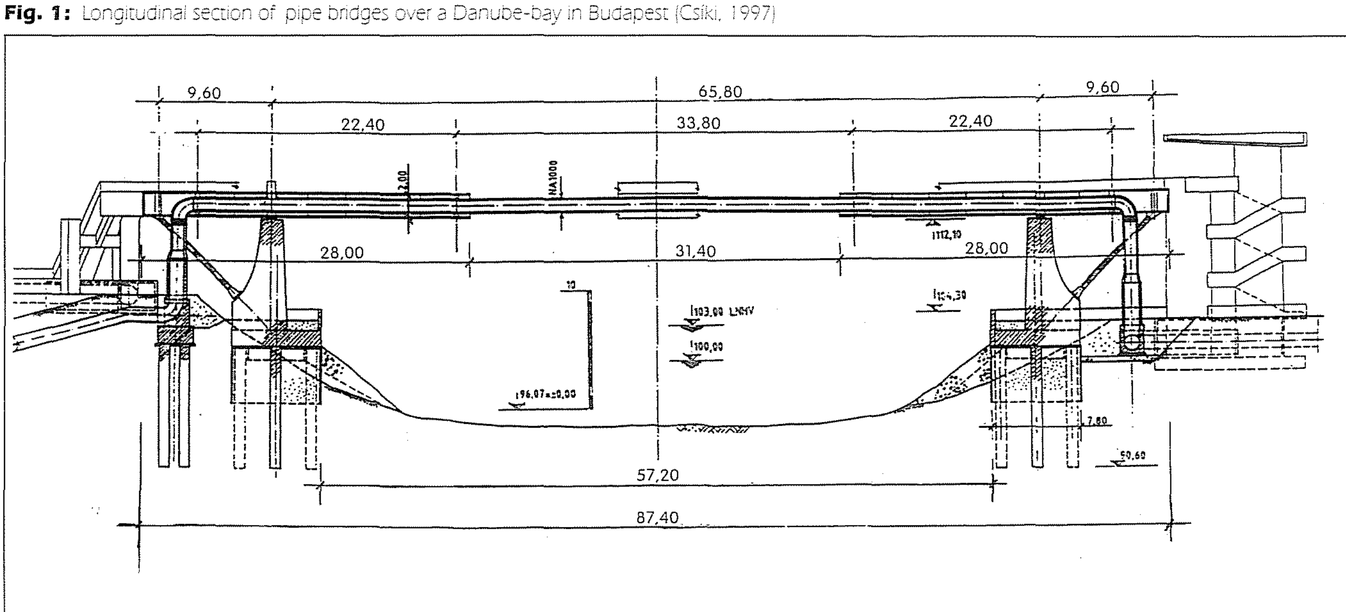
The *aim* of the study – actually on the occasion of the *fib* Symposium "*Keep Concrete Attractive*" held in Budapest from 23 to 25 of May in 2005 – is to present an example where the unusual structural arrangement has resulted in an attractive civil engineering solution. Due to the unique arrangement of the pipe bridges the design of very dominant, carefully shaped, high volume reinforced concrete bank-piers (columns) was reasonable. The main focus will be on the strong connection between the architectural and structural considerations influencing the appearance of the columns (piers) contributing to the attractiveness of the whole structure.

2. GENERAL LAYOUT OF THE STRUCTURES

Before emphasizing the structural role in detail of the reinforced concrete columns a certain overview of the general layout followed by that of the overall structural behaviour of the pipe bridges seems to be necessary.

The **two pipe bridges** with parallel horizontal axes and the same axial heights and length surround symmetrically an ex-

Fig. 1: Longitudinal section of pipe bridges over a Danube-bay in Budapest (Csíki, 1997)



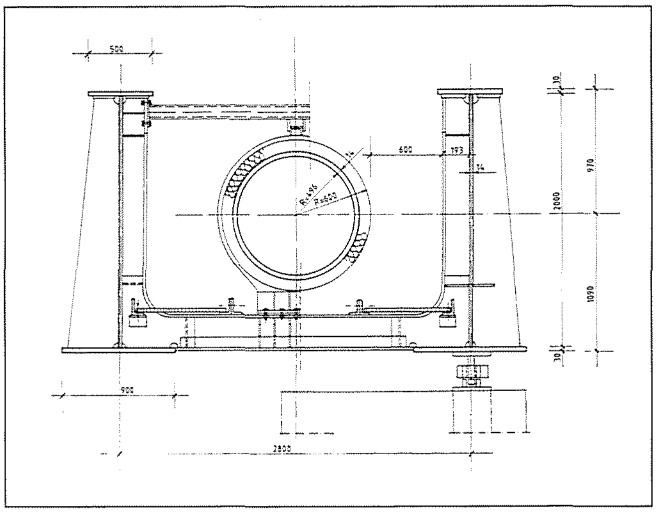


Fig. 2: Cross section of main beam of a bank unit with the pipe (Csíki, 1997)

isting combined foot- and pipe bridge which had been built a few decades earlier. The arrangement of the three bridges is a result of consideration of several factors presented in the study of Csíki (1997) which are beyond the scope of this paper. The longitudinal section of *one* of the two recent bridges is given in *Fig. 1*.

The two structures are of the same length of 87.40 m and of the same specific arrangement. Both of them consists of three main parts: the two joined, supporting (or bank) units on the banks of the Danube-bay and the pipe itself 1000 mm in diameter spanning the bank units. The two supporting units are arranged symmetrically to the middle of the whole structure on the banks.

The **bank units** are assembled from three main structural elements: the eccentric, high volume, 9.60 m high, reinforced concrete pier on pile foundation, the horizontal steel main beam of 28.00 m length - with a 17.20 m cantilever toward the middle of the whole structure - supported on the top of the pier, and anchored at the end to the bottom of the pier by the third element, an inclined steel bar. The main beam and the anchoring bar are joined structures themselves made of welded steel elements. They consist of doubled girders with axial distance of 2.80 m, connected by transverse beams and stiffening bars (*Fig. 2*). The structural depth of the I-shaped cross-section doubled girders of the horizontal main beam and of the inclined anchoring bar are 2.00 m and 0.40 m, respectively.

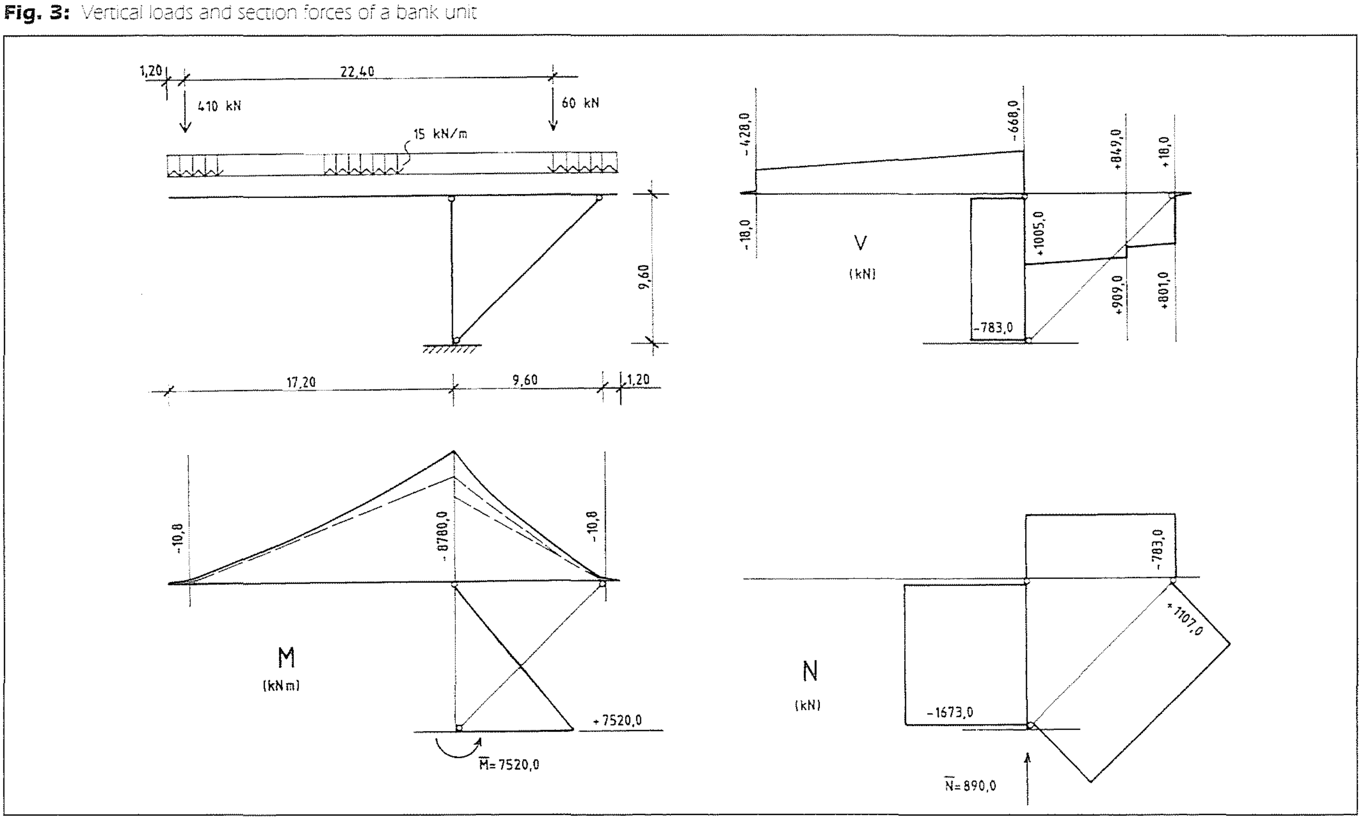
The **steel pipe** at both bridges spanning freely the bank units is hidden in the horizontal main beams at the banks (*Fig. 2*). Statically the pipe - being *independent* structurally from the connecting parts of the network arriving to or leading away the bridge - is a continuous beam of 81.80 m length on four elastic supports. Each bank unit contains two-two pipe-supports in the main beam located on the transverse beams being at the end of the cantilever and at the vicinity of the anchoring bar. The required camber of the pipe was produced by setting a specified difference between the absolute heights of the two pipe-supports on each bank unit before placing the pipe. This difference determined by *thorough stress and strain calculations* could be created by adjusting a prescribed inclination of the "quasi" horizontal main beam containing the pipe supports. The conceptual scheme of this *geometrical adjustment* of the bank units harmonising with the aspects of the assemble of the elements is presented in Ref. Csíki (2001).

3. STRUCTURAL ROLE OF BANK UNIT COLUMNS

The structural requirements to satisfy by the reinforced concrete bank-piers can be classified according to the following aspects:

- Hiding the special steel-to-concrete connections to secure the geometrical adjustment of the superstructure during construction,
- Being able to carry the *combined* loads obtained from the superstructure through the steel-to-concrete connections,

Fig. 3: Vertical loads and section forces of a bank unit



– Ensuring the anchorage of the cantilever main beam and, at the same time, securing a proper load distribution at the top of the pile foundation.

Conditions of the first point are beyond the scope of the present study. A detailed description of the special steel-to-concrete connections – being also in relation with the shape of the columns – is in a previous paper (Csíki, 2001).

The aspects of the *latter two points* were of primary importance when shaping the reinforced concrete columns. A simplified structural model of a bank unit with the *dominant vertical loads* on it (including dead loads and weight of waste water in the pipe, only) and the correspondent section forces along its elements are presented in *Fig. 3*. (For the sake of simplicity the effects of the weight of the column and of the real eccentricities are not included in the figures.)

Concerning the structural behaviour of the *columns* the statements listed below are of most importance basing on the section and reaction forces in *Fig. 3*:

1. Bending moment along the column is *linearly* increasing,
2. Constant shear force vanishes at the bottom of the column (no horizontal *reaction* force from the dominant loads),
3. Value of the bending moment *reaction* is rather high comparing to the vertical *reaction*.

Simultaneous consideration of the points listed at the structural requirements and the points 1 to 3 above concerning the structural behaviour led to the following conclusions:

- Application of columns of *non-constant cross section* was economic because of the increasing bending moments,
- Application of columns of *high volume (weight)* was needed to decrease the eccentricity of vertical loads with respect to the centre of pile-groups used for foundation of the columns,
- For the same purpose, *eccentric placing* (or *eccentric weight distribution*) of the high volume columns with respect to the centre of the pile-groups was reasonable to avoid tension at the outer piles.

Shaping of the bank piers to correspond the previous conclusions is described in the following chapter.

4. SHAPING OF THE PIERS

To secure the *high volume* of the column of the bank units relatively large dimensions in vertical and horizontal directions were needed, obviously. The application of *columns of non-constant cross section* served two different structural purposes. On the one hand, it was advantageous regarding the increasing bending moments. On the other hand, the columns with changing cross sections ensured the proper *weight distribution* needed to keep all piles of the foundation under pressure.

The 9.60 m height of the columns above the top plate of the foundation and the 3.70 m constant width (the measure perpendicular to the axis of the bridges) were almost fixed being in relation with the possible vertical and horizontal line-placing of the pipes at the bay-spanning. The non-constant dimension of the columns being parallel with the axis of the bay-spanning increases toward the bottom from 2.00 m to 4.90 m (*Fig. 4*). According to considerations of *aesthetics* the cross-sectional change is *curvilinear* instead of using linear increase basing strictly on the structural analysis.

The shape of the piers presented above proved to be advantageous concerning both structural behaviour and appearance, moreover substantially contributed to mirroring back the overall structural behaviour of the pipe bridges. In addition, the shape of the columns was suitable to ensure the anchorage of the inclined steel bar of the superstructure, and to hide the special steel to concrete connections (*Fig. 5*) secured the geometrical adjustment of the whole structure during construction.

Fig. 4: View of a pier during construction

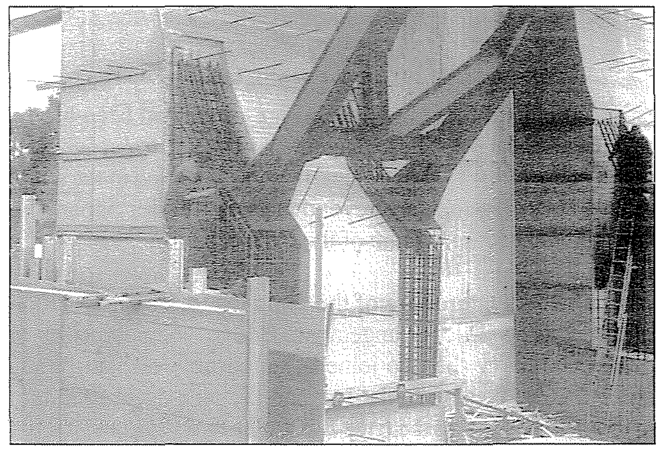


Fig. 5: Bottom of a pier during construction

It should also be mentioned that due to the high volume and the aesthetic aspects of the piers strict concrete technology together with application of specially stiffened reinforcement mesh were necessary to produce the expected quality of the columns. Details of these questions are beyond the scope of the present study.

5. CONCLUSIONS

The structural considerations influencing the architectural appearance of the dominant reinforced concrete columns of two recent pipe bridges over a Danube-bay in Budapest, Hungary (*Fig. 6*) were presented in the paper.

The direct correlation between the structural role and the appearance of the piers and their contribution to the unique appearance of the pipe bridges have been emphasized. The high volume columns of cross sections increasing curvilinearly from the top to the bottom proved to be advantageous concerning both, the structural behaviour and the necessary eccentric weight distribution. The contribution of the piers to attractiveness and economy of the whole structure is obvious (*Fig. 7*).

Finally, the author concludes by emphasizing that at *civil engineering structures* beauty and responsibility of *creating* structures of proper *appearance* are solely "in the hand" of

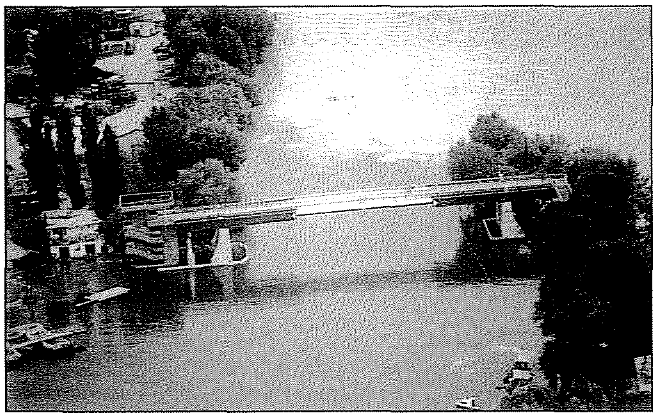


Fig. 6: View of complex pipe bridge system over a Danube-bay in Budapest (Csíki, 1997)

structural engineers. No-one can take over this beauty and responsibility including the much *thinking* needed to design attractive structures.

6. ACKNOWLEDGEMENTS

The author, working for Mélyépterv Complex Engineering Co. at the time must recall his former colleagues whose contribution was indispensable to the realization of these special structures:

Primarily, *Dr. László Tóth*, general manager of the Company, who recognizing the unique nature of the structures strongly promoted the realization at the investor from the very beginning. Furthermore, *István Szabó* head of Structural Department of the Company and all his colleagues who were taking part in the detailing of structural design. The co-designing work by *Ágnes Rokob* and *Miklós Hertelendy* involved in the network and hydraulic design of the 4 km long pair of pipes under pressure - including the bay-spanning at Népsziget – was also of most importance.

Last but not least, the proper cooperation with the delegates of the construction companies listed in the 1st chapter should also be commemorated.

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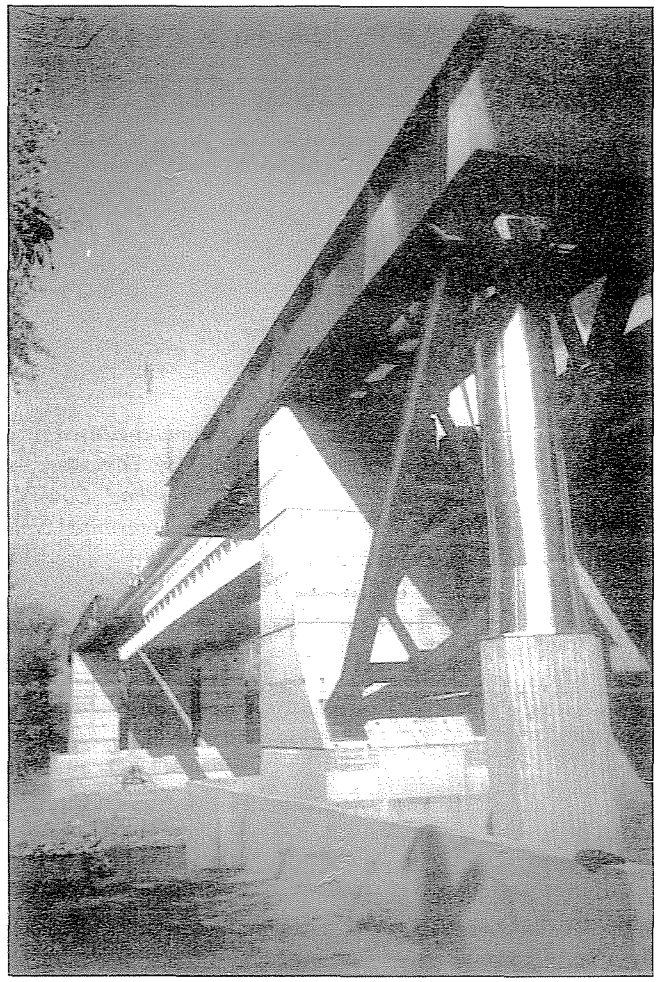

Fig. 7: View of southern pipe bridge over a Danube-bay in Budapest (Csíki, 2001)

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EFFECT OF ELASTIC DEFORMATION DUE TO PRESTRESSING ON THE FORCES IN PRESTRESSED CONCRETE MEMBERS

Prof. Géza Tassi

It seems necessary to study again the phenomena caused by elastic deformation in prestressed concrete elements. In the light of this, the problem of losses is to be discussed, too, The paper emphasises the importance of exact definitions and the role of precise terminology, furthermore of the calculation method. Considering the losses, there is a difference between the various types of prestressed pre-tensioned and prestressed post-tensioned concrete structures. The article shows calculation procedures to follow the effect of elastic deformation of concrete.

Keywords: Elastic deformation, prestressing force, losses, pre- and post-tensioning

1. INTRODUCTION

Analysing the forces and deformations in load bearing structures, generally, we suppose that the construction materials behave as linear elastic at the beginning of the application of external loads up to a state far of failure. We accept this statement even in case of concrete knowing that in initial level of short term loads the stress-strain relation differs slightly from that which corresponds to the generalised Hooke's law. Earlier and new codes (e. g. Hungarian codes *MSZ* 1970, *MSZ ENV* 1999) prescribe the application of methods of linear elasticity – among other cases – for determination of forces in statically indeterminate prestressed concrete structures. The effects caused by prestressing are analysed using the elastic theorem in serviceability limit state (besides reckoning with rheological phenomena).

Discussions were continued in professional circles about the interpretation of the change of prestressing force caused by elastic strains in P. C. members. We have to unambiguously give first the definitions for the sake of a precise answer on the question. Based on these, we can explain consequently whether we meet a loss and how to include it in calculation.

It became clear at the time of earliest emerging of the idea of prestressing that the prestressing force decreases related to the force exerted by the tensioning device. Initially, there was made no difference in principle between the different effects causing decrease of prestressing force, At the beginning of the 20th Century, conditions were not yet ripe to produce P. C. elements having the waited theoretically provable advantageous properties. Also, the calculation methods started to develop later. It was known that the prestressing force decreases because of elastic deformation of concrete (except a few special cases). The engineering society gained more and more knowledge about the phenomena causing losses (the relaxation of prestressing steel, the shrinkage and creep of concrete etc.). Many research works brought to light the formation of the effective prestress, various calculation methods were introduced. There remained beside these discussed questions, definitions without general validity, and the instructions for practical design were incomplete.

In connection with the latter, it is to be noted that sometimes it seems unessential to give a precise definition and an exact calculation upon correct basic assumptions. Sometimes, disputed problems are meaning rather smaller uncertainties in numerical results than the influence of parameters substituted in "exact" formulae of several computation methods. However, the clearing of principles is not vain.

2. A LITTLE DETOUR ABOUT THE TERMINOLOGY

The Department of Bridge Construction No. ii. at the Faculty of Civil Engineering in Budapest described in details the two basic processes of prestressing. This was in 1957, before the elaboration of the Hungarian codes of that time. Until that year, relatively long time before, on the example of the French "béton précontraint", the English "prestressed concrete"; the German "vorgespannter Beton" (which was used already then in the shorter form "Spannbeton"); and the Russian "predvaritel'no napryazhennyy beton" or shortly "pred-napryazhennyy beton" were created. In Hungary, the "feszített beton" (in metaphrase "stressed concrete") was introduced, furthermore the "feszített vasbeton" ("stressed reinforced concrete"). The expression "reinforced concrete" didn't want to refer to the fact that in the beam there can be normal reinforcing bars (generally there are). The reason for this use of words is to express that it is spoken about such a reinforced concrete structural element, where internal forces, stresses are acting without application of external loads or loading effects (disregarding the influence of shrinkage, etc.). The Hungarian terminology corresponded to the more precise, although longer Russian version "predvaritel'no napryazhennyy zhelezobeton".

Let us disregard those seldom occurring structures, where the prestressing force is not exerted by tendons but e. g. external presses, wedges or similar.

I don't want to recall deeply the discussion on "iron-steel". The German terminology changed many years ago the word "Eisenbeton" ("iron-concrete") for "Stahlbeton" ("steel-concrete"). The Hungarian use became current as "betonacél" ("concrete steel"= reinforcing steel), "vasbetét" ["iron inlet" = (internal) reinforcement], "vasalás" ("ironing" = reinforcement).

The English verbal prefix "pre-", the German "vor-", the Russian "pred-" refer to it that the stressing precedes the acting of external loads. (Not to complicate here the question, let us omit the fact that in majority of cases the self weight of the member acts simultaneously with operating the jack(s) because of the camber due to prestressing.)

It would be needed to agree about the terminology of non-metallic reinforcement and prestressing tendons. The Hungarian word "betét" ("inlet"), and "feszítő betét" (prestressing inlet) would be acceptable. Only the external tendons "hang out" from this circle. The question arises: Is it compulsory to be wedded to material type word for word? When I am writing these lines, whether eye glasses are helping my weak vision, or "eye acryl" maybe "eye plastic"? Or let us use the material-neutral "gig-lamps" (U. K.), "spects" (U. S. A.) or in Hungarian "pápaszem" (="pope-eye") familiar words? Whether my grandmother didn't iron the body linen, or, she "brassed" it, because her tool to smooth away the creases was made by chance of brass? I could continue the joking very seriously. Notwithstanding, I am not able to judge, but even not to formulate a good suggestion. Anyway, it would be good to agree in principles and terminology in Hungarian. How lucky are French and English speaking engineers. We can envy the expressions "béton armé" and "reinforced concrete". These declare the essential situation without touching the question of the material which is strengthening the concrete. It would be convenient if the commission work suggested by *G. L. Balázs* could solve Hungarian terminology-problem under the chairmanship of *A. Erdélyi*. Much help could be contributed by specialists, modern technical *Ferenc Kazinczy* (1759-1831) or *Gábor Szarvas* (1832-1895), who cultivated the Hungarian language so much.

It is a further question of terminology in the topic of prestressed concrete the relative moment of the tensioning of steel and casting of concrete. As it is well known, there are two basic cases: The tendons are tensioned before or after the hardening of concrete. (Let us omit here the possible third case when the tensioning of steel and the hardening of concrete proceed simultaneously, using expanding cement.)

In Hungarian, there is really no equivalent to the already mentioned expressions created by the English, German, Russian prefixes "pre-", "vor-", "pred". It does not matter, we speak simply about "stressed structures". The "pre-" ("elő-") and "post-" ("utó-") prefixes are only used to show the relative moment of tensioning and casting. In this question, the English expression is exact and concise, i. e. the terms "prestressed pre-tensioned" and "prestressed post-tensioned". Misunderstanding is avoided, however, not frequently used because its length, by the Russian expressions "predvaritel'no napryazhennyy zhelezobeton s natyazheniem armatury do zatverdeniya betona" ('previously stressed reinforced concrete with reinforcement tensioned before the hardening of concrete') and "predvaritel'no napryazhennyy zhelezobeton s natyazheniem armatury na beton" ('previously stressed reinforced concrete with reinforcement tensioned on the concrete'). In the German language, the versions of these expressions properly speaking do not exist. The term "Spannbeton mit sofortigem Verbund" ('stressed concrete with immediate bond') and "Spannbeton mit nachträglichem Verbund" or "... ohne Verbund" ('stressed concrete with subsequent bond' or '... without bond') refer to the connection of prestressing steel to concrete. We understand what the question is, but the German technical terms do not express actually the relative time of tensioning and concrete casting.

There is no use in Hungarian for the expressions "précontrainte", "prestressing", "Vorspannung", "prednapryazhenie" (French, English, German, Russian). The substance would be properly expressed as "előzetes megfeszítés" ='previous stressing'. However, this did not come into usage, mainly because its length. The simple term "feszítés"='stressing' fills well its part. Nominating the structure or structural member, we say "feszített tartó, feszített szerkezet" = 'stressed girder, stressed structure'.This makes its effect that the structure is in self stress state, i. e. internal forces are acting even if no external effects are present. The diverseness of the terminology (or maybe the uncertainty over decades) is to be seen even in titles of the papers in the reference list of this article. As an example, how to be bewaring of verbatim translation, is the German word "Nachspannung". This could mean in Hungarian word by word "utófeszítés" = 'post-tensioning'. However, in the German terminology this means the subsequent adjustment of the prestressing force in post-tensioned cables to correct the effect of losses etc. As already mentioned, there is no exact German equivalent for "előfeszítés" = 'pre-tensioning' and "utófeszítés" = 'post-tensioning'.

The used Hungarian technical terms were fixed by *E. Bölcskei* in 1957, after a discussion with linguists of the Hungarian Academy of Sciences. There were some not significant changes after that time, as seen in Hungarian standard *MNOSZ 1957* and *MSZ ENV 1999.*

The expression "utófeszítés" = 'post-tensioning' is unambiguous in Hungarian language. We apply it when the tensioning of cables proceeds after the hardening of concrete. The tendons can be placed in ducts, grouted or slipping on the wall of the duct, or in grease covered by plastic hose. The tendons can be led free, over deviators (saddles) inside of box girders or fully free, maybe outside of the concrete girder.

Let us disregard now the few structures where pre-tensioning and post-tensioning are both present.

3. THE CLEARING OF NOTIONS

For the sake of the clear and simple discussion of the question put in the title of this article, let us suppose the following. Both the concrete and steel are linearly elastic, and no other effects (relaxation, shrinkage, creep etc.) occur.

The primary aim of prestressing is to create a stress state which is advantageous from the point of view of resisting loads and other effects. The unloaded structure is free of stresses and strains under the conditions described in the first paragraph of this chapter. We meet such a definition in the technical literature *(Dulácska, Polgár, Szabados, 1989)* as "The designed prestressing force is a force which is produced in the prestressing reinforcement at the completion of prestressing." This definition is correct in that sense, what the Authors explain i. e. when considering the so called technological losses (due to wedge slipping, etc.). Besides the mentioned conditions, in case of pre-tensioning the exerted prestressing force is equal to the force which is to be reckoned with in the calculation. Speaking about post-tensioning, the definition is to be completed or modified, if we speak about the prestressing force to be considered at the accurate calculation and not about the force produced by the tensioning device.

Obviously, the definition can be formulated as follows: *The prestressing force is the force acting in the prestressing reinforcement, when the concrete is free of stresses.* Considering the above conditions, it also can be said: *The prestressing force is the force in the prestressing reinforcement belonging to the strain free state of the concrete.* We repeat frequently, but it has to be emphasised, that we suppose that the condition in the first paragraph of this chapter is valid.

The prestress is the stress corresponding to the prestressing force defined above. Undoubtedly, the loss due to elastic deformation is the value by which the force acting in the prestressing steel is less than the force according to the above definition of this paper. This value can be equal or not equal to zero depending on the prestressing method.

The prestressing force exerted by the prestressing device (mostly hydraulic press, "prestressing jack") is not equal in all cases to the force corresponding to our definition. Generally, in the practice, the former also is called prestressing force. Usually, this originates no significant error. Nevertheless, for clearing the notions, in the following the force produced by the prestressing device (e. g. read by the manometer of the press) will be called as *exerted force.* This is called sometimes 'effective prestressing force', but this can be confused with the word used for the prestressing force decreased by the losses.

This paper only analyses the effect of the elastic deformation, however, it seems to be necessary to formulate here the notion of loss of prestressing force and prestress.

The loss of prestressing force in a moment, at a point of a prestressing tendon is the difference between the force exerted by the prestressing device and the force at a place of the tendon, at a given moment in case of no external load. In this difference, the change is not to be calculated between the force which occurs at the completion of the tensioning procedure in the member behaving according to linear elasticity, and the force belonging to the deformation-free state of concrete.

As a matter of fact, generally we cannot refer to concrete free of strains. Let us think of the relaxation of steel, which means even a decrease of stress in case of permanent length.

The formulated definition intends to consider many points of view, therefore, it is too long. I would not devote it to everyday use, only for clearing the notions.

Naturally, it can be needed to formulate more exactly the notions playing role in the definitions. Here, let us note only, that now we consider no difference between the designed and perfectly constructed structure. This way, the notion of the exerted prestressing force corresponds to the formulation cited in the 2nd paragraph of this chapter *(Dulácska, Polgár, Szabados, 1989).*

The precise formulation of the prestressing force decreased by losses, and that of losses themselves arises further questions. The various phenomena, the deviation from the idealised state, the different ways of considering of losses at various calculation methods need a detailed analysis, which exceeds the frames of this paper. It is but suitable to improve the chain of ideas. The closing subordinate clause of the definition given in this paper can be disputed, mainly, because the calculation method is connected with the message of this article.

Notions will be mentioned also in other chapters of the paper.

4. THE ELASTIC DEFORMATION CAUSED BY THE PRESTRESSING FORCE AND THE STRESSES IN BONDED PRESTRESSED PRE-TENSIONED MEMBER

The codes in general and literature resources (e. g. *MNOSZ 1956, Code for Highway Bridges, 1956, Szalai, 1984)* contain the procedure according to which the internal forces and stresses are to be calculated based on theory of elasticity.

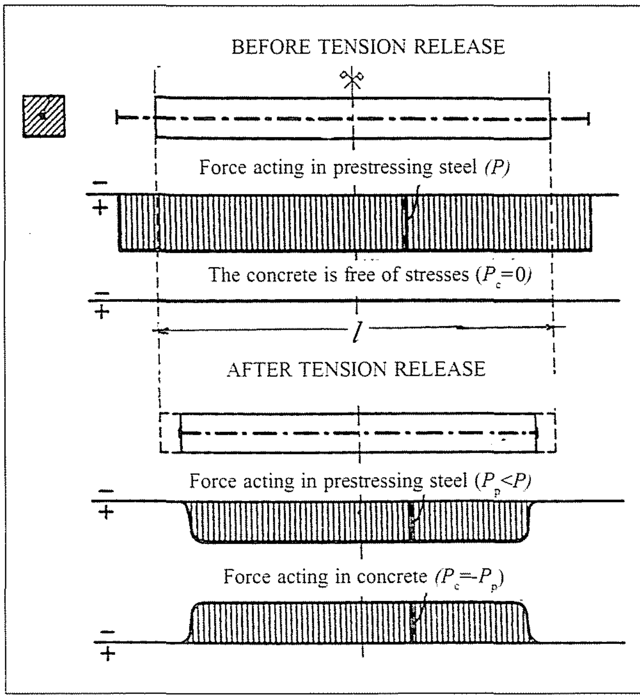

Fig. 1: Internal forces of prestressed pre-tensioned concrete element at tension release

Let us disregard all the losses as defined in Chapter 3. The deformation and the stresses of the elastic member are to be analysed only. The force (and stress, resp.) acting in the tendon fixed to the prestressing stand (or other means) at tensioning is itself the prestressing force (the prestress resp.) according to the definition.

Let us suppose (as it is the case generally at all well designed and manufactured elements) that the prestressing force is transmitted at tension release along a relatively short length at both ends of the member. There is no relative displacement between prestressing steel and concrete at the section between the transmission lengths. For sake of simplicity, let us consider a beam with centric prestressing *(Fig. 1).*At tension release (in German 'Entspannung', in Russian 'peredacha napryazheniya na beton') the prestressing force is acting only. From the equilibrium of forces acting in the steel and the concrete, furthermore the equality of the shortening of steel and the concrete after tension release, the stress in the concrete and in the steel can be calculated *(Tassi, 1957, Bölcskei, Tassi, 1984,)* It can be simply shown that in a prestressed pre-tensioned member the concrete stresses can be calculated dividing the prestressing force (considered as external load) by the ideal (reduced) cross section of the member (reckoned with the concrete cross section and added to it the prestressing steel cross section multiplied by the modular ratio α, i. e.

$$\sigma_c = \frac{P}{A_i} \text{ where } A_i = A_c + \alpha A_p. \qquad (1)$$

The elastic concrete shortening which corresponds to this stress is gained dividing it by the Young's modulus of concrete. Multiplying this strain by the Young's modulus of prestressing steel, the decrease of the prestress in the steel at tension release is given as follows:

$$\delta\sigma_p = \alpha\sigma_c = \alpha\frac{P}{A_c}. \qquad (2)$$

This decrease can say by no means a loss. Surely, the aim of prestressing is the compression in advance of the concrete, and this is unavoidably accompanied by the shortening. At tension release *(Bölcskei, Tassi, 1984, Szalai, 1988)*, it follows from the above definition of the prestressing force that we cannot speak about loss in connection with this question. Namely, the stress decreased by the value defined as (2) does not belong to the strain free state. If such an external force is applied on the centrally prestressed concrete member, which results in zero stress in the concrete, the steel stress increases to the prestress.

It is easy to understand that the situation is principally not else, if the prestressing force is eccentric. Naturally, it has to be considered that in majority of cases the self weight starts acting at tension release because of the camber. Furthermore, to the rehabilitation of the strain-free state of concrete the action of an eccentric external load – or a centric tensile force and a positive moment – is needed (thinking of a simply supported beam).

Omitting a wider range of literature survey, let us refer a few resources.

A chapter of the first Hungarian book on prestressing *(Menyhárd, Vajda, Gábory, 1952)* relates the losses in prestressed pre-tensioned concrete elements. Among these losses the decrease because of the elastic deformation of concrete, according to sense, is not listed. However, the chapter uses the expression "elastic loss", but this does not cause any misunderstanding in practical design. Namely, it is noted that the prestressing force determined by the "actual steel stress" is to be considered, if the characteristics of the cross section are not calculated as those of the ideal (reduced) one but only of the concrete cross section.

The textbook "Reinforced Concrete Structures I" *(Kollár, 1997)* does not define the prestressing force, but the hint for calculation is exact. It refers to the fact that the change of stress in prestressing steel is given automatically by applying the ideal cross section, and, of course, the loss does not emerge at all. (Only the difference – from the discussed point of view – between the prestressed pre-tensioned and by subsequent grouting bonded prestressed post-tensioned members needs a correction to be more precise) Similar conclusions can be drawn from other recent publications *(e. g. Jankó, 1999)*.

In the light of all these, it can not be interpreted in the Hungarian version of the *MSZ ENV 1999* the expression "ΔP_c - the loss due to elastic deformation", and it it is a correct comment referring to this *(Farkas, 2001)*, that the accepted interpretation of the prestressing force, in case of prestressed pre-tensioned elements $P_c = 0$. Following the collision of different concepts, opinions, taking care of phenomena, principles and calculation models, the Hungarian standardization will put in process the solution of the question.

5. LOSSES OF PRESTRESS DUE TO ELASTIC DEFORMATION IN POST-TENSIONED MEMBERS

5.1 The interpretation and definition of the prestressing force

It is reasonable, if the interpretation of the prestressing force does not differ from that which was defined in Chapter 3. However, the deviation in the values of design because of the inaccuracy of the definition amounts only a few percents.

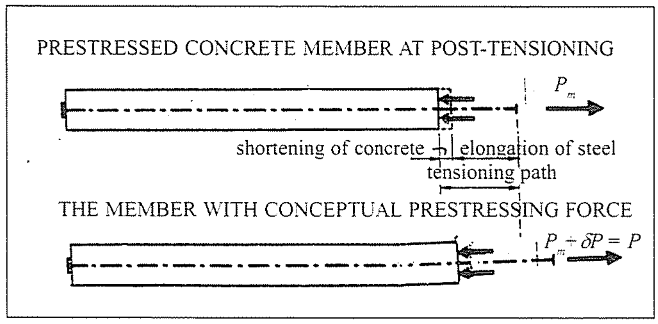

Fig. 2: The interpretation of the conceptual (per definitionem) prestressing force in case of prestressed post-tensioned element

Let us see first such a prestressed post-tensioned member, acting on it a single centric prestressing force *(Fig. 2)*. (To remember: Also now, it is supposed that all the materials are linearly elastic and no other losses occur.) If the adjusted dynamometer of the prestressing device indicates at post-tensioning value P_m, we have to know that this is not the value according to the mentioned definition, but a less force. Namely, during the post-tensioning process, the prestressing force is reacted by the concrete (reinforced concrete) member itself. This means that a compression rises in the concrete, the element will be shorter. The conceptual, *per definitionem* prestressing force is received if an external tensile force is acting on the member, which produces the decompression state, i. e. zero stress (and strain) in the concrete. We can even understand this that way that the prestressing force according to the definition is such a force which acts as pre-tensioning force in case of a prestressed pre-tensioned member, and causes the same concrete stress after tension release (of course disregarding the anchorage zones) as it is present in the prestressed post-tensioned member *(Bölcskei, Tassi 1984)*.

The conceptual, *per definitionem* prestressing force used to be called *fictive* prestressing force. However, this is not a lucky expression. It is true that this force is fictive, imagined in that means, that it generally does not occur. This is the force which is to reckon with at the analysis of the girder. As mentioned, the force exerted by the prestressing device is also called as effective prestressing force. It is true; this is really the force acting in the tendon (disregarding the losses), but as it was explained in Chapter 3, this can be misunderstood.

The discussed difference is denoted as δP, i. e. $P = P_m + \delta P$.

The interpretation of prestressing path enlightens well the stress state produced by prestressing *(Klatsmányi, 1978)*. The shift registered at the prestressing device, e. g. the slip between the piston and the cylinder of the jack, consists of two parts. One of these is the elongation of the prestressing tendon; the other is the shortening of the concrete member. The post-tensioned member can be calculated accurately analogously to the pre-tensioned element, if we adjust the prestressing force in the analysis to the deformation-free state of the concrete.

The difference between the *per definitionem* (conceptual) prestressing force, and that exerted and controlled by the jack, in case of a member centrally prestressed by a single cable (or uniform bundle of tendons) can be simply calculated. The difference can be about 8% of the exerted prestressing force, if the stress in the concrete due to prestressing is approximately the half value of the cylinder strength of concrete. (Of course, the conceptual force is larger than the exerted one.) In customary post-tensioned girders, this difference is significantly less and it is not essential for the practice. Well, the standard deviation of the losses, the geometric date etc. in sum mean a much higher inaccuracy. Therefore, it can be accepted for the practice, that instead of the "accurate" conceptual prestressing force, the exerted one is introduced in the calculation (of course taken into consideration the losses). In case of following experimental work by calculation and other structures demanding highly exact level, it is advisable to calculate the conceptual prestressing force. There is a difference between bonded and unbonded structures, but this is only mentioned and not discussed here.

Let us note that in more sophisticated cases occurring in the practice, (more tendons with changing, various eccentricity etc.) the calculation of the conceptual prestressing force is even more complicated. In these cases, also if we need a more accurate calculation, approximate methods are suggested (e. g. substituting cable and eccentricity), which are usual in calculating of losses, too.

5.2 The determination of the loss of prestress

We deal with loss due to elastic deformation if the conceptual prestressing force (belonging to the deformation-free state of concrete) and the corresponding stress in the tendons are less than the values corresponding to the exerted force (emphasizing that now we disregard other effects).

It is well known that in the application of a single prestressing tendon, or if all coaxial cables are simultaneously post-tensioned, no loss occurs. It is to be mentioned that this is not only a theoretical case. We meet this – among others - in case of the gap widening method *(Leonhardt, 1962)*.

Let us study an example where we meet more, theoretically centric, uniform tendons, each of them subsequently post-tensioned by the same force. The member will be shorter as the influence of the firstly post-tensioned cable. A conceptual prestressing force can be calculated as the result of the exerted force operated by the jack. This conceptual force is larger than the exerted one, as shown in *Fig. 2* of Point 5.1. The same proceeds in the time-sequence of tensioning, at tensioning of the second cable. However, when the concrete (reinforced concrete) member is compressed because of the tensioning of the second cable, the first, already anchored cable will be shorter, too. According to this, the force in it will decrease, and naturally also the conceptual force *(Bölcskei, Tassi, 1984)*.

The number of cables is denoted as m, the in sequence of tensioning i-th cable will be shortened m-i times. The measure of the shortening due to the tensioning of one cable is

$$\varepsilon_{el} = \frac{P_1}{E_c A_c}. \tag{3}$$

This means that

$$\Delta\sigma_{el,i} = (m-i)\frac{E_s P_1}{E_c A_c} = (m-i)\alpha\frac{P_1}{A_c}, \tag{4}$$

and the total loss of prestress, the mean value of losses occurring on m cables will be

$$\Delta\sigma_{pe,i} = \frac{m-i}{2}\alpha\frac{P_1}{A_c}. \tag{5}$$

As a matter of fact, if a centric tensile force is applied on the member as long as the concrete will be free of strain, the conceptual prestressing force will act *(Fig. 3)*. Furthermore, let us note that the case will not be discussed here, when the ducts are subsequently grouted during the full post-tensioning process, and the injected mortar is hardened before the tensioning of a later cable. The change of the ideal (reduced) cross section because of grouting is not discussed.

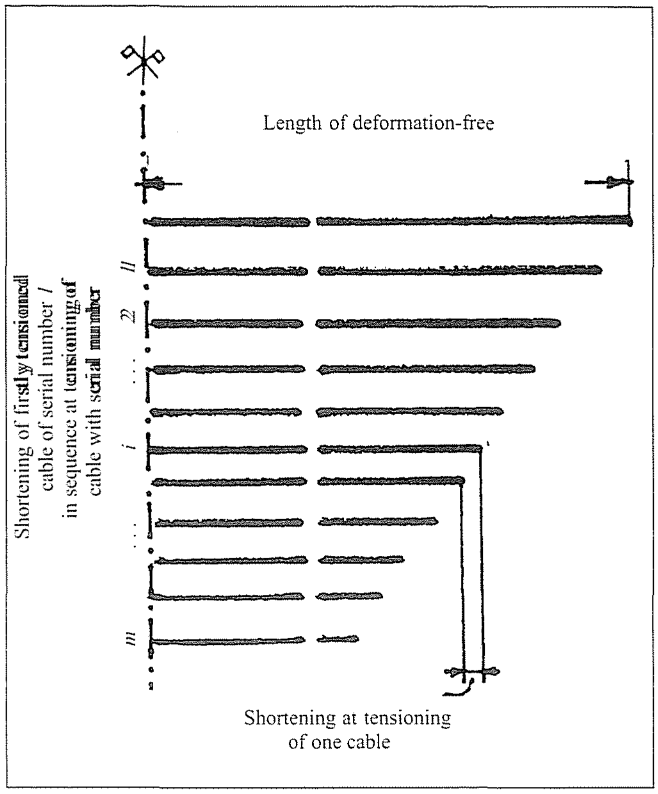

Fig. 3: Scheme of the loss and of the conceptual prestressing force in case of post-tensioned member

The exact calculation is complicated if more cables are subsequently tensioned, and the eccentricity of them is various. For a uniform calculation, the substituting cable and eccentricity can be applied as an approximation. The approximate value of the loss of prestress using these data can be written as

$$\Delta\sigma_{p\,el} = \frac{m-i}{2}\alpha\frac{P_1}{A_c}\left(1+\frac{z^2}{i_c^2}\right) \tag{6}$$

The conceptual value of prestressing force is gained even in case of eccentric, subsequently tensioned cables on the following way.

Also in this case, we speak about *cable;* this means an individual tendon or a group of cables which are equal in all aspects. The material, cross section, layout is uniform as well as the force exerted at tensioning. Performing this calculation all other losses are disregarded, so the loss due to friction and slip at the anchoring device, too.

A prismatic concrete (reinforced concrete) element of length l is post-tensioned by centric cables. The force produced by one cable is P_1. The cables are subsequently tensioned and anchored. The decreasing of the force in each already tensioned and anchored cable due to the post tensioning of the next tendon is ΔP_1.

If there are m cables, and in sequence the i-th one is tensioned, the prestressing force acting in the cables is

$$iP_I - \left(\frac{i}{2}\right)\Delta P_I. \tag{7}$$

The second term of this expression is obviously the loss of prestressing force after tensioning of i cables. This means the same what was earlier explained.

I do not know any data in the literature for the value of the conceptual prestressing force considering the losses due to elastic deformation.

If an external axial centric force is acting on a centrally post-tensioned member, resulting a state free of deformation (and of stress), after post-tensioning of m cables, the force in cable tensioned in i-th place of the series will be *(Fig. 3)*.

$$P_\mathrm{I} + (m - i + 1)\,\Delta P_\mathrm{I}.$$

The conceptual prestressing force after tensioning of all m cables is

$$iP_\mathrm{I} + \frac{i(2m+1-i)}{2}\Delta P_\mathrm{I}. \tag{8}$$

This force is less than the conceptual prestressing force in case of simultaneously tensioning of m cables, which value can be calculated as Eq. (7). The difference between the forces received from Eqs. (7) and (8) means the loss - due to elastic deformation of concrete - of the conceptual prestressing force. Comparing this value with the loss of exerted prestressing force in Eq. (5), expressed in force, the difference between the conceptual and everyday used data can be defined.

As already mentioned, the difference between the prestressing force used in practical design and the conceptual value is not significant, and comparable to inaccuracies in different steps of calculation. This way, there is no reason to introduce the application of the conceptual prestressing force for the practical design. Furthermore, in practical cases, the calculation is more complicated and the difference is smaller than in the above discussed case of centric prestressing.

It is known that a post-tensioned simply supported girder is internally statically indeterminate to the m-th degree, if m is the number of anchored but not grouted cables. If in consequence, the i-th cable is tensioned, the pre-tensioning force in this cable acts on a system which is statically internally indeterminate to the i-1 degree.

5.3 Examples for calculation of losses due to elastic deformation of concrete

Let us analyse elementary examples, where the girder is calculated as statically internally indeterminate system.

The first example is a simply supported concrete (reinforced concrete) girder with uniform cross section. For sake of simplicity, let us apply two cables only. The sequence of post-tensioning is given by serial numbers *1 and 2* (*Fig. 4*).

The force in primary pre-tensioned cable does not provoke any redundant quantity. However, after tensioning and anchoring of the first cable, concerning the further loads (included post-tensioning forces) the member behaves as a system statically indeterminate to the first degree. But, for the time being, there is no question about losses as explained in Point 5.2. When the second cable is tensioned, the girder post-tensioned by one cable suffers a deformation and the force in cable *1* will be changed. This change is decreasing generally.

Solving the problem by the force method, the primary system is taken advantageously by cutting the cable *1*. So, the calculated redundant quantity is the loss itself. Of course, the load is the force which is exerted by tensioning the cable *2*.

In the simple examples shown in *Fig. 4,* there are taken

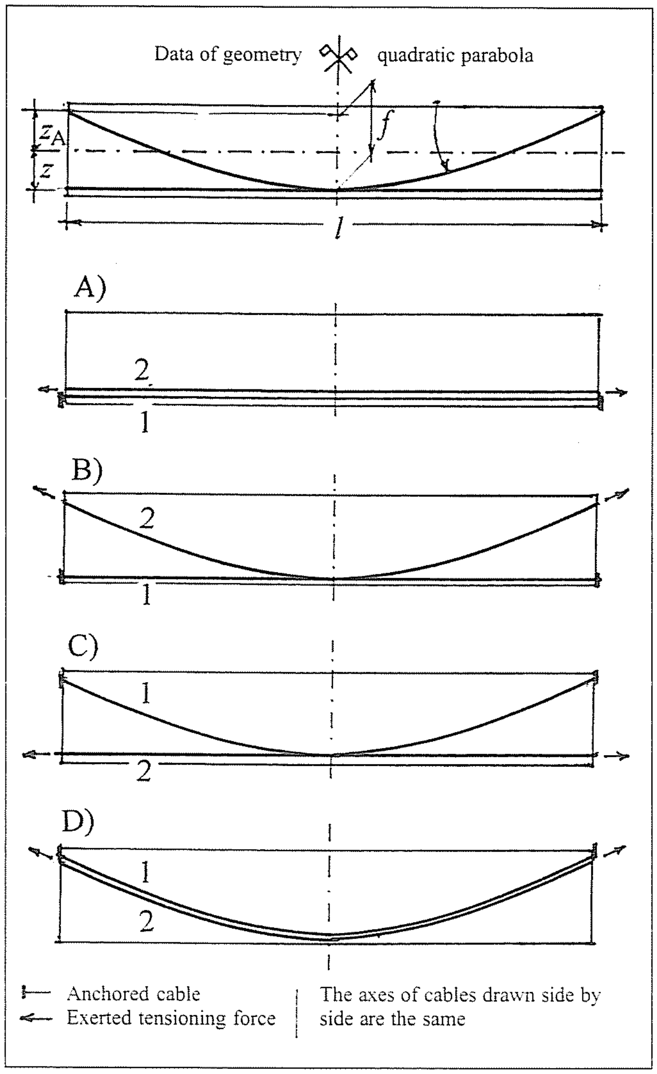

Fig. 4: Examples for simple post-tensioned girders

eccentric straight and quadratic parabola shape cables. The calculation is carried out based on the mentioned assumptions. The length of the parabolic cable is taken approximately as l, the error in practical cases is 2~4‰.

Omitting details of calculation, only the statically indeterminate values are given. These values are meaning for different cases the loss of prestressing force in cable *1*.

The results are the following:

$$\left.\begin{aligned}
&\text{Example A)}\quad P_m\frac{G}{G+L}, && G=\frac{z^2}{I_c}+\frac{1}{A_c},\\
&\text{Example B)}\quad P_m\frac{H}{G+L}, && H=\frac{z}{I_c}\left(\tfrac{2}{3}f-z_A\right)+\frac{1}{A_c},\\
&\text{Example C)}\quad P_m\frac{H}{K+L}, && K=\frac{1}{I_c}\left(\tfrac{8}{15}f^2-\tfrac{4}{3}fz_A+z_A^2\right)+\frac{1}{A_c},\\
&\text{Example D)}\quad P_m\frac{K}{K+L}, && L=\frac{1}{\alpha A_p}.
\end{aligned}\right\} \tag{9}$$

We wish to show by formulae (9) that the calculation which can be said exact is rather complicated even in such simple cases. The numerical values are not high, especially, if the cable analysed from point of view of the loss, is parabolic. Namely, in this case the cable runs alternatively along compression and tensile zones of concrete, if eccentric cables are tensioned. The cable is deformed according to the integral value

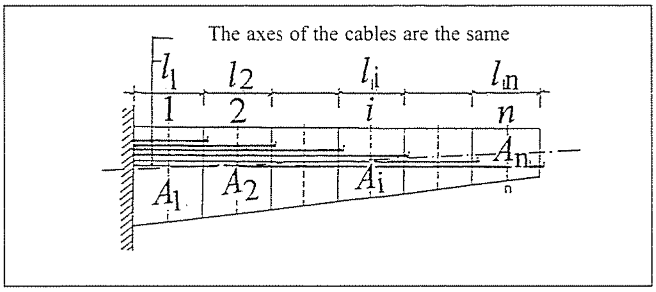

Fig. 5: Sketch of a free cantilevered post-tensioned concrete structure

of strains in the neighbouring to the tendon parts of the concrete member. In special case, the change (the loss) can be even equal to zero.

The usual method of theory of structures can by applied according to sense in case of more cables of various layouts. Of course, in this case a linear system of equations of higher order is to be solved. It gives a further complication in design if the structure is externally statically indeterminate. In this case, e. g. considering a continuous girder, the deformation due to moments at the supports, also influences the losses.

All these support the approximation suggested by codes *(e. g. MSZ ENV 1992-1-1)*. According to this, the loss of prestressing force due to elastic deformation of concrete is equal to the half value of the force calculated from the change of the cable length. This change is gained from the strains corresponding to the stresses in the concrete along the line of the substituting cable.

Let us consider a more sophisticated case. An intermediate phase of the assembly of a segmental free cantilevered prestressed post-tensioned concrete bridge is shown in *Fig. 5*. We meet in this phase a cantilever, i. e. a statically determinate system. The depth of the girder is varying. The layout of tendons is straight and parallel to the upper edge of the bridge. The cables are placed in one row, this means that their eccentricity is the same, but this can be displayed in *Fig. 5* only by help of an explaining text.

The structure is statically indeterminate to that degree – in all phases of assembly and post-tensioning – as the number is of already anchored cables (or uniform cable groups). Applying the force method, as redundant quantities are considered the change of forces in the already anchored cables produced by post-tensioning forces exerted in sequence of further cables. These changes in forces are equal to the losses due to elastic deformation also in this case.

The main diagonal elements of the coefficient matrix of the linear system of equation written for the case of assembly of a segment are the following:

$$a_{ii} = \sum_{k=1}^{i} \left\{ \frac{z_k}{E_c I_k} + \frac{l_k}{E_c A_k} \right\} + \frac{\sum_{k=l}^{i} l_k}{E_s A_{pi}} \tag{10}$$

The off main diagonal elements are

$$a_{ij} = \begin{cases} c_i, \text{ if } i < j \\ c_j, \text{ if } i > j \end{cases}$$

where

$$c_i = \sum_{k=1}^{i} a_k; \qquad a_k = \frac{z_k^2 l_k}{E_c J_k}. \tag{11}$$

The inverse matrix of the coefficient matrix built up by unit coefficients (10) and (11) is to be produced, and this matrix has to be multiplied by the load vector to receive the change of forces in the already anchored cables. These changes give the losses due to elastic deformation even in this case. The primary system was formed by cutting the already anchored cables. The elements of the load vector are this way the relative displacements at the places of cutting the cables caused by the post-tensioning forces acting in the subsequently tensioned cables. The analysis carried out on an analytical way *(Tassi, Rózsa, 1992)* shows that in a relatively simplified case the calculation is rather difficult. Let us think of it, that the analysis has to be carried out for each phase of assembly of precast segments. All the effects are to reckon with, which were disregarded in this study for sake of simplicity. N. B., although an analytical method can be applied for this simplified example, a recursion cannot be avoided even in this case. The recollection of the analysis of free cantilevered, prestressed post-tensioned concrete bridge served the aim to let to feel how labyrinthine and labour consumptive is to follow the effect of elastic deformation of concrete. This is true even for the presented structure which has a regular arrangement and is mostly simplified.

The sketchy presented examples show that it is possible to carry out an "exact" calculation to determine the losses due t o elastic deformation of concrete in prestressed post-tensioned structures. The analysis involves extremely much calculation work in case of externally indeterminate structure with varying cross section and miscellaneous cable layouts. This work is especially enormous if the analysis is to be carried out for many phases of the construction work. Naturally, the possibility of performing "exact" calculations is given by adequate systems of informatics, practically without limitation. The clearing up of principles and notions gives an aid to build up sophisticated calculations. By all means, there is a need for approximate methods.

6. CONCLUSIONS

The composed questions and formulised suggestions discussed here arise repeatedly the need of further clarifying of notions and terminology relating to prestressed concrete structures.

Hopefully, this task will be fulfilled by the Hungarian national instruction to Eurocode.

The clear description and interpretation of phenomena in connection with the elastic deformation of concrete is important mainly from point of view of the pure engineering logics and the knowledge of mode of working of structures. There is a need of calculation of decrease of prestressing force in prestressed pre-tensioned members if the concrete stresses are not calculated by means of ideal (reduced) cross section characteristics. This value is also needed if we wish to control by measurements the behaviour of structural elements. Anyway, the decreasing of the prestress at tension release cannot be considered as loss. We can speak about losses due to elastic deformation of concrete in case of post-tensioned structures. The need of a possibly exact calculation occurs in similar situations as in case of prestressed pre-tensioned beams. At least an approximate calculation must not be omitted.

The "exact" determination of losses due to elastic deformation of concrete in prestressed post-tensioned members is rather troublesome even in relatively simple cases. In case of more complicated structures, there is a need of a mathematical procedure and application of methods of informatics. Nowadays, there are hardly such phenomena which cannot be followed by numerical methods. Nevertheless, there is a sense of exact description of phenomena and calculations by good approximate methods.

7. NOTATIONS

P	the conceptual prestressing force (belonging to the deformation-free state of concrete)
P_c	force acting on concrete of prestressed pre-tensioned member
P_p	the force acting in prestressing steel in pre-tensioned member
P_m	the force exerted by the tensioning device
P_1	prestressing force acting in a tendon (group of tendons)
ΔP_1	decrease of force in an anchored cable at tensioning of a cable
A_c	cross section area of concrete
A_p	cross section area of prestressing steel
A_i	the ideal (reduced) cross section
I_c	moment of inertia of cross section
i_c	radius of inertia
E_c	Young's modulus of concrete
E_s	Young's modulus of prestressing steel
α	the modular ratio
σ_c	concrete stress
δP	the difference between the conceptual and exerted tensioning force
z	eccentricity of straight cable
z_A	eccentricity of parabolic cable at support
f	arrow height of parabolic cable
i	serial number of cable in sequence of tensioning
m	number of cables
l	span of the girder
l_i	length of a precast segment
a_{ii}, a_{ij}	unit coefficient
$\Delta\sigma_{el,i}$	loss due to elastic deformation of concrete in cable of serial number i

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TESTING OF PRESTRESSED CONCRETE COLUMNS – RESULTS AND CONCLUSIONS –


István Völgyi – Prof. György Farkas

Some experiments were carried out at the Structural Laboratory of the Budapest University of Technology and Economics to examine the behaviour of slender prestressed concrete columns under gravity load and transversal dynamic loads. The experiments included static and dynamic measurements too. The research aimed for showing how the changing of longitudinal (axial) prestressing affects the dynamical parameters of high thin columns. These parameters are: eigenfrequency, damping factor, ductility and bending moment capacity. A numerical method is shown which is partly able to determine the effect of prestressing.

Keywords: prestressing, dynamic behaviour, column, earthquake

1. INTRODUCTION

The mass placed on the column top and the transversal forces have a disadvantageous effect on the stresses working on high columns. This is the kind of problem that is encountered for example with bridges over deep valleys and water towers. Because of the grand height, the bending moment grows significantly while the axial force working on the structure changes only slightly compared to short columns with the same load. This fact results in the increase of the cross section dimensions while the axial load capacity is far greater than the actual load. By applying longitudinal prestress, bending moment capacity can be increased without the growth of the weight and the material cost. What is also known is that with the increment of the axial force, the ductility of reinforced concrete cross section decreases. These results are shown by Dulácska (1998), Almási (1974) and Kurama (1999). The main goal of this study is to provide numerical and experimental results for the variation of dynamic characteristics.

2. THE SPECIMENS

The specimens were 80 cm high reinforced concrete columns with a cross-section of 6×6 cm constrained at their bottom with a concrete block. Symmetric longitudinal reinforcement was applied at each corner of the cross section. Transversal reinforcement was used as well. An unbonded prestressing wire was placed into the central point of the cross section. There were used specimens without prestress and others using tendons prestressed to 450 or 900 N/mm². The longitudinal reinforcement was anchored in the bottom concrete block.

The specimens were made of two different mixtures. One with 30 N/mm² mean value of compressive strength and the other with 55 N/mm². Nine specimens were of each type of mixture (1-9 and 10-18). The wires used for the specimens with the lower strength had the 4.7 mm diameter and the ones with the higher strength 7 mm.

3 LOADING DEVICES

The specimens were tested on a compound vibratory table equipped with a pulsator. After the fixing, the twelve pieces of fixed steel plate were placed on the top of the column. This mass was able to model the real structure.

The specimens were loaded horizontally using three different methods. The first one was plunking using variable mass force. This mass generated a horizontal force on the top of the specimen resulting horizontal displacement. The force was suddenly ceased. As a result, the column started vibrating around the neutral, vertical position. The second loading method (forced vibration) was carried out by moving the computer-controlled vibratory table under the specimen using different frequencies and amplitudes.

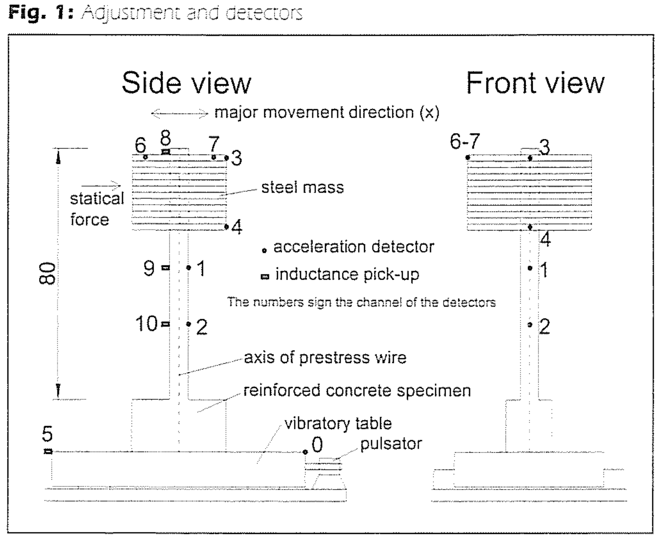
The third method was displacement controlled static loading. A quasi-static load was generated by moving the table very slowly under the specimen. The horizontal displacement of the top point of the column was blocked by a steel cantilever equipped with a pressure meter cell (*Fig. 1*). Explication see in the paper of Völgyi (2003).

4 MEASURING SYSTEM

Inductance pick-ups and different sensitivity acceleration-detectors were placed on certain points of the specimen (*Fig. 1*).

This way accelerations and displacements of these points in both major and minor direction could be studied. There was no torsion vibration detected.

Fig. 1: Adjustment and detectors



5. THE EXPERIMENTS

The experimental programme can be divided into three parts. In the first phase the so called crack-free columns were tested by very small vibrations. The specimens were deflected by an increasing horizontal load. A quasi-plunking was caused by suddenly ceasing the force. After this, a sweep-test with small amplitude was conducted.

In the second phase, the specimens were stimulated with high frequency until cracking. The frequency used was much higher than the first eigenfrequency of the column. In this case, the displacement was caused by the mass force. Because of the big difference between the eigenfrequency and the frequency applied the displacement of the top point was equal to the shift of the table. First the displacement of the specimens was 2 mm. Then the change of the dynamic parameters were examined in several steps while the forces and displacements increased. Several plunkings and sweep tests were carried out. After a new high-frequency stimulation with bigger amplitude (4 mm), the above programme was repeated. The second phase included also the white noise stimulation thusly simulating an earthquake.

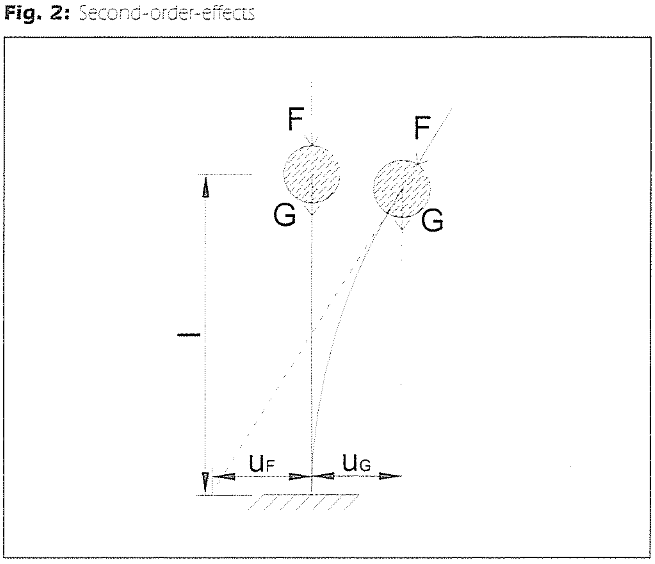
In the third phase, a force-displacement diagram was made under a quasi-static horizontal unilateral cyclic load. From the diagram ductility and variation of stiffness could be evaluated.

6 FORCE ACTIONS

The first eigenform is dominant because of the simple static model. The prestressing tendon placed into the specimen was unbonded so during the experiments the stress in it was almost constant. The tendon followed the curve of the concrete specimen because the duct was narrow enough. This way the line of action of the prestressing force was always normal to the top cross-section. Consequently, the prestressing force got an eccentricity in the constraint section.

This resulted an opposite effect to the second order P-Δ effect, which latter was caused by the displacement of the mass. It was a quasi-return-force (*Fig. 2*). Thus, as a result of the prestressing, not only the growth of the moment capacity but also the growth of the virtual stiffness of the column could be expected. This increased stiffness can be shown even at high intensity horizontal loads since the return force not uniquely in the case of an uncracked structure existed (Kurama, 1999).

Fig. 2: Second-order-effects



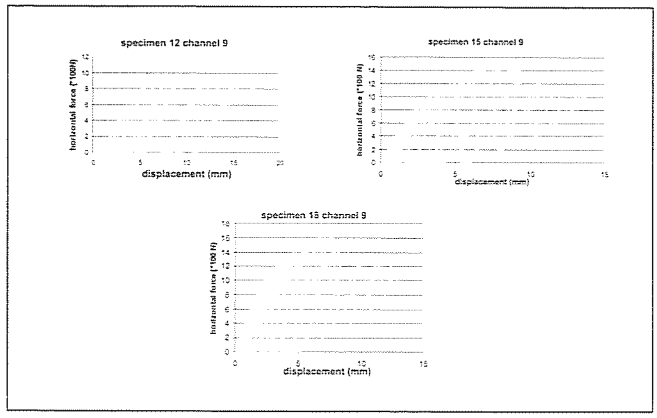


Fig. 3: Force-displacement diagrams of the specimens on the three prestress levels

7. EFFECTS AND RESULTS OF THE EXPERIMENTS

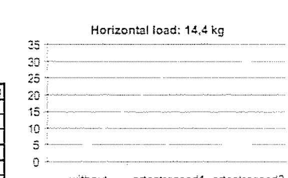
Fig 3. shows the data of one of the inductance pick ups (N°. 9. channel 9. see *Fig. 1*) and of the pressure meter cell.

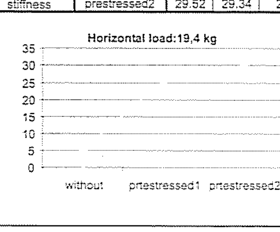
Plastic deformation of prestressed concrete structure is small at the top point of the force-displacement curve. The prestressed specimen's behaviour is similar to that of a bilinear elastic system, while the plastic deformation of the column without prestressing force increases monotonously after the elastic behaviour. After passing the top point of the curve the specimens were considered as ruptured. In reality after passing this point prestressed consoles have more deformation back. By significant decreasing of the maximum displacement ductility factor hardly changed, although the dissipated energy radically decreased. Radical increment of detected eigenfrequency was caused by prestressing force. On the other hand damping factor was heavily decreased by prestressing. The significant growth of eigenfrequency is not only caused by the crack free state of the concrete material. The growth does not depend significantly on force level. Increment can be detected by stages, when cross sections are surely fully prestressed by both pression level. At the first pression step growth is much greater than at the second step. Diagrams are shown in *Fig. 4*.

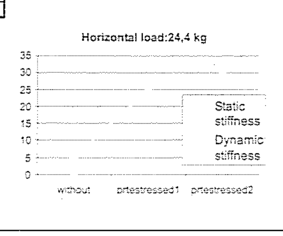
Consequently cracks must have occurred on the specimen. As bending moment at the constraint cross section from the first load level is not large enough for making cracks there, cracks must have been caused by striping and transport. Capture cross section stayed under decompression at both prestress level. Consequently detected eigenfrequencies do not depend on initial condition of the concrete.

Fig. 4: Variation of statical and dynamical stiffness

	Load	14.4 kg	19.4 kg	24.4 kg
Index-number of static stiffness	without	1.26	0.75	0.49
	prestressed1	4.62	2.64	1.63
	prestressed2	5.36	3.5	3.04
Index-number of dynamic stiffness	without	5.98	5.65	5.4
	prestressed1	26.55	25.47	25
	prestressed2	29.52	29.34	29







The decrement of the Young's modulus while increment of the horizontal load at the same prestress level can be observed as well. That means that specimens were checked forced by higher and higher extreme fiber stress.

As a consequence increment of the first eigenfrequency is partly caused by the second order effect of the prestressing force. The values on both figures were determined according to the places and values of the local extremums of the vibration's time-acceleration diagram.

8. CALCULATION METHOD OF THE EFFECT OF PRESTRESSING

Engineers used not to calculate generally the variation of the stiffness of prestressed concrete elements. It can cause mistakes by statically indeterminate structures under static loads, because structural elements of larger stiffness have to support more load. The consequence is much worse at structures under dynamic loads. Increment of stiffness and eigenfrequency might cause the change of the load's magnitude order.

The effect of prestressing force can be determined using the dynamic stiffness matrix. It needs to be changed by the effect of eccentric force. Dynamic stiffness matrix can be determined at simple structures in accordance to the Eq. (1) (Györgyi, 2000).

$$K_{din} = K_{st} - \omega^2 M. \tag{1}$$

Effects of static longitudinal load, inertia and shear strain can be determined using the corrected formula. This equation is shown in the Eq. (2).

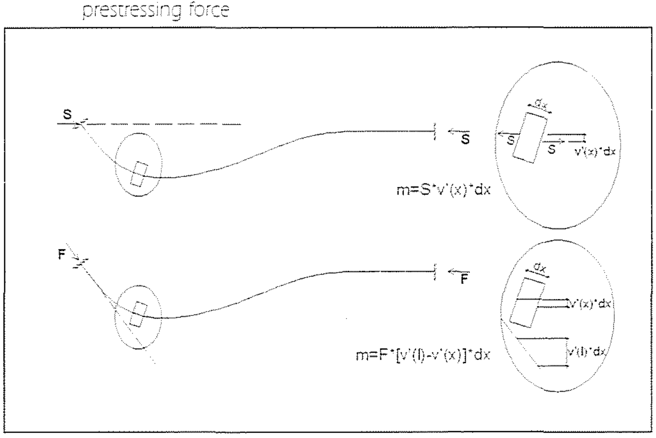
$$K_{din} = (K_{st} + K_G) - \omega^2 (M + M_\phi). \tag{2}$$

Neither of these formula is able to determine the effects of prestressing wires. That is why we developed a method which is able to include the effect of prestressing too. A new adaptive element is needed to be determined for the dynamic stiffness matrix according to Eq.

$$K_{din} = (K_{st} + K_G + K_F) - \omega^2 (M + M_\phi). \tag{3}$$

This method calculates the work of the dynamic loads on the static deformations. Consequently, bending moments on the bar caused by the prestressing force has to be determined. Calculation of this moment is shown in the *Fig. 5*. N'_{mod} is necessary for defining the additive stiffness matrix. The structure of this matrix is similar to the structure of matrix N_{mod} used for defining the effect of static longitudinal force (*Table 1*). The real difference is smaller than it seems to be, because by this method displacement caused by dynamic loads is replaced by the static one. Consequently end point values of the displacement are constant, which means that structures of both matrixes are the same, the differences are only the values of them.

Fig. 5: Determination of bending moments caused by normal force and prestressing force



$$\begin{pmatrix} u_a(x) & 0 & 0 & u_b(x) & 0 & 0 \\ 0 & v_{ya}(x) & v_{\varphi a}(x) & 0 & v_{yb}(x) & v_{\varphi b}(x) \end{pmatrix} = N$$

$$\begin{pmatrix} 0 & v'_{ya}(x) & v'_{\varphi a}(x) & 0 & v'_{yb}(x) & v'_{\varphi b}(x) \end{pmatrix} = N'$$

$$\begin{pmatrix} 0 & v'_{ya}(l)-v'_{ya}(x) & v'_{\varphi a}(l)-v'_{\varphi a}(x) & 0 & v'_{yb}(l)-v'_{yb}(x) & v'_{\varphi b}(l)-v'_{\varphi b}(x) \end{pmatrix} = N'_{mod}$$

Table 1: Matrixes necessary for defining elements of dynamic stiffness matrix

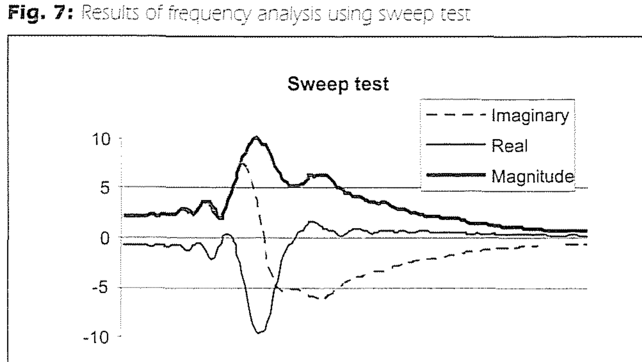
$$K_F = \begin{pmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 55.2 & 4.6\times 10^3 & 0 & 0 & 0 \\ 0 & 4.6\times 10^3 & 6.133\times 10^6 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 55.2 & -4.6\times 10^3 \\ 0 & 0 & 0 & 0 & -4.6\times 10^3 & 6.133\times 10^6 \end{pmatrix}$$

Fig. 6: Additive matrix of effect of prestressing

Calibration was carried out for determining K_F modifying matrix (*Fig. 6*). The elements of the stiffness matrix are growing caused by the new additive matrix. This increment of $K_{din6,6}$ is about 5%, which is significantly lower than the real growth of the stiffness. Consequently, modifying of the dynamic stiffness matrix using this only method is not enough for modelling the real change of stiffness.

Eigenfrequency is grown by increasing the prestressing force. The higher the eigenfrequency is the faster the stress changes. What is also known, Young's modulus of concrete material under continuous load is much lower than under short term load. It had been determined before that Young's modulus of concrete under quick dynamic loads might be much higher than that under short term static loads. Some experimental results have already been published dealing with the change of concrete's Young's modulus depending on the load intensity velocity change (Szatmári, 2002). The stiffness of reinforced concrete or composite structures under quick or dynamic loads is much higher than the stiffness detected before under static loads.

Fig. 7: Results of frequency analysis using sweep test



The significant growth of stiffness can be caused by this effect above. The cogency of the inclusion of this effect needs to be proved by some other experiments.

Asymmetric, floating curve was detected instead of flat bell curve at frequency analysis which is usual by nonlinear systems. It can be caused partly by this effect too (*Fig 7*).

9. CONSEQUENCES

Results of the experiments show that stiffness variation of no tensile stress prestressed concrete columns is significant. This growth can be higher than 20%. It can cause the remarkable variation of the load on the structure. That is why determination of the value of it at dynamic analysis is important.

10. NOTATIONS

K_{din}	Dynamic stiffness matrix of the structure
K_{st}	Static stiffness matrix
M	Mass matrix
ω	Circular frequency
$K_{st'}$	Modified static stiffness matrix
K_G	Geometric stiffness matrix
M_ϕ	Modifying matrix of static normal force
K_F	Modifying matrix of prestressing force

11. ACKNOWLEDGEMENTS

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EAST AND WEST – WELDED REINFORCEMENT

Dr. Gyula Fogarasi

International investment and construction companies started construction works in Eastern Europe. Those companies are teaming up from investors, designers, contractors, subcontractors, precastors of different nationalities. Their product based on Eurocode has to be approved by authorities of another natonality. The foreign experts meet the unique standard designs, standard products, standard precasting and erection technologies born in that splendid isolation. Very frequently not only the difference of several languages, but the difference in the technical background makes understanding difficult.

This short article summarizes a few of those differences: the basic types of precast concrete structural products in Eastern Europe, their unique welded reinforcement, the interdependence between testing and design requirement for hot rolled welded reinforcement. A few examples for welded reinforcement of precast concrete members illustrate the difference in members and their reinforcement alike.

Keywords: welded reinforcement, long line, fixed bed, flow line

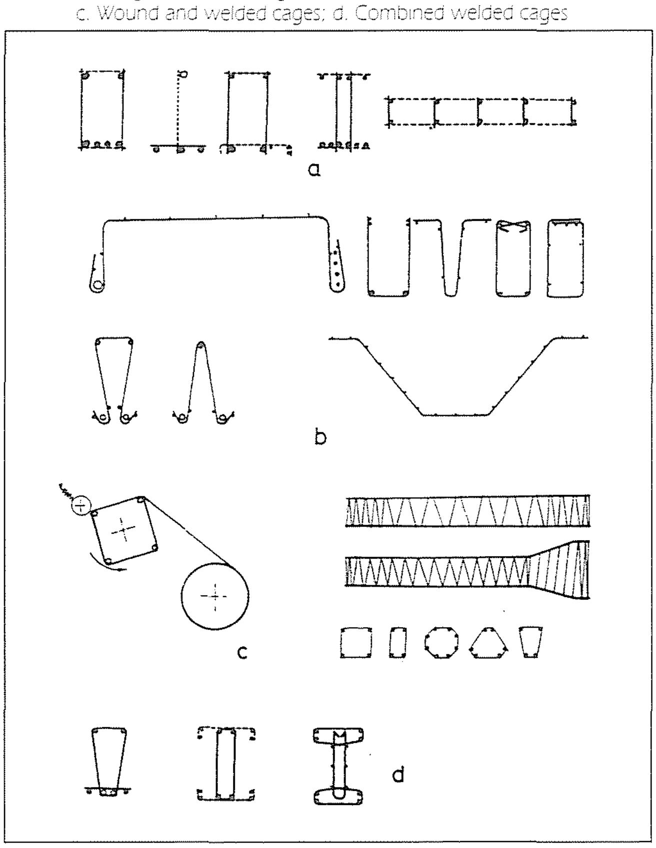
1. INTRODUCTION

"We all emerged from the cloak" of Professor Leonhardt, admired his work on the 'Art of Reinforcement' and studied unceasingly his book, which revolutionized that industry (Leonhardt, Mönnig, 1974). His books sent me by himself were always on my desk, when I wrote a few books and articles on the other side of this art, the welded reinforcement of the East. I was the proudest when he said a few good words about my work based merely on the illustration.

Just as the holy orthodox icons this branch of industry had his rigorous canons: the compulsory use of standard designs, standard cages produced on standard machines. However, in this Babel of standardization it was extremely difficult to me to find the keys of the interdependence between testing and design requirement for hot rolled welded reinforcement. Thanks to my good luck and my stamming Russian I met personally I. Y. Yevgeniev who was the gray eminence of the research, standardization and design of the "other way" of welded reinforcement. The book on welded reinforcement (Fogarasi, 1988) could not be born without him.

To my greatest surprize the disappearance of barbed wires and crumbling of walls reinstated the interest for the unique products of the East in the field of welded reinforcement and precast products alike.

Fig. 1: Production of welded reinforcement
a. Open cages assembled from welded ladders or fabrics by welding; b. Closed cages bent of welded wire fabrics and ladders
c. Wound and welded cages; d. Combined welded cages



2. MANUFACTURING METHODS OF WELDED REINFORCEMENT

Welded reinforcement was and is the best answer to the high manpower requirement of the reinforcement manufacture. Resistance welding is used both for the production of welded wire meshes of small diameter cold drawn wires and of cages of large diameter reinforcing bars (Fogarasi, 1986).

In many Eastern European countries, especially in Russia, Ukraine and Bulgaria welded ladders and spatial cages made of hot rolled rebars are today still the most popular production modes *(Fig. 1.a)*. Reinforcing cages where stirrups are not surrounding the main bars but connected to them by welding may be called *open reinforcement.*

In the western half of Europe welded wire meshes made of cold drawn wires are used both as plane fabrics and as cages bent of welded fabrics (*Fig. 1.b*). These cages are generally bent around the main bars or the prestressing tendons, they behave like traditional ties and stirrups and may be called *closed reinforcement.*

A highly productive way of reinforcement production is winding and welding cages (*Fig. 1.c*), where small diameter stirrup is wound, bent and connected by resistance welding to the main bars. This method used both for cold drawn wires and hot rolled bars produces always closed cages. Finally combinations of the above mentioned three types are possible (*Fig. 1.d*).

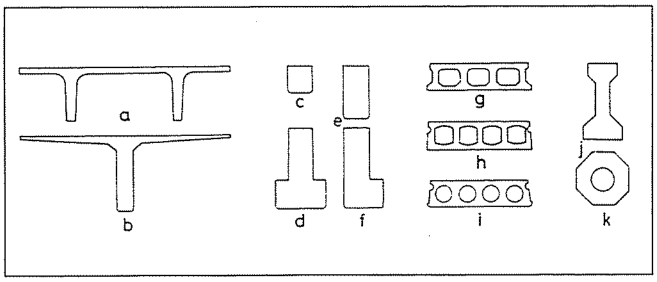


Fig. 2: Typical Western type precast concrete structures
a – double tee, b – single tee roof and floor slabs, c – through
f – beams and columns, g– through, i – hollow core slabs,
j – bridge girder, k – solid or voided pile

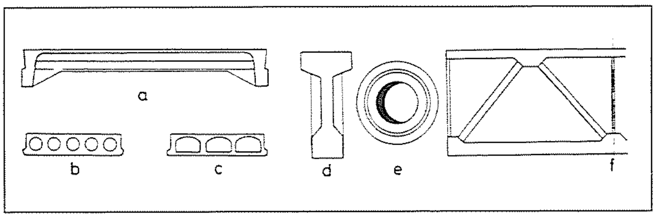


Fig. 3: Traditional Eastern type precast concrete structures
a – 3x6 and 3x12 m standard roof units, b – hollow core slabs,
d – bridge girder, e - pressure pipe, f - trusses

The principles of manufacture, testing, design and detailing of the open and the closed reinforcing cages are completely different. In the past that was a problem only for those few companies which imported a building system with technological equipment, in our case with resistance welding machines. Introduction of large panel housing systems with housing factories were typical examples.

Recently this question occurs in another way. Polgár (2004) listed in his article the problems his Hungarian general contractor company faces when works e.g. with German designers, Rumanian subcontractors, Ukrainian precastors in Russia. And working on more than fifty building sites of seven different countries this is not the exception, but the rule. One of those many problems is the difference in the precast structures of East and West.

The Western palette consists of prestressed double and single tee units, beams, columns, girders, piles and hollow core slabs (*Fig. 2*). All those elements of continuous sections are manufactured in long line fixed bed precasting technologies (PCI, 2000 and Fogarasi, 1986).

The traditional precast concrete products in Russia, Ukraine and other Eastern European countries consist of standard 3×12 m and 3×6 m trough waffle type roof units, bridge girders and industrial beams full of ribs and stiffeners, trusses of changing sections for 18, 24, 30 and 36 m spans, all pointed towards the minimum use of material (*Fig. 3*). Most of those elements are manufactured in flow line conveyor technologies, many prestressed by electrothermic methods (Fogarasi, 1986).

The other huge difference is in standards, design and practice of welded reinforcement in the participating countries. Our book (Fogarasi, Adamik 1976) presents a detailed survey of the base materials, of the reinforcement systems, of the manufacturing methods and equipment, of testing and qualification, and of optimum values of resistance welding parameters. The book also discusses the principles of structural design and detailing of welded reinforcement, as well as a large number of examples of its use in cast-in-place and in precast concrete structures.

The testing, design and detailing principles for the cold drawn wire fabrics are well known.

Let's have a short survey of the curiosities of the testing, design and detailing of cages welded of hot rolled rebar.

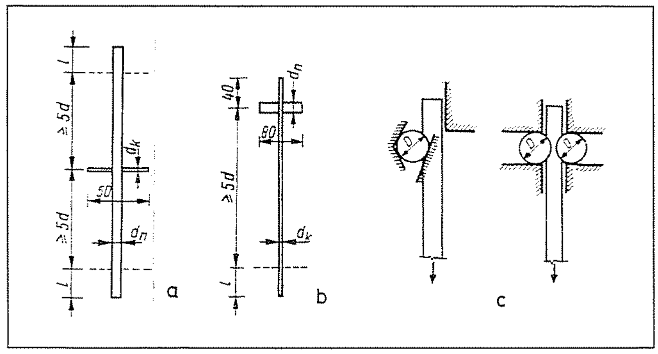


Fig. 4: Welded cross specimens according to GOST 10922
a. – welded cross specimen for testing the welded base material
l length of gripping; b. – welded cross specimen for testing the shear capacity of a welded connection of two crossing bars;
c. – arrangement of the shear capacity test for single and double cross bars

2. TESTING REQUIREMENTS FOR WELDED HOT ROLLED REINFORCEMENT

The Russian code for testing welded hot rolled reinforcing bars: GOST 10922 specifies two tests:

2.1 Testing the welded base material

Requires welded cross specimens as shown in Fig. 4. a. The welded base material is satisfactory if the tensile test stress calculated by dividing the tensile force by the nominal cross sectional area of the bar is equal or greater than the nominal strength of the steel bar. In a maximum of 10 % of the test specimens that stress can be maximum 20 % lower then the nominal strength of the bar.

2.2 Testing the shear capacity of a welded connection of two crossing bars

Requires welded cross specimens as shown in *Fig. 4. b.* Three specimens should be tested for shear from every batching. The shear capacity of the weld is satisfactory if the value of the shear force V is equal or higher than

$$V > C\,A_s\,R_y$$

where

A_s is the nominal cross sectional area of the smaller diameter bar tensioned in the test

R_y is the nominal yield strength of the smaller diameter bar tensioned in the test

C is the weld shear coefficient, which is for different grade of the smaller diameter bar:

for grade A-I bars C = 1.50
for grade A-III bars C = 1.25

According to Russian terms grade A-I bars are smooth rebar of 240 MPa minimum yield strength, equivalent to S 240 bars, while grade A-III bars are deformed (ribbed) bars of 400 MPa minimum yield strength, equivalent of S 400 rebars.

In a maximum of 10 % of the test specimens that shear capacity of the weld can be maximum 20 % lower then the required minimum shear force specified above.

3. DESIGN REQUIREMENTS FOR WELDED HOT ROLLED REINFORCEMENT

SNiP II-V.1, the Russian design code for concrete and reinforced concrete structures contains several requirements for welded hot rolled bars and cold drawn wires.

3.1 Design strength of welded bars

And wires is the same as those of tied reinforcing elements. E.g. the design strength of the welded grade A-III bars (corresponding to grade S 400) of 400 MPa yield strength is the same 340 MPa as of the tied ones. This is based on the high testing requirement for the welded base material described in section 2.1.

3.2 Design shear strength of the welded connections

The Russian design code does not apply any reduction factor for the shear capacity of a welded connection or by other words for the shear capacity of a welded crossbar of a ladder connected to the main bar by welding only. However, there is a general reduction factor for any shear reinforcement whether it is welded or tied, open or closed.

The reduction factor for hot rolled shear stirrups is 0.8, for cold drawn wire stirrups is 0.7, whether it is open or closed, tied or welded to one or more main bar.

This requirement mirrors the wide-spread use of the welded reinforcement in Russia and Ukraine.

4. WELDED REINFORCEMENT FOR PRECAST CONCRETE STRUCTURES

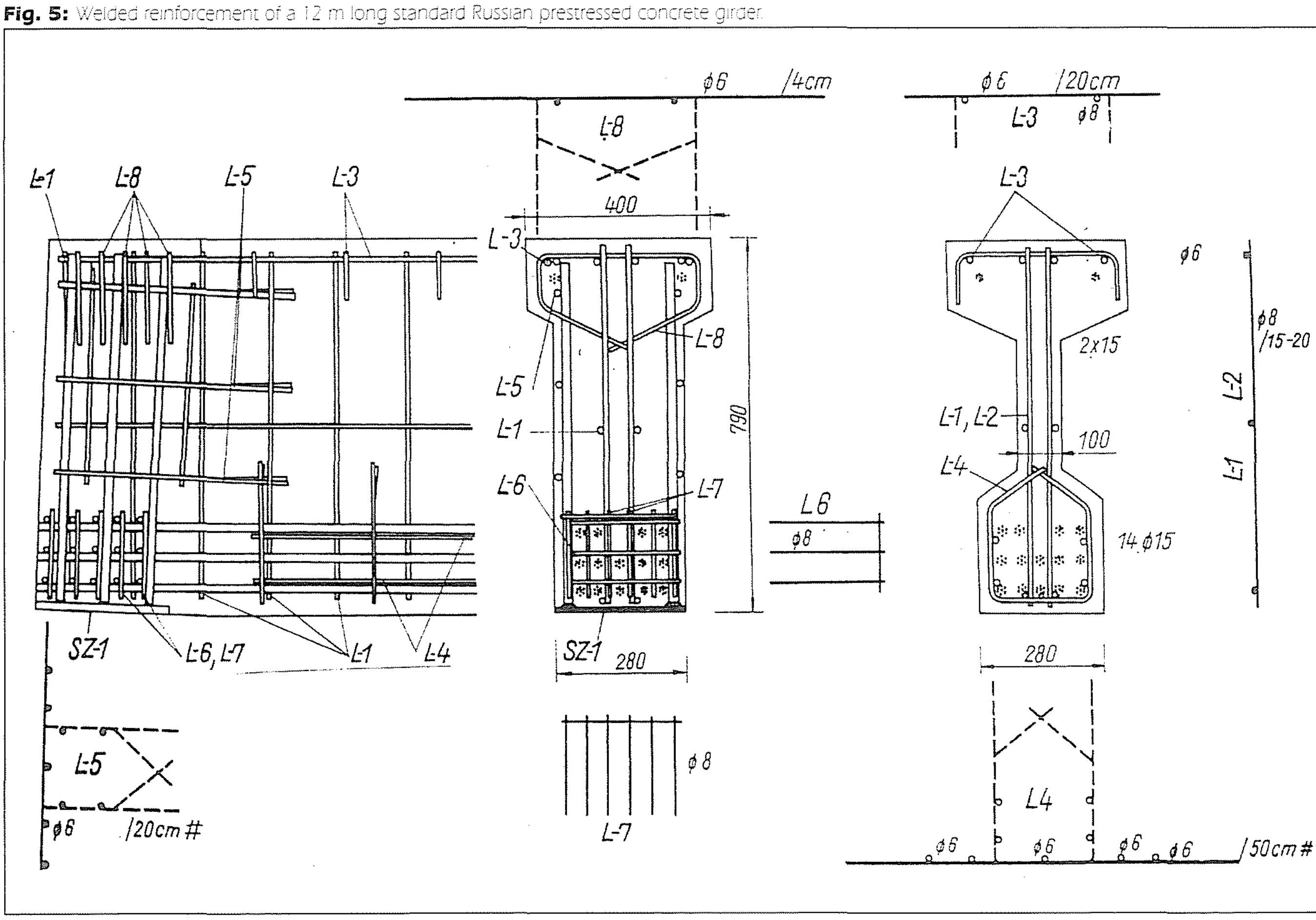
The design and detailing of the welded reinforcement of precast concrete structures should follow the technological sequence of the placing and fixing of the prestressed and non-prestressed reinforcement, the hardware, the hollow formers and the thermal insulation layers. For non-prestressed members best if the complete reinforcement containing other materials can be prefabricated and placed in one unit into the form. For prestressed members best if the complete non-prestressed reinforcement can be installed before or after the placement of the prestressing tendons. If that is not possible, the non-prestressed reinforcement should be cut into two complete units. The first unit (e.g. bent rib fabrics) should be placed before, the second (e.g. top flange fabric) after threading the tendons.

A few examples of welded reinforcement for Russian standard precast members illustrates the unique solutions designed, investigated, tested and manufactured by local experts (Fogarasi 1988).

Fig. 5. hows the welded reinforcement of a 12 m long standard Russian prestressed concrete girder. After placing the L-4 bent welded ladder stirrups threading and tensioning the 14 pieces of 15 mm diameter prestressing strands follows. Than L-1, L-2 web ladders, L-3 top flange ladder, L-5 through L-7 end reinforcing combs and L-8 bent top flange ladder finish the cage.

Fig. 6. illustrates the welded reinforcement of the 12x3 m standard prestressed industrial roof unit. The waffle unit consists of a 30mm thin top slab, two longitudinal main ribs, ten internal, one central and two end ribs. After placing the H-2 bent wire fabrics into the end zones of the main ribs groups of bottom and top prestressing wires are laid in. H-1 main fabric

Fig. 5: Welded reinforcement of a 12 m long standard Russian prestressed concrete girder



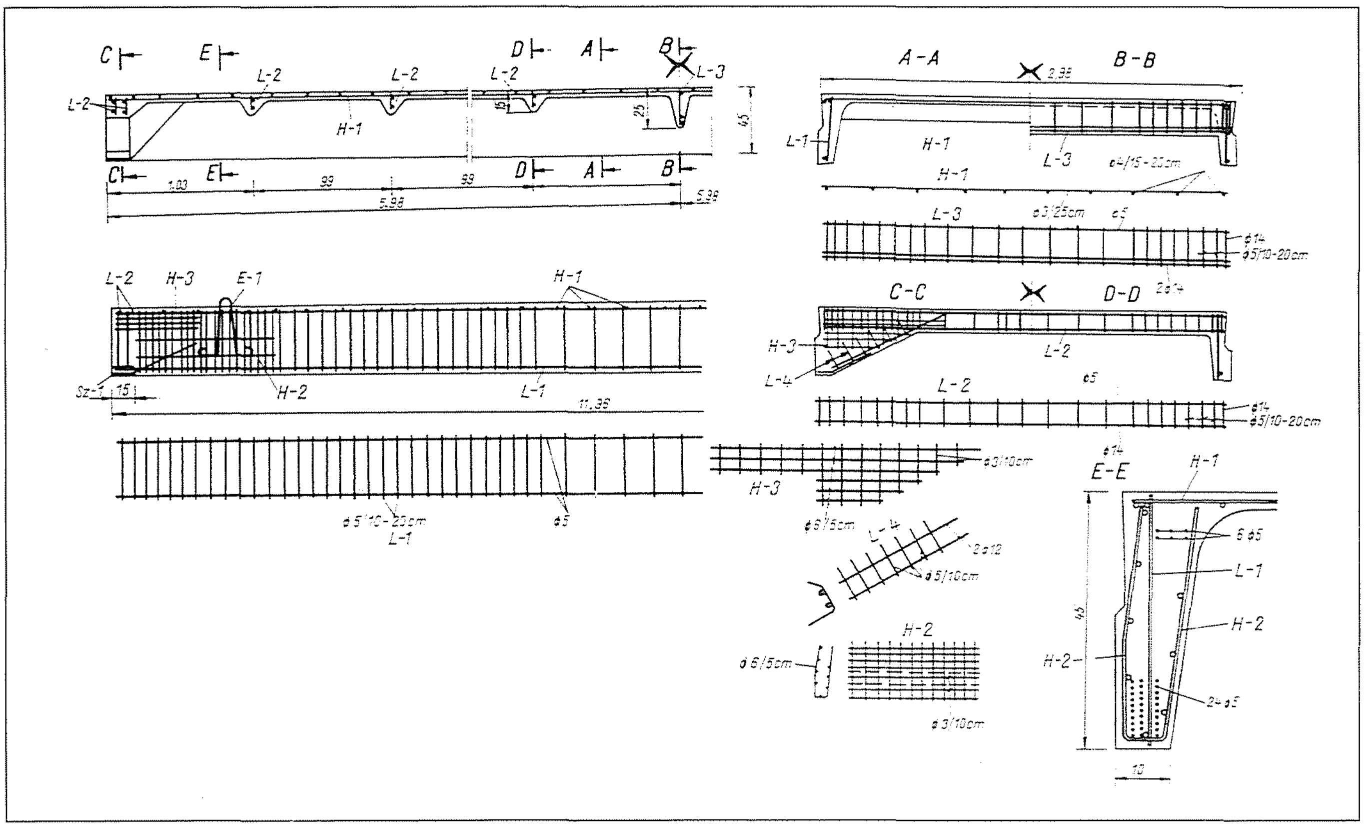


Fig. 6: Welded reinforcement of the 12x3 m standard prestressed industrial roof unit.

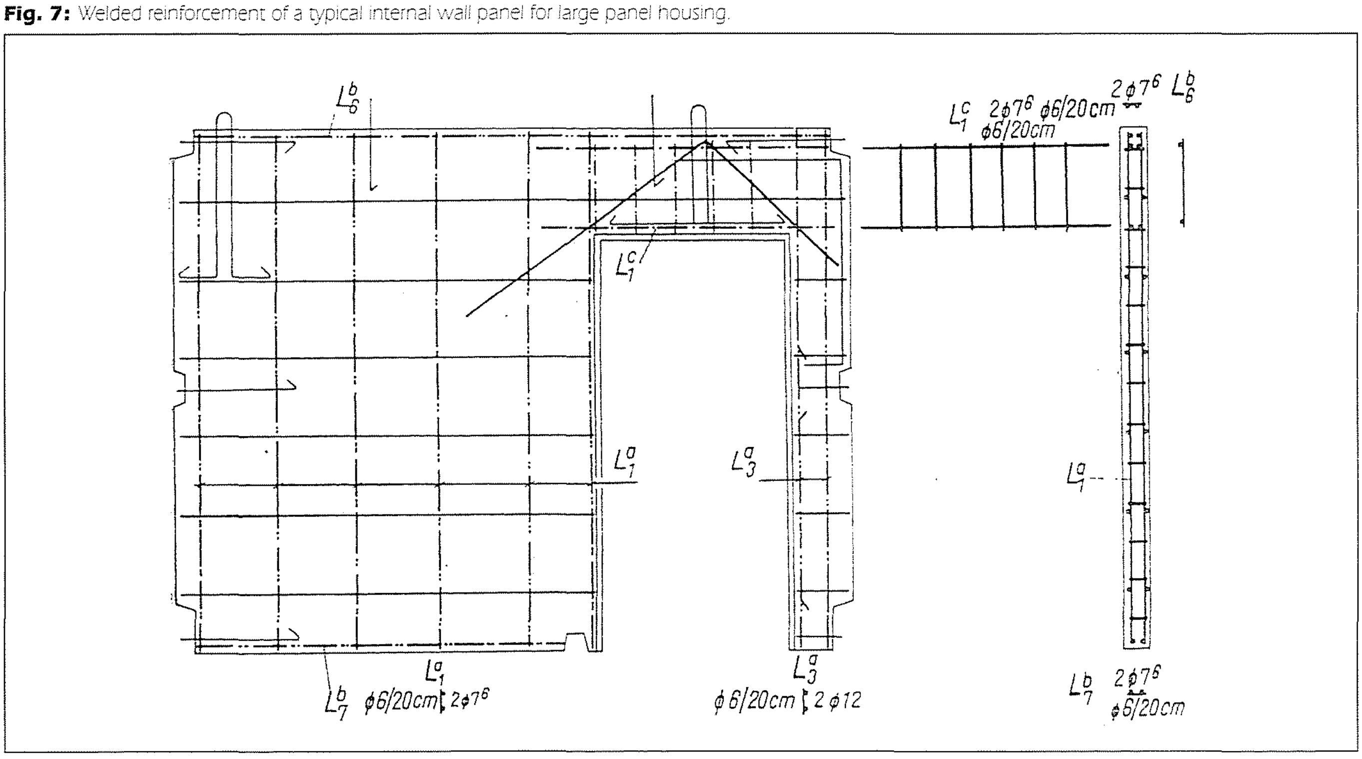
covers the rib reinforcement consisting of two L-1 longitudinal rib ladders, 14 L-2 lateral rib ladders and one L-3 central rib ladder. End reinforcement closes the sequence.

Fig. 7 is a typical internal wall panel for large panel housing. The light reinforcement consists of six La1 and two La3 vertical ladders, a top Lb6 and a bottom Lb7 horizontal ladder, two Lc1 horizontal lintel ladders and 2x6 pieces of horizontal single wires welded to the La1 ladders by suspended welding pliers. All bars are 6 or 7.6 mm diameter cold drawn wires.

Fig. 8 shows the reinforcement of a 3x3 m precast concrete grain silo cell used as building blocks for 36 m tall silos. The four sides are reinforced by 2x4 pieces of welded wire fabrics. The welded cage of the corners shown in detail "A" are reinforced with No 1 external, two No 2 internal bent bars connected by No 3, 4 and 5 transverse straight wires. Six layers of these welded reinforcement are connected by welding of ten pieces of No 6 vertical wires into a corner cage. The simple welded reinforcement completely follows the complicated shape and satisfies the positive and negative bending moment zones in the cell.

Those classical examples illustrate the refined art of welded reinforcement, where East and West mutually can serve each other with their experiences to their complementary advantages.

Fig. 7: Welded reinforcement of a typical internal wall panel for large panel housing.



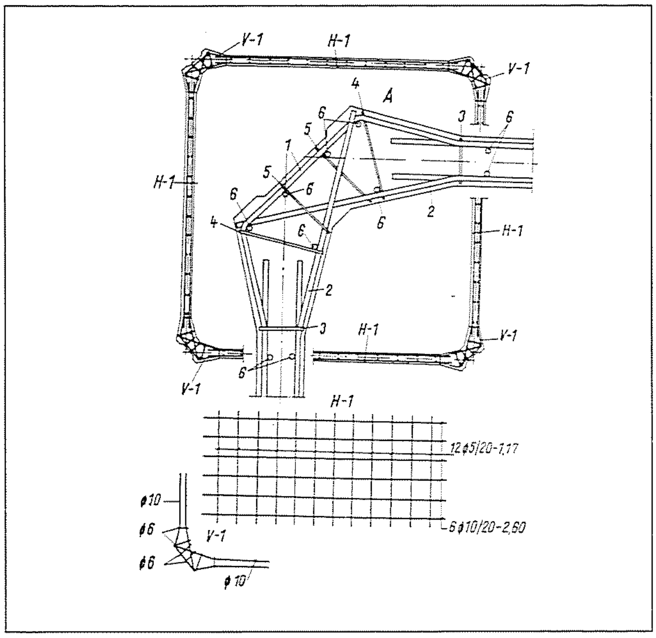


Fig. 8: Welded reinforcement of a 3x3 m precast concrete grain silo cell used for 36 m tall tower silos.

5. CONCLUSIONS

The activity of multinational investment and construction companies in the Eastern half of Europe aroused the interest toward the unique standard designs, standard products, standard precasting and erection technologies born in that splendid isolation. One can like it or chide it but can simply not neglect the existence of 4000 standardized precasting plant pouring hundred millions of tons of standard precast concrete units on millions of standard machines based on compulsory standard designs.

One slice of this gigantic technology is the manufacture of different types of welded reinforcement of different base materials on different resistance welding equipment. Any type of construction activity in this zone assumes at least the basic knowledge of that closed world even for a competitor.

This short article summarizes the basic types of precast concrete structural products in Eastern Europe, of their unique welded reinforcement, the keys of the interdependence between testing and design requirement for hot rolled welded reinforcement. A few examples for welded reinforcement of precast concrete members helps to understand that closed world.

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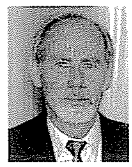
Dr. Gyula F. Fogarasi (1937) Professional Engineer of Nebraska and New Jersey, graduated in 1960 and received his doctorate in 1980 from the Technical University of Budapest. He started his professional experience at the KÉV-METRÓ construction company, than his design carrier with IPARTERV Institute for Industrial Design as principal designer of the first fully precast buildings at TVK, TMV, BVK, DOM and other large industrial investments. 24 years long he was head of product design, technical development and contracting at BVM Concrete and Reinforced Concrete Works, the ruling Hungarian prefabricating company. In 1989-91 he was Visiting Professor of the Nebraska University and part time consultant of Wilson Concrete Company in Omaha, NE. In 1991-2001 he was Senior Designer and Adviser of Parsons Transportation Group, New York, NY. He was principal designer of the second deck design of the world recorder Tagus River Bridge in Lisbon, Portugal, of the reconstruction of the long span Tacony-Palmyra and Burlington-Bristol bridges on the Delaware River. He made the conceptual design for the second Tacoma Narrows Bridge, a suspension alternative of the Qarkuinez Bridge in California and the preliminary design of the incremental launching of the cable-stayed bridge on the Charles River, Boston, MA.

He was extramural professor in the Pratt Institute and in the Polytechnic University, New York. He is teaching in the Ybl Miklós Technical School of the St Stephen University, Budapest.

He is author of dozen of technical books and notebooks in several languages and author of several patents in the field of precasting. He is construction expert of UNIDO.

In the period of 1986-1990 he was Vice President of FIP and is the Honorary President of the Hungarian *fib* Group. LIST OF FIGURES

STEEL FIBRE REINFORCED CONCRETE, FROM RESEARCH TO STANDARDS



Prof. Dr.-Ing. Horst Falkner – Dr.-Ing. Volker Henke

This paper deals with a new design concept for SFRC structural members such as piled industrial floors where steel fibre reinforcement carries parts of the total tensile forces. Systematic tests and analysis of material parameters and its characteristic values were carried out. A structural safety analysis of these members was performed. Based on the first order safety theory, a calculation and verification method is introduced, especially taking the combination of bar and steel fibre reinforcement into account. This paper shows that the reinforcement combination can increase the structural safety remarkably.

Keywords: Steel fibre, reinforcing bars, cracking, non-ballasted track, structural safety, flat slab, underwater concrete, HPFRC

1. INTRODUCTION

Recently there is an increasing number of projects with structural elements made of steel fibre reinforced concrete. Steel fibres are used instead of ordinary steel reinforcement or in addition to reinforcing bars. Both under service and ultimate loading conditions, the fibre reinforcement is subjected to resist tensile forces.

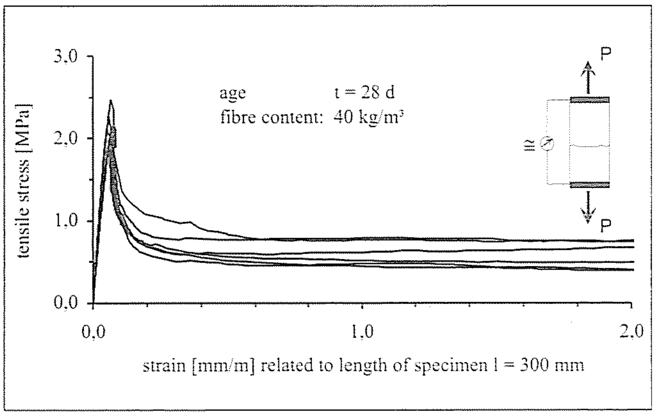
The addition of steel fibres to plain concrete changes its mechanical properties. Depending on the type and amount of fibres an increase in ductility and better cracking behaviour can be achieved. Especially by the use of steel wire fibres remarkable stresses can be transferred across cracks. The fibre itself can be seen as a kind of reinforcement. A verification concept for SFRC structural members can be derived from this principle.

A review of present design codes and recommendations such as JCI, ACI and DBV (German Concrete Association) showed, that the defined parameters for toughness and equivalent flexural stresses are not sufficiently applicable for structural components. Constitutive models for SFRC, based on ultimate crack widths were defined. A safety analysis based on ultimate limit state criteria was carried out.

Based on these investigations and experiments, applications of SFRC for structural elements were introduced. Under certain circumstances steel fibres can be seen as reinforcement carrying tensile forces in serviceability and ultimate limit states. Applications of SFRC for underwater concrete slabs, industrial floors on piles, ductile high performance concrete columns and foundations for housing structures have proven steel fibres as structural reinforcement.

Based on the recent research work concerning steel fibre concrete and the constructions, using steel fibre concrete, carried out so far, a design recommendation for this kind of material is compiled by the DAfStb (German reinforced concrete group). An outline of this recommendation is given at the end of this paper.

Fig. 1: Results from direct tensile tests



2. MATERIAL PROPERTIES AND CONSTITUTIVE MODELS

2.1 Direct tension and bending tests

In order to investigate a stress-crack width relationship strain controlled direct tension test were carried out on cylinders with 150 mm diameter and 300 mm length. After demoulding of the cylinders, they were cured in a water basin for 28 days. Steel plates were glued onto each end, and tensile forces were applied in the same direction than concreting. *Fig. 1* shows the test results of a concrete with 35 MPa compressive strength and a fibre content of 40 kg/m³ DRAMIX® RC 80/60 steel wire fibres with hooked ends.

It can be derived from the direct tensile tests that, after formation of the crack, there is a stress transfer across the crack by the steel fibres which is independent from the crack width. Therefore, the wire fibres themselves act as a kind of reinforcement.

2.2 Tests on SFRC beams with and without additional bar reinforcement

As direct tension tests were seen as a complicated method for practical use, several specifications for indirect bending tests on beams have been established.

For statistical investigation of characteristic values a series of 71 identical specimens of a cross section of 150 by 150 mm was tested under four-point-bending loads.

Fig. 2 shows that from beginning of the formation of the

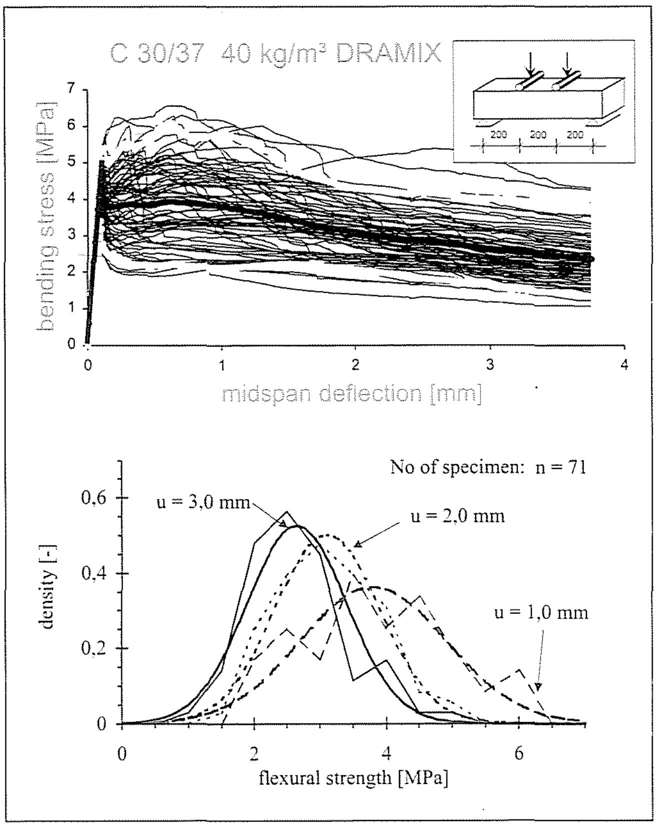


Fig. 2: Four-point-bending tests on SFRC beams – results and statistical analysis

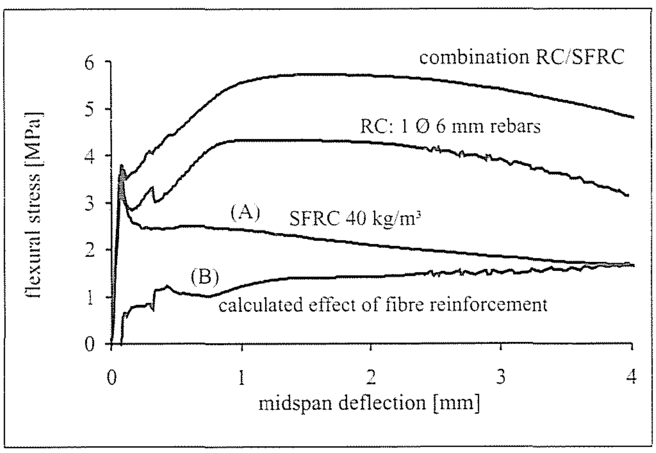


Fig. 3: Tests of SFRC and combined reinforced specimens

cracks the load-displacement curves vary in a very wide range. By the use of statistics, the curves can be described by the normal distribution. Due to declining effects of concrete fracture processes by the increase of deflection and crack widths the coefficient of variation decreases. Therefore, larger coefficients of variation are determined by the use of test methods that derive the mean value of post cracking strength of steel fibre concrete from the integration of the load displacement curves.

Four point bending tests carried out on combined bar and steel fibre reinforced members show that the effects of fibre and bar reinforcement on load bearing capacity cannot be simply added for combined reinforced members (*Fig. 3*). While load-deflection curve (A) represents the mean value of SFRC members, curve (B) is the calculated mean value of the effect of the steel fibres on the load bearing behaviour of combined reinforced beams. As curve (B) is derived by subtraction of measured values of combined and pure bar reinforced members, it is free from any fracture mechanical effects. At larger crack widths both curves (A) and (B) converge.

3. CRACKING BEHAVIOUR

The innovative development of the non-ballasted track for the DB AG (German Rail) is still far from its final stage. Although the DB AG has already approved 5 non-ballasted track systems and is testing 5 other systems under operating conditions, various construction companies have announced a number of additional developments, which, as all the others, will be protected by patent rights.

As the ballast on the highly frequented track between Waghäusel and Neulußheim on the Rhine Valley Line had to be renewed after 25 years, the DB AG, in a unique large scale test, gave 7 construction firms the opportunity to try out their systems under operating conditions, by erecting a 390 m long test stretch for the actual cost of the ballast renewal.

3.1 Track cross section and laboratory tests

The cross-section of one of the systems, invented by a consortium of the construction firms HOCHTIEF / Schreck-Mieves / Longo, is given in *Fig. 4*. It can be seen that the system basically consists of a reinforced concrete pavement with a thickness of 200 mm on a cement treated base with an underlying anti frost layer. After concreting the slab, the connecting stirrups for the supports of the rail fastenings are placed into the fresh concrete. In a third step the supports are concreted in such a way that the exact rail height is achieved. As can be seen in *Fig. 1* the longitudinal reinforcement of Ø20, cc = 180 mm has to be placed in the middle of the slab in order not to interfere with the fixings of the rail supports.

This amount of reinforcement is, according to the German concrete design regulations, sufficient for an allowable crack width of 0.5 mm due to restraint forces in longitudinal direction, which have to be expected in such a continuous endless reinforced concrete strip. In order to check the serviceability behaviour of such a system in transverse direction under loading, a standard test set-up for railway sleepers was used. This test set-up is indicated in *Fig. 5*, the tests were carried out at

Fig. 4: Non-ballasted track - cross-section

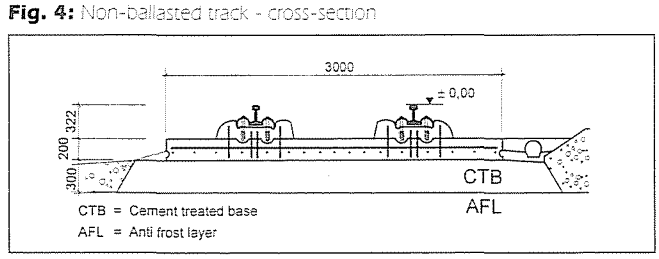
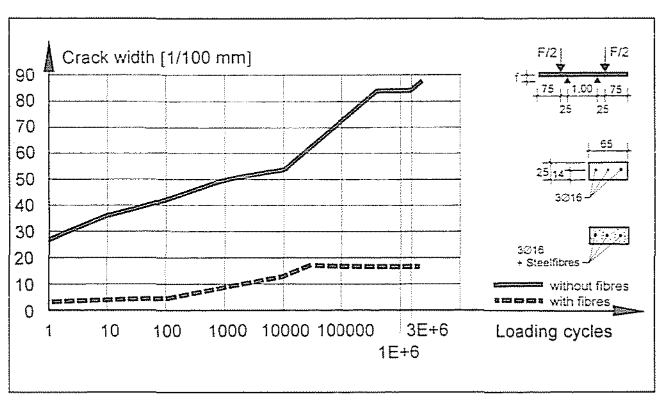


Fig. 5: Test set-up and crack width for the reinforced concrete test specimens with and without additional steel fibres.



the iBMB of the Braunschweig University of Technology. It should be noted here that the transversal bar reinforcement has a concrete cover of 110 mm.

It is debatable whether or not this test represents the real bedding conditions of such a slab, but it can be seen in *Fig. 2*, that the initial maximum crack width of approximately 0.3 mm for the reinforced concrete test specimen increased to 0.85 mm after 3 million loading cycles. This, from the serviceability point of view, is an unacceptable crack width, as, according to the DB AB regulations, only a maximum surface crack width of 0.5 mm is allowed.

The test specimen with an additional steel fibre reinforcement of 40 kg/m³ showed an entirely different behaviour (see *Fig. 5*). For this case the initial crack width of approximately 0.05 mm increased to only 0.18 mm after 3 million loading cycles. This indicates clearly that the fibres are able to reduce the surface crack width considerably, thus resulting in an improved serviceability behaviour.

4. SAFETY ANALYSIS

The probability based study of structural reliability of steel fibre concrete enables the verification of design criteria, based on characteristic values and partial safety factors. By the use of the first order reliability method a comparison of structural safety of this design method with established design codes for reinforced concrete structural members was carried out.

The safety level can be expressed by the use of the failure probability p_f or by a derived safety index β. In the European Design Codes, a value of $\beta = 4.7$ for a corresponding period of one year is fixed for normal constructions i.e. office buildings, dwellings, storage houses etc. The safety index β mainly depends on the parameters that highly influence the load bearing capacity and/or have high coefficients of variation. The basic variables used are given in *Table 1*.

For steel fibre concrete members under tension or in bending both criteria are applicable for the fibre action. Therefore, it has to be expected, that for the verification of such structures by the use of partial safety factors, the factors for the fibre action will be higher compared to other material related safety factors, i.e. for steel reinforcement or concrete under compression. It has been proven by Gossla (2000), that for steel fibre concrete under tension or bending partial safety factors of approximately $g_f = 1.65$ have to be taken into account in order to achieve a comparable level of structural reliability as for $g_s = 1.15$ for reinforcing steel. But this does not allow an economical design of structural SFRC members in general. Nevertheless, combined reinforced concrete members or SFRC structures with local bar reinforcement reach a higher level of structural safety due to the interaction of two independent types of reinforcement.

A safety analysis of the present SFRC slab on pile structure with column strip reinforcement in comparison to a reinforced concrete slab has been performed. The calculated safety index is drawn as function of the increase of design load carrying capacity due to additional bar reinforcement in the column strips by variation of the parameters of partial safety factor g_f on the fibre action. *Fig. 6* shows that due to the interaction of both types of reinforcement, a remarkable increase in structural safety can be achieved. Very high steel reinforcing rates in the column strip cause a second failure type of the inner fields only reinforced by steel fibres. Therefore, the safety coefficient drops to its original values of only fibre reinforced slabs.

Table 1: Basic variables

variable	design value	safety factor	mean value	standard deviation
pile grid [m]	4.00	-	4.00	0.05
pile diameter [m]	0.25	-	0.25	0.01
thickness [m]	0.25	-	0.24	0.0075
concrete cover [m]	0.030	-	0.035	0.005
yield stress [MPa]	500.0	1.15	560.0	30.0
compression strength [MPa]	35.0	1.5	43.0	5.0
equivalent flexural strength [MPa]	1.69	1.5 / 1.8 / 2.1	2.48	0.496

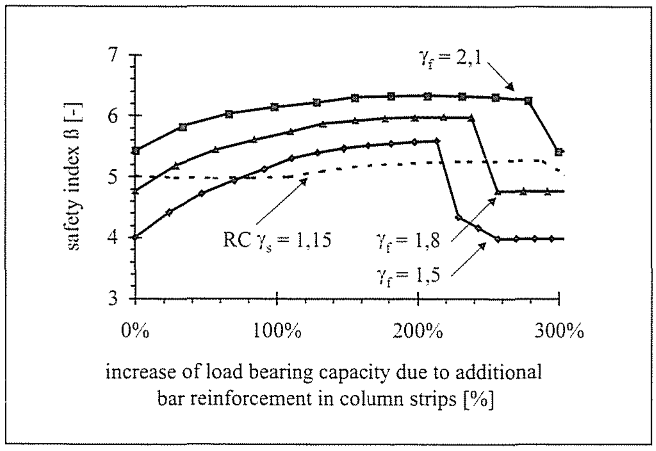


Fig. 6: Reliability analysis of RC/SFRC slabs

4. LARGE SCALE TESTS ON STRUCTURAL SFRC MEMBERS

Large scale tests on flat slab type specimens with three different reinforcing conceptions were carried out. The slabs were rested on 9 rigid columns and loaded in the centre of each of the four fields. *Fig. 7* gives an overview on the test set-up. Each column was equipped with a load cell and could be lowered in order to simulate uneven settlements. Concrete strain measurements were carried out using strain gauges. Deformation measurement was done by 16 LVDTs at locations of mayor interest.

The test-slabs consisted of steel fibre concrete with 40 kg/m³ DRAMIX® fibres RC-80/60. These are steel wire fibres with a length of 60 mm and a diameter of 0.75 mm. The fibres have hooked ends in order to achieve a better anchoring behaviour and are glued together. This provides a better in mixing performance.

The reinforcing of the slabs is seen in *Table 2*.

While the first slab P-1 only consisted of SFRC, a second slab P-2 with additional reinforcement by 6 bars of 10 mm diameter in each column strip was tested. In the third prestressed concrete slab P-3 internal unbonded 0.6" SUSPA®-tendons were placed straight in the neutral axis. Each tendon was prestressed with 0.173 MN, resulting a normal compressive stress of approximately 1 MPa in the slab. An overview on the reinforcement types and ratios is given in *Table 2*.

As the load carrying behaviour of pile supported slabs under service conditions is similar to those of flat slabs, the ma-

Table 2: Reinforcing conceptions of three different test slabs

specimen No.	P-1	P-2	P-3
fibre content [kg/m³]	40	40	40
column strip bar reinforcement BSt 500	-	6 ∅ 10 mm	-
prestressing tendons St 1570/1770	-	-	2 ∅ 0.6"

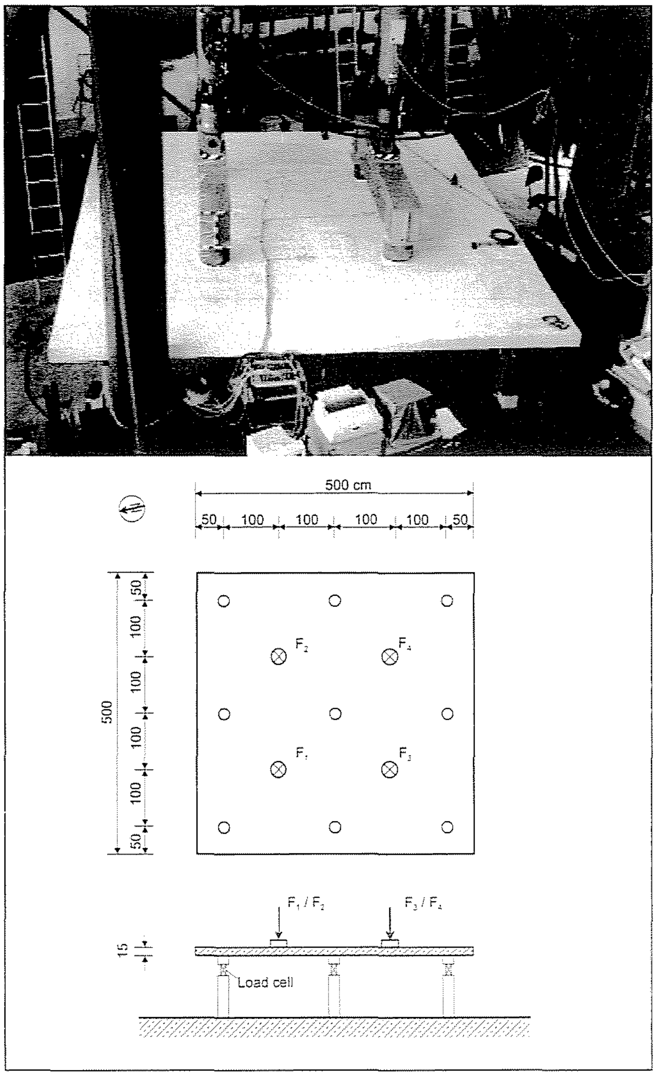


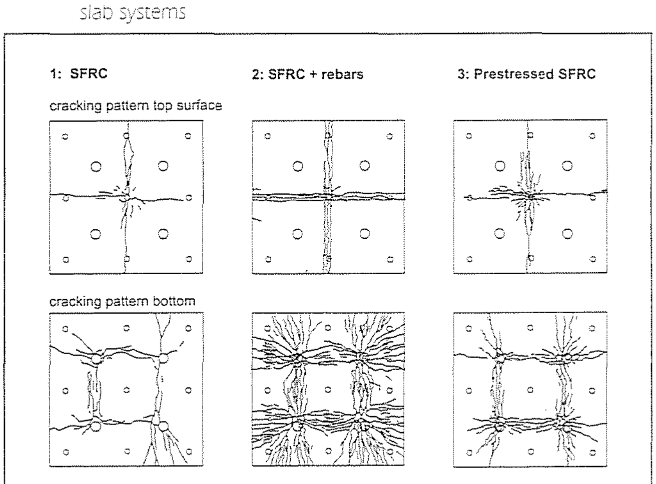
Fig. 7: Photo of test specimen with dimensions of 5.00 m by 5.00 m

jor interest was on the post cracking behaviour of the reinforced steel fibre concrete in combination with additional reinforcement or prestressing.

The tests comprised partial and full loading of the slab, a static cyclic phase, estimation of failure load as well as a simulation of uneven settlement by partial removal of either 2 or 4 edge columns. Whereas the SFRC slab failed with no load increase after developing a yield line mechanism, the slab with additional reinforcement and those with unbonded prestressing tendons showed a ductile behaviour.

It can bee seen in *Fig. 8*, that all slabs developed the outlines of the same yield line mechanism. The ductile behaviour of the 2nd slab was caused by the additional bar reinforcement in the column strips, that crossed the yield lines of positive moments in the fields and those of negative moments above the piles. Though the increase of total reinforcement ratio by the rebars in the column strips was only 20% compared to the fibre reinforcement, crack development of the whole slab could be controlled due to the higher local reinforcement ratios in the critical zones.

Fig. 8. Cracking pattern and load deflection curve of the three different slab systems



Crack control of the prestressed concrete specimen P-3 could be achieved as the toughness ratio of SFRC increases with additional normal forces. Nevertheless, the load bearing capacity had to be derived from integration of plastic deformation work in the yield lines, which are nearly the same than those of the first slab P-1.

All test slabs failed in bending, but cores drilled out of the slab near to the centre support showed a beginning punching failure mechanism in outlines. As there was no additional shear reinforcement in the slab, an influence of steel fibre action on the punching load capacity could be noticed.

5. APPLICATION OF SFRC FOR STRUCTURAL ELEMENTS

5.1 Underwater Concrete Slabs

One of the first huge construction measures in the centre of Berlin after the reunification of Germany was the erection of a new multi-functional town quarter in the area of the Potsdamer Platz, close to the former border, extending over an area of more than 70,000 m^2. As most of the buildings reach far below the groundwater level, the construction of deep building pits was necessary. The foundation depths are between 9 and 18 m in groundwater that lies approximately 2 to 3 m below the surface. A new railway station is also being built at the Potsdamer Platz, with foundation depths of up to 20 m.

At present, the verification of such an underwater concrete is based on a simple computational model. It is normally assumed, that the external loading is carried by spatial arches within the slab towards the anchoring points of the tension piles, whereas the resulting horizontal force is balanced by the external water and earth pressure on the surrounding walls. Such a simple computational model is normally sufficient for the erection of underwater slabs in smaller, straight building pits, as the tensile strength of the concrete can be neglected in this model, plain concrete can be used. For large area pits with irregular shapes and misalignments within the slab, as it was the case for these building pits, bending moments within the slab due to varying deformation behaviour of 2,000 tension piles, water pressure and the normal external force are unavoidable.

Therefore, it was highly desirable to avoid the brittle behaviour of a plain concrete slab and to try to obtain a more robust and ductile construction. Experience from tests on steel fibre reinforced concrete industrial floors on grade, which showed a highly ductile and redundant load-carrying and deformation behaviour, suggested that this material might be appropriate for these underwater concrete slabs too.

In order to investigate the suitability of SFRC slabs, three laboratory tests were carried out. The test were carried out on one plain concrete and to SFRC slabs 3 × 3 m and 28 cm thick. One important feature, the simulation of an evenly dis-

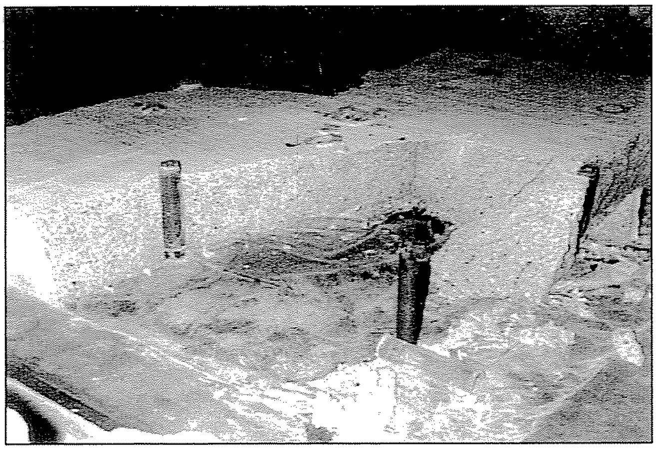


Fig. 9: Plain concrete slab after failure

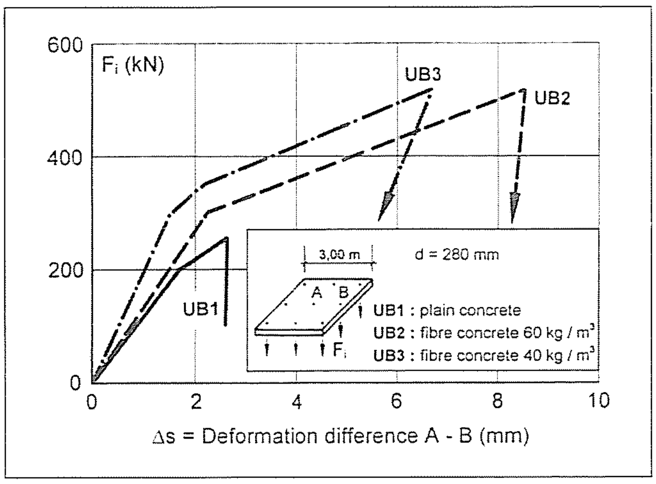


Fig. 10: Results of test loading - slabs with plain and steel fibre concrete

tributed high water pressure, was realised with a simple but effective layer of cork plates beneath the test specimen, while the load was applied by nine hydraulic jacks. In *Fig. 9* the cork layer and the anchoring points of the jacks can be seen.

For the first fibre reinforced specimen, the fibre content was 60 kg/m³ DRAMIX™ 60/0.8 and for the second 40 kg/m³ DRAMIX™ 50/0.6.

The result of these tests can be summarised to state that the ultimate load-bearing capacity of the plain test slab was reached by exceeding the concrete tensile strength. At this point a sudden failure occurred and the slab broke up into several pieces (*Fig. 9*).

In comparison to this brittle failure, the steel fibre reinforced slabs showed an entirely different behaviour. As can be seen from *Fig. 10*, the ultimate load-bearing capacity of the SFRC slabs was more than doubled compared to the plain concrete slab. It should be noted here that the tests on the two SFRC slabs had to be stopped at an average pressure of 525 kN/m², as the ultimate capacity of the hydraulic jacks was reached. Another aspect is the high deformability of the SFRC slabs. *Fig. 10* gives the ultimate test load of the three different slabs as well as the deformation difference between the middle and the edge of the slab, which was 4 to 5 times greater than that of the plain concrete slab. Other tests, carried out with a single load in the middle of the slab, but not included in this paper, lead to a deformability which was 10 to 15 times higher than for a plain concrete slab.

These test showed that SFRC slabs have an inherent redundancy and a higher degree of safety in practice. One of the SFRC slabs can be seen in *Fig. 11*, where the yield lines are clearly visible by felt pen marking. From this yield line pattern a simple

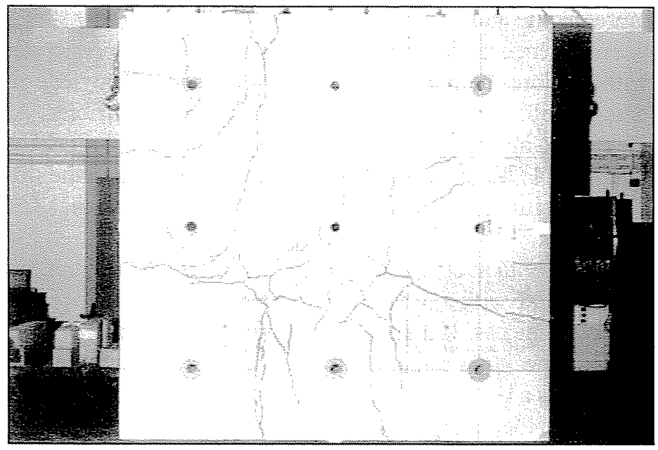


Fig. 11: Specimen with 40 kg/m³ steel fibre reinforcement

model was derived for the dimensioning of these slabs.

5.2 Pile Supported SFRC-Slabs-System

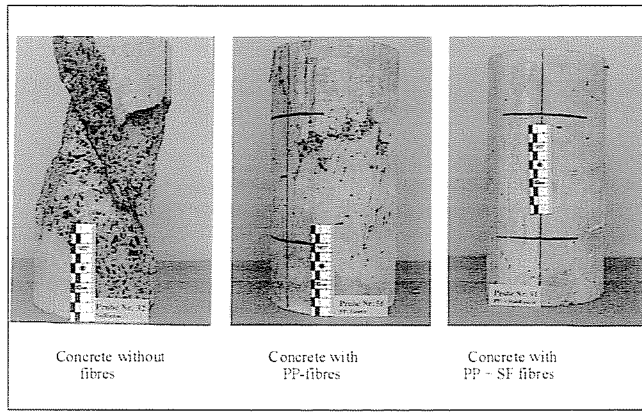
On the invention of the combination reinforcement for industrial floors resting on piles, an international patent could be achieved. In February 1998, the first project using the new combined reinforcement was carried out at Groningen, The Netherlands. In the meantime, more than 100,000 m² of such industrial floors on piles were installed in 5 European countries.

Though the cages of the column strip reinforcement were arranged in site while concreting, this work took less time than estimated and caused no problems. In order to achieve a good compaction of the fresh steel fibre concrete, a vibration cylinder should be used to vibrate those areas of the additional reinforcement. In future, standardised and prefabricated elements shall replace the reinforcing work on site. Such elements already have been developed at the iBMB lab of the Braunschweig University. By the use of prefabricated elements, only one type of cage is needed for the whole slab. This does not only result in less work on site, but also increases the performance quality of the execution.

5.3 High Performance Fibre Reinforced Concrete Columns using High Strength Steel

High strength concrete under compression shows a sudden and explosive failure. This brittleness can be altered by the addition of steel fibres to the concrete, in order to achieve a

Fig. 12: Concrete cylinders after compression strength test



more ductile and less brittle behaviour. *Fig. 12* clearly shows the difference in the failure mode of high strength concretes without fibres compared to concretes with polypropylene fibres or with a mixture of polypropylene and steel fibres. As mentioned already, high strength concrete under compression fails with a "big bang" and without proper protection the off-flying fragments can pose quite a danger to personnel and equipment.

Contrary to such a behaviour, a compression strength test of a concrete with a sufficiency high fibre content takes place almost noiseless. If a suitable concrete composition is used, the test specimen remains optically almost undamaged and still shows a remaining load bearing capacity of 20 – 30 % of the failure load (*Fig. 12*).

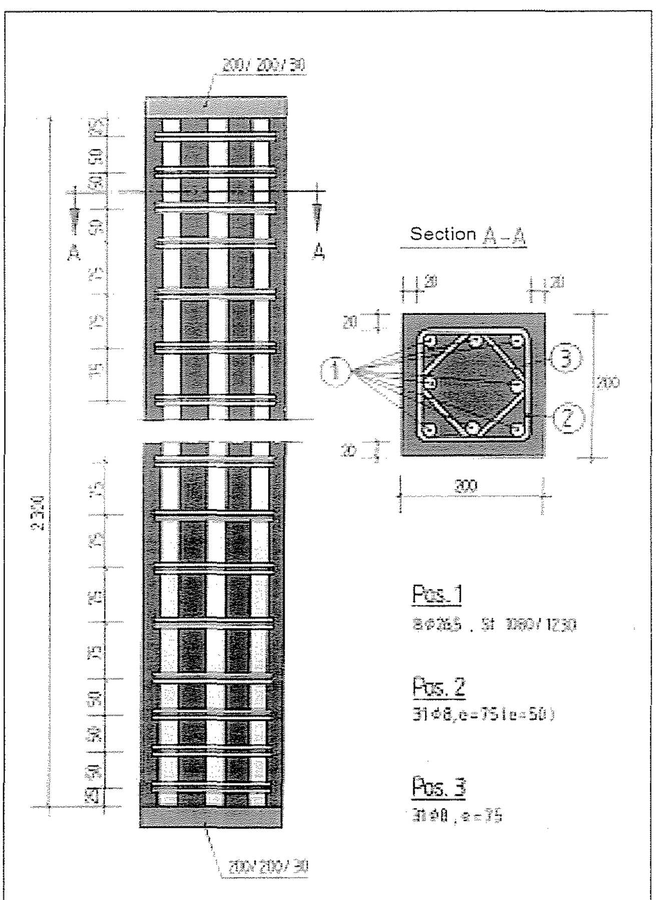
5.4 HH-Columns – The Test Specimen

For the determination of the load bearing capacity columns with a cross section of 20 × 20 cm and a length of 2.3 m were chosen. This corresponds with a l/d ratio of 11.5. This ratio was chosen as columns with a length of 4 – 4.5 m and a cross section of 40 × 40 cm will be used in the actual building.

For the concrete a compression strength $f_c \geq 125$ N/mm² was aimed at and a high strength steel St 750/1200, with a compression yield point of 930 N/mm² was used. The reinforcement consisted of 8 ∅ 26.5 mm, resulting in a reinforcement ratio of 11 %. For the stirrups ∅ 8 and 15 cm distance was chosen, the stirrup distance was reduced to 7.5 cm at the column ends. *Fig. 13* shows the column reinforcement layout.

For the load application into the columns steel plates with a thickness of 30 mm were used. The connection between the reinforcement bars and the steel plate was carried out by contact with an additional adhesive bond connection.

Fig. 13: Column reinforcement



The comparison of the stress-strain relationship obtained from cylinder tests shows the difference between normal and high strength concrete. Whereas compressive strains between 2 and 2.3 ‰ can be obtained for normal concrete, these values increase to 3 and 3.5 ‰ for high strength concrete under short term loading.

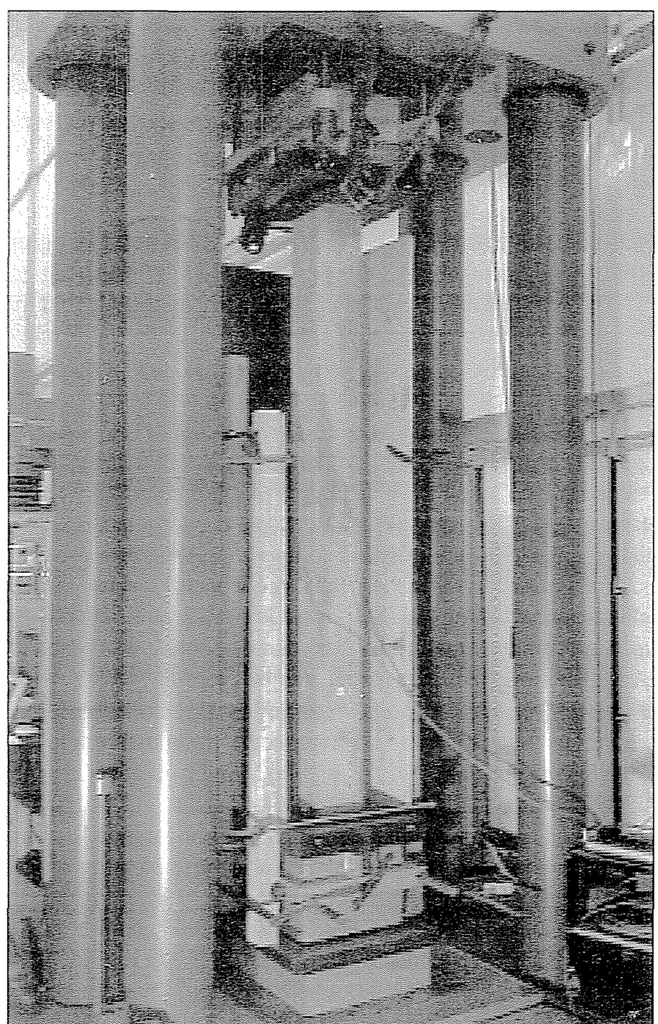
5.5 Test Procedure

The tests were carried out deformation controlled in a testing stand with a 10,000 kN hydraulic jack. *Fig.14* shows the testing stand with the test column. The load bearing capacity with concrete strength after 28 and 56 days was obtained to 6,800 and 7,300 kN respectively.

For tests like this a different loading rate alone can result in an increase of the load bearing capacity from 7,100 to 7,300 kN. The smaller load bearing capacity was obtained for a quick loading time, i.e. 15 minutes to ultimate failure, whereas the higher load bearing capacity results from a loading time of 25 hours. This influence can be explained by the beginning concrete creep at a longer loading time, resulting in a load redistribution from the concrete towards the steel. This means a load reduction in the concrete and a load increase in the steel.

This effect will be even more noticeable during the practical application, i.e. during high rise building construction, as the construction of 30 storeys can take months or even longer. In this case the concrete creep influence on the load bearing capacity of the HH-columns will be even more noticeable. This redistribution effect becomes possible by the use of a

Fig. 14: Testing stand with test column



high strength steel and the at the same time thereby higher allowable serviceability compression strain.

It is intended to further investigate the magnitude and the influence of these load redistributions from the high strength concrete towards the high strength steel in long-term tests. During these tests the influence of different fibre contents and reinforcement ratios will be studied as well.

6. CODIFICATION

In the previous chapters examples for the use of steel fibre concrete in civil engineering were given. In order to further promote the use of steel fibre concrete it is necessary to codify its properties and constitutive laws. As mentioned already in the I, introduction various standard organisations are developing their design recommendations for steel fibre concrete.

At the time being, the German Reinforced Concrete Association (DAfStb) is working on a design recommendation for steel fibre concrete, which will be published in 2005. In the following the main aspects of this design recommendations will be outlined.

As the steel fibres have to act as the "tensile component" in a structural member, together with normal rebars or not, the cross section forces an be given in such a section as shown in *Fig. 15*. The strain definitions are given in *Fig. 16*.

Fig. 15: Section forces of the steel fibres F_f and of the rebars F_s in a cross section.

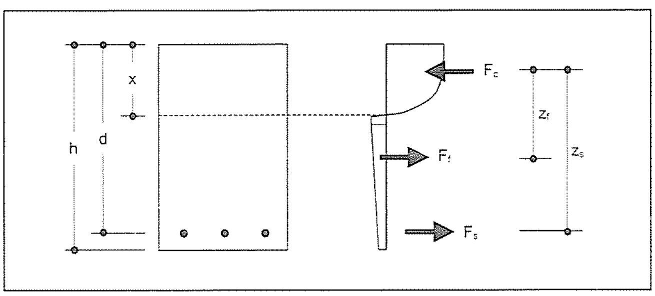


Fig. 16a: Strain definitions

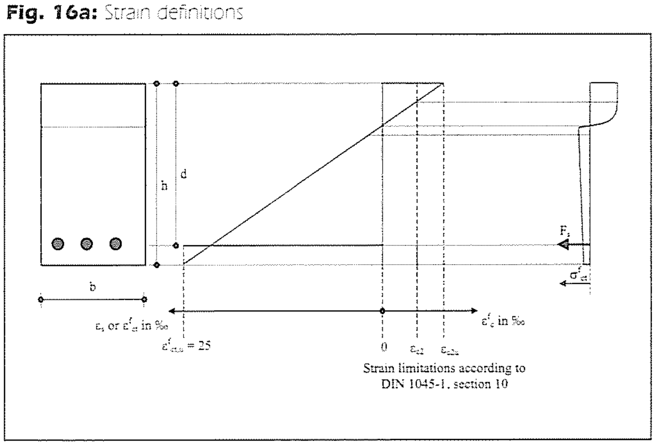
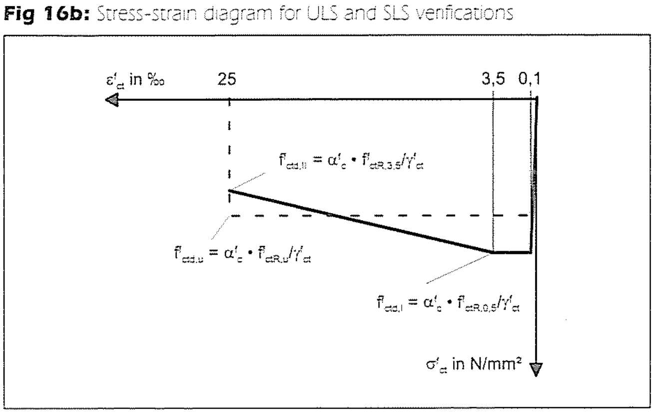


Fig 16b: Stress-strain diagram for ULS and SLS verifications



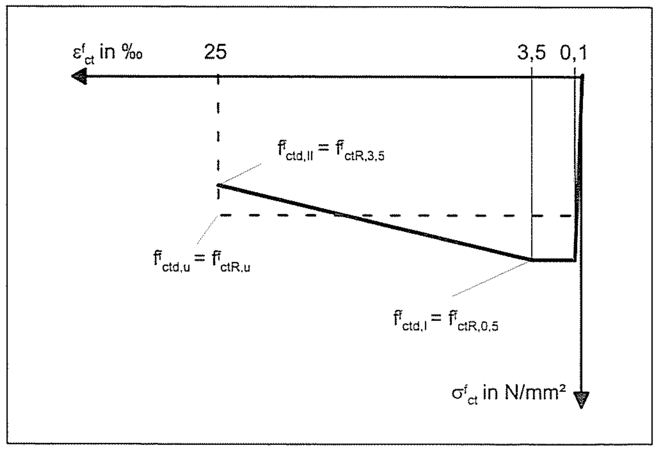


Fig.17: Stress-strain diagram for non-linear verifications

The actual structural analysis can be carried out with the stress-strain diagrams given in *Figs. 17* and *18*. *Fig. 17* shows the stress-strain diagram used for ULS verification. For SLS verifications, the safety factors γ^f_{ct} and the fatigue strength factor α^t_c (normally 0.85) can be omitted and the $f^f_{ctR,S}$ has to be used. In this case the maximum allowable strain is limited to 3.5 ‰.

The values $f^f_{ctR,0.5}$, $f^f_{ctR,3.5}$, $f^f_{ctR,u}$ and $f^f_{ctR,S}$ are derived from the basic post cracking tensile stresses for different performance classes, as given in *Table 3*. These values are derived from standard bending test as shown in *Fig. 2* for a mid-span deflection of 0.5 (deformation I) and 3.5 mm (deformation II) and have to be understood as characteristic centric tensile stress values, derived from this standard test post cracking characteristic stress values f^f_{cfk}.

$$f^f_{ct0,\delta} = f^f_{cflk} \times \beta_\delta$$
$$f^f_{ct0,u} = f^f_{cflk} \times \beta_u$$
$$f^f_{ct0,S} = f^f_{cflk} \times \beta_S$$

with:

$\beta_{0,5} = 0.37$; for deformation I ($\delta = 0.5$ mm)
$\beta_{3,5} = 0.25$; for deformation II ($\delta = 3.5$ mm)
$\beta_u = 0.37$; for the simplified stress-strain diagram
$\beta_S = 0.40$; SLS verifications: stress-strain diagram and simplified stress-strain diagram.

Table 3: Basic values of the centrical post cracking stress f^f_{ct0}

	Basic values of the centrical post cracking stress f^f_{ct0} in [N/mm²] for a deflection δ (mm)			
Efficiency-class	Deformation I (δ = 0.5 mm)		Deformation II (δ = 3.5 mm)	
F^f_{cflk}	$f^f_{ct0,S}$	$f^f_{ct0,0.5}$	$f^f_{ct0,3.5}$	$f^f_{ct0,u}$
1	*2*	*3*	*4*	*5*
0	< 0.32	< 0.30	-	-
0.8[a]	**0.32**	**0.30**	**0.20**	**0.30**
1.2	**0.48**	**0.44**	**0.30**	**0.44**
1.6	**0.64**	**0.59**	**0.40**	**0.59**
2.0	0.80	0.74	0.50	0.74
2.4	**0.96**	**0.89**	**0.60**	**0.89**
2.8	1.12	1.04	0.70	1.04
3.2	**1.28**	**1.18**	**0.80**	**1.18**
3.6[b]	1.44	1.33	0.90	1.33
4.0[b]	1.60	1.48	1.00	1.48

a) only for plate like structures (b > 5h)
b) for steel fibre concrete of these efficiency classes an individual approval by the building authorities is necessary.

Finally

$$f^{I}_{ctR,I} = \kappa_b \cdot f_{ct0,i}$$

with

$\kappa_b = 0.125\left(5 + \frac{b}{h}\right)$ representing the influence of the cross section.

The value κ_b is limited between $0.75 \le \kappa_b \le 1.25$.

Using these efficiency-classes a steel fibre concrete according to this recommendation is therefore defined for example as

C30/37 L1.6/1.2

with

C30/37 the concrete compression strength according to DIN EN 206-1 and DIN 1045-2, and

L1.6/1.2 steel fibre concrete of the efficiency-class 1.6 for deformation I and the efficiency-classes 1.2 for deformation II.

6.1 Safety concept

The safety concept according to these recommendations and the partial safety factors indicated in *Figs. 16* and *17* are given in *Table 4.*

Table 4: Safety factors

Partial safety factor for	STEEL FIBRE CONCRETE	STEEL FIBRE CONCRETE WITH NORMAL REINFORCEMENT
Reinforcement (tension) γ_s	acc. to DIN 1045-1	
Steel fibre concrete – post cracking γ^f_{ct} ¹)	1.25	1.25
Steel fibre concrete – uncracked γ^f_{ct} ²)	1.8	–
SYSTEM LOAD BEARING CAPACITY FOR NON-.LINEAR VERIFICATION γ_R ³)	1.4	1.35 (without exact verification)⁴⁾
Permanent actions γ_G	Acc. to DIN 1055-100 with the consideration of combination values	
Variable actions γ_Q		

¹) at least efficiency class 0.8 for deformation II

²) acc. to DIN 1045-1, section. 5.3.3 (10) for dimensioning of non-reinforced building members

³) EXPLANATION OF F_F AND F_S – SEE *FIG 15*

⁴) WITH EXACT VERIFICATION: $1.3 + \frac{0.1 \cdot F_f}{F_f + F_s}$

7. CONCLUSION

Based on this work a new construction principle for a reinforcement combination for industrial floor slabs resting on piles was developed. Large scale tests carried out at the laboratory of the Braunschweig University of Technology, Germany showed, that a partial bar reinforcement of SFRC slabs can increase the overall serviceability and ultimate load performance. For example, applications of this international patented construction have been executed in UK, the Netherlands, Sweden, Belgium and Germany since 1998.

The design recommendations of the DAfStb, which will finally be published in 2005, will certainly promote the use of steel fibre concrete in civil engineering.

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THE BRIDGE OVER THE KRKA RIVER ON THE ZAGREB – SPLIT MOTORWAY


Prof. Zvonimir Marić Dr. Zlatko Šavor

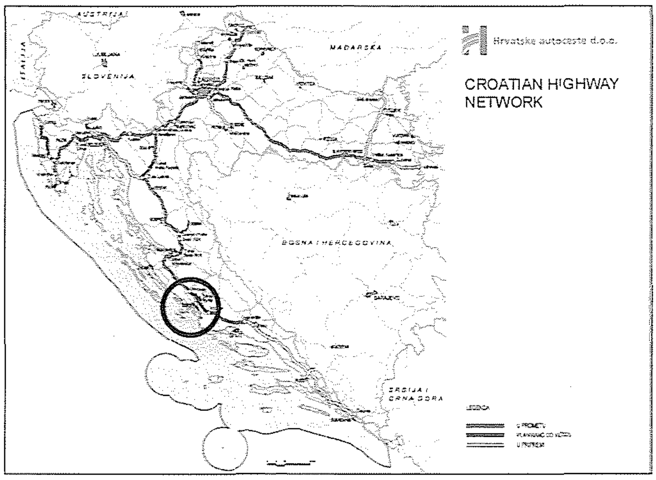
An inadequate design with respect to durability and, first of all, an insufficient concrete cover, as well as severe marine environment conditions, have caused serious damage to the large Croatian reinforced concrete arch bridges built by free cantilevering method in the 1960's and 1970's. The bridge across the Maslenica strait on the Zagreb – Split motorway, completed in 1997, was designed using recent knowledge concerning the durability, but its concrete deck structure was too heavy, which also affected the arch dimensions and the temporary supporting structures. The designer of the recently completed bridge over the Krka river, on the same motorway, adopted an elegant composite deck structure, and thus eliminated the above drawbacks, furthermore, it resulted in the remarkably pleasing appearance of the bridge.

Keywords: durability, concrete arch bridge, free cantilevering, composite deck structure, appearance

1. INTRODUCTION

The large Croatian reinforced concrete arch bridges, built by free cantilevering method in the sixties and seventies, considerably contributed to the renaissance of concrete arch bridges. However, inadequate design with respect to durability (and first of all insufficient concrete cover) as well as severe environment conditions, have caused such damage that the maintenance and repair costs have become increasingly higher. Moreover, the famous Krk bridge, with the second largest arch span-length in the world, has suffered such damage that engineers have not as yet found a proper remedy. The new momentum in the Croatian motorway construction (*Fig. 1*) has given opportunity to designers to apply the knowledge recently gathered about the durability of concrete structures in severe marine environment on new projects. The direct result of such approach was the bridge across the Maslenica strait on the Zagreb – Split motorway, completed in 1997 (Čandrlić et al., 1998). However, rigorous respect of requirements concerning durability of deck structures resulted in the use of heavy precast prestressed concrete girders (thick webbed T-beams), which in turn made it necessary to adopt mighty head beams on the piers, some of which are very tall (the head beams were needed for supporting the precast girders). These head beams had three principal drawbacks: they presented a considerable additional dead load to the arch; their erection was complicated and time-consuming; and their appearance was definitely not pleasing (this last issue cannot be disregarded, having in view the position of the bridge – in a region which is a favourite tourist destination). Therefore, the designer did a radical step forward: instead of a heavy concrete deck structure, he adopted an elegant composite structure, which made it possible to completely abandon head beams and, what is also important, the arch itself thus became considerably thinner and lighter. Thanks to this modification, the structures for the temporary support of the arch become significantly lighter and cheaper. And, of course, the overall construction time of the bridge is considerably reduced. This will be briefly presented through description of the bridge over the Krka river on the same motorway, based on the papers written by the designers (Šavor et al., 2005) and that written by the site managers (Trlaja and Ljutić, 2005).

Fig. 1: The Croatian motorway network

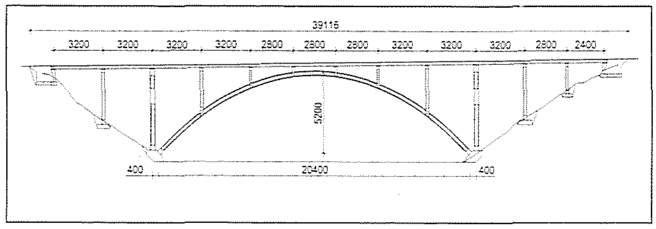


2. DESIGN

The bridge lies both horizontally and vertically in a straight line, with the grade line sloping at 1.326 %, approximately 66 m above the river level (*Fig. 2*). The width of the river is about 190 m, and the distance between the valley banks at the level close to the grade line is approximately 390 m. The overall width of the bridge is 22.56 m, comprising two carriageways separated by a median strip 3.0 m in width (*Fig. 3*).

The main structure is a concrete arch, 204 m in span, with the rise of 52 m (*Fig. 2*), and with the rise to span ratio f/L = = 1/3.92. The arch is fixed and has a double cell box cross-section with constant outer dimensions. The arch axis is an inverted catenary's function, so that bending moments due to permanent loads are minimum. The arch cross-sectional di-

Fig. 2: Longitudinal layout of the bridge



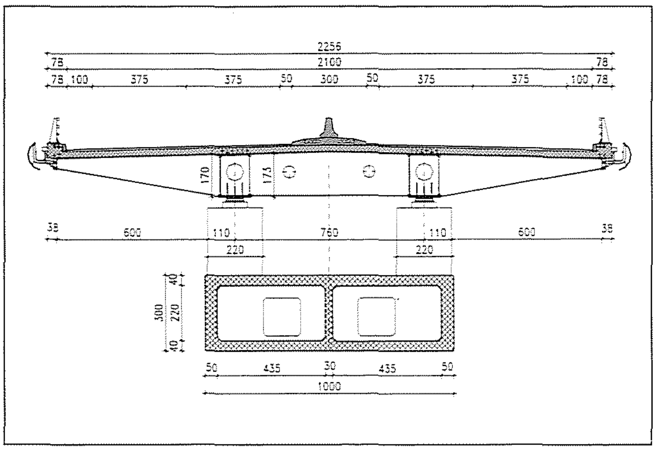


Fig. 3: The bridge cross-section

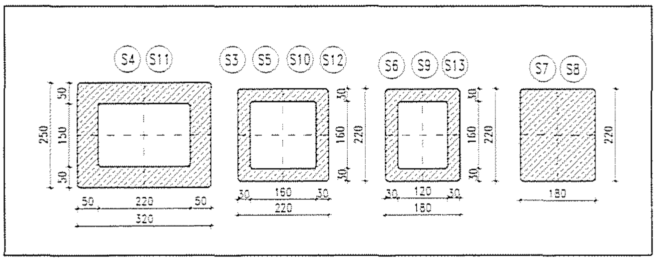


Fig. 4: Cross-sections of the columns

mensions are constant in the length of 92 m, measured symmetrically from the crown. The dimensions of the arch box-section are also presented in *Fig. 3*. The thickness of the arch slabs gradually increases to 0.6 m over the first 10 m (measured from the springing in the horizontal projection). The diaphragms under the spandrel columns are vertical and their thickness is equal to the width of the columns (i.e. to their outer dimension parallel to the bridge axis). The concrete class C45/55 was adopted for the arch. The concrete class of all structural elements was determined according to E DIN 1045-1 (1998). Such a high strength was primarily motivated by the durability considerations. An overall quantity of the arch concrete is 2,988 m^3, while the total mass of the arch reinforcement is 747 t.

Bridge piers, 3.39÷55.57 m in height, consist of two individual columns, spaced at 7.6 m from one another. They are of a box-type cross-section, except for the two shortest ones near the crown. The cross-sections of the columns are presented in *Fig. 4*. The concrete class C35/45 was used for the piers. It should be emphasized that, thanks to the use a composite deck structure, the head beams could be abandoned. This brought considerable advantages both in terms of alleviating and shortening construction procedure and in improving the bridge appearance, which is highly important because the bridge is located in an exceptionally demanding environment: very close to the Krka National Park. There is an unusual detail in the longitudinal layout of the bridge: the abutments are marked as U1 (*upornjak* in Croatian) and U14, and the piers start with S3 (*stup* in Croatian), instead of S2 (*Fig. 2*). This happened during the elaboration of the design: due to poor terrain conditions the longitudinal axis of the bridge had to be shifted laterally for some 30 m. As a result of this, the bridge was shortened for approximately the length of one span and hence the pier S2 was abandoned.

The bridge abutments are full-closed with the stepped-up wings of variable length, in accordance with the terrain configuration. The concrete class C30/37 was used for the abutments. The arch and the piers at the arch springs have common massive foundations. Combined footings are used for other piers. The concrete class C25/30 was used for all foundations. The bridge is founded on the rock with an allowable rock stress of 1.5 MN/m^2.

The deck structure runs continuously over twelve spans, L = 4×32.0 + 3×28.0 + 3×32.0 + 28.0 + 24.0 m, and the overall length of the bridge is 391.16 m. It is a composite structure consisting of the steel plate grillage and the precast concrete deck slab. The composite cross-section is formed of two welded steel plate main box girders spaced at 7.6 m from one another, and of steel plate I-shaped cross-beams spaced at 4.0 m intervals, connected to the deck slab with welded shear connectors. Outer ends of the cross-beams are interconnected with secondary longitudinal stringers. The depth of the main girders is 1.7 m (measured in the mid-plane of the outer web) and its upper chord follows the carriageway slope. This depth is constant along the bridge. The thickness of the upper chord ranges between 20 and 55 mm, and the lower chord thickness ranges from 25 to 60 mm. The webs are 12 mm thick in the span, and 16 mm in the zone close to the supports. The box width is 1.2 m (measured between the webs) and the upper chord width is 1.3 m.

The cross-beams between the main girders are of a slightly variable depth (the upper chord, #400×20 mm, follows the carriageway slope). The lower chord, #400×25 mm, is horizontal, while an average depth of the web is 1,624 mm, and its thickness is 14 mm. At the support, the web thickness increases to 20 mm, and the lower chord width to 500 mm. The cross-beam cantilevers are 6.52 m in length, and the web depth ranges between 400 and 1,624 mm, while its thickness varies from 14 to 20 mm. The upper chord cross-section is #400×20 mm, and the width of the lower chord is variable: from 500 mm at the support to 300 mm at the end. Its thickness remains constant: 25 mm. The secondary longitudinal stringers are also I-shaped, but their cross-section is constant: the chords are #300×20 mm, and the web is #400×12 mm.

The deck slab is made of precast elements, 25 cm in constant thickness, which cover fields of the steel grid. As the ratio of side lengths is close to 2 ≈ (7.6-1.3):(4.0-0.4), the main bearing direction is longitudinal. The elements are interconnected by means of wet joints over the steel grid girders where their reinforcement is overlapped (*Fig. 5*). The concrete class C45/55 was used for the slab. The slab is heavily reinforced: with as much as 83 kg/m^2 of St 500/550.

The concept used in selecting places with longitudinally fixed bearings is interesting. One of natural places for such selection is above the arch crown, because in such a case the longitudinal horizontal forces may be taken over by the arch in the most straightforward and efficient manner. However, earthquake forces may pose a considerable problem, because

Fig. 5: Detail of the deck slab wet joint over a cross-beam

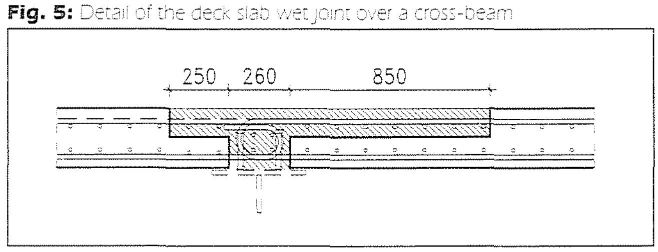
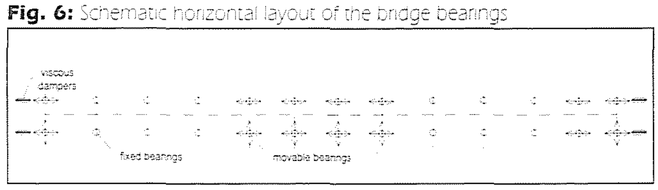


Fig. 6: Schematic horizontal layout of the bridge bearings



there is no bearing that could take over the earthquake force caused by the deck structure weight. Of course, one could establish a fixed connection between the short piers near the arch crown and the deck structure, but any major earthquake would destroy them. Therefore, the designer decided that the arch should take over regular horizontal forces (due to temperature variations, braking force, etc.) via fixed bearings on one of short piers near the crown and, in case of an earthquake, these fixed bearings would suffer a controlled transformation into the longitudinally movable ones, and the earthquake force is taken over by the dampers located on the abutments (smaller portion is taken over by the tall piers (S3÷S5 and S10÷S12, *Fig. 6*). In this way, the whole bridge is made sufficiently stiff for regular actions, and sufficiently safe in case of an earthquake. The dampers and the bridge bearings were produced by the Italian manufacturer FIP. The expansion joints are provided at the abutments only. Their displacement capacity is ± 240 mm, and they are made by the Mageba company.

The deck slab is protected against water penetration by a single-layer waterproofing made of welded bitumen sheets. The asphalt-concrete pavement consists of the 4.0 cm base course, and the 4.0 cm wearing surface. The cross slope of the carriageways is 2.5 %, to facilitate drainage. Due to the vicinity of the Krka National park, the closed drainage system of the bridge is designed, with transverse openings in the outer edge beams, spaced at 4.0 m, and longitudinal open galvanized steel channels, hung on the outside, transporting the water to the bridge ends. Longitudinal plastic pipes, to be used for various installations crossing the bridge (traffic signalling, road lighting, telecommunications), are embedded in the median strip concrete. A particularity of this bridge is that the drainage water of a portion of the motorway is transported across the bridge by means of two plastic pipes 400 mm in diameter, placed between the main longitudinal girders (*Fig. 3*).

The safety barriers placed in the median strip are New Jersey type barriers, while the outer ones are type BN4 steel barriers, fully integrated with the railing. This type of barriers (first introduced by the French Road Authority) is 1.0 m in height and its bottom depth is 0.5 m. They are used for preventing vehicles from falling from the bridge. Still they are sufficiently transparent, which is very important having in mind geographical location of the bridge – in a region frequently visited by tourists (the Krka National Park was visited by as many as 600,000 tourists last year, and the number of visitors is steadily increasing). The barriers have expansion joints in the same cross-profile as the deck structure.

Although this bridge is not situated in a particularly aggressive maritime environment (the salinity of the Krka river water at the bridge axis is approximately 4.5 times less than that of the sea water), its structural dimensions were chosen as if this were the case, based on the design of the Maslenica bridge, completed eight years ago. In addition, a low permeability concrete was used, Portland cement PC-30z-45s with 20 % of slag, which should increase the bridge durability. The concrete cover for the structural elements was set as follows: piers: 5.0 cm; arch: 6 cm (outer), 4 cm (inner); deck slab: 4.5 cm (top), 3.5 cm (bottom) and foundations: 10.0 cm.

A stringent quality control system and thorough concrete curing methods were prescribed in the design. The monitoring devices used on the bridge consist of the displacement, local temperature and corrosion sensors installed at mostly stressed points in the arch and in the deck structure. The bridge is designed on the basis of German DIN standards, because the corresponding Croatian codes have not as yet been developed.

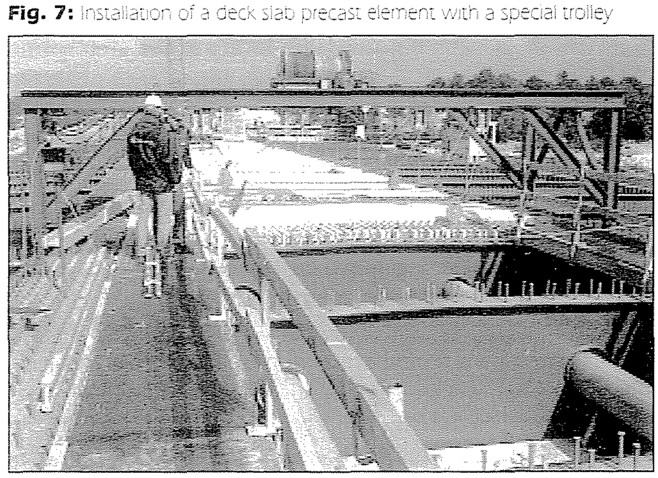
3. CONSTRUCTION

The preparatory construction works started in September 2002, and the construction works in January 2003. It should be noted that the construction period was scheduled for 22 months, and that the deadline was respected. To achieve this, the number of workers had to be increased by some 25%, and every calendar day was regarded as a working day. The concreting was generally done during the night. The arch abutments (integrated with foundations of S4 and S11 piers) were cast after completion of the roads providing access to the locations of bridge piers and abutments. As these foundations are 7.5 m in height, they had to be cast in eight layers and the concrete hardening temperature was measured at 12-hour intervals. A special type of scaffolding, supporting the reinforcement cage and working platforms, was made of light steel tubes and it remained embedded in the concrete. The installation of temporary rock anchors needed for free cantilevering erection of the arch started at the same time. Altogether 76 anchors of Dywidag type were installed, 38 on both banks. They were distributed as follows: third level: 18x12 Ø 0.62"; second level: 12x9 Ø 0.62" and first level: 8x9 Ø 0.62" The length of rock anchors was 25-27 m, out of which 18 m was free length, and 7-9 m was anchor length. The allowable capacity of each 0.6" strand is 235.5 kN.

The abutment and wing walls were cast in the Doka formwork, and the piers in the Doka climbing formwork. The 6.5 t cable-crane, 510 m in span length, with the possibility of lateral movement of 14.5 m in both directions, was used for site transport. When practicable, two tower cranes, installed at the S4 and S11 foundations, were also used. The piers were cast in segments up to 5.0 m in length.

The steel grid elements, manufactured in the Đuro Đaković plant in Slavonski Brod, were transported by train to Šibenik and from there by trucks to the site. The steel grillage portions extending over the approaching spans were assembled and welded on two flattened areas behind the abutments. The assembled units (extending from the abutments to the arch spring piers /S4 and S11/) were incrementally launched. These parts of the deck structure had to be erected before completion of the first fourths of the arch. Namely, the stay pylons supporting the third level of stays and backstays were installed on the deck structure above the piers S4 and S11. The portion of the deck slab between the main steel girders was also to be completed before installation of the stay pylons. The precasting of the deck slab elements was performed on the site. The elements were transported by a truck crane to the abutments, and were then moved along the main girders and placed into final position with a special trolley (*Fig. 7*).

Fig. 7: Installation of a deck slab precast element with a special trolley



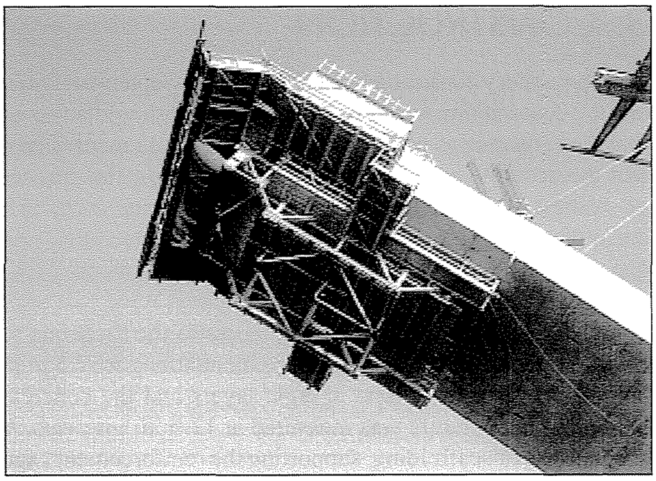


Fig. 8: The formwork carriage

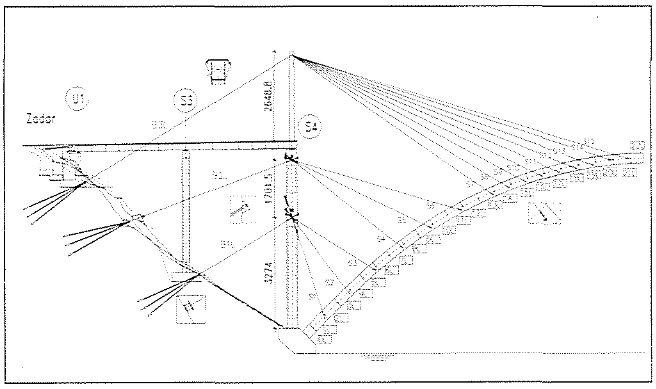


Fig. 9: The three-level system for temporary support of the arch

After successful completion of the Maslenica bridge in 1997, the procedure of arch erection by free cantilevering became better known and therefore the number of unexpected working design details was reduced. Still, having in mind numerous differences between the two designs, particularly in the rise to span ratio and in outer dimensions of the arch, full attention had to be paid to all details throughout the arch erection. The formwork carriages, previously used for the Maslenica bridge arch concreting, were slightly modified for this use (*Fig. 8*), but their weight remained virtually the same: about 55 t. The length of the segments was the same as that used on the Maslenica bridge: 5.25 m, starting symmetrically from the initial segments 3.0 m in length, with solid cross-section. These initial segments were cast in two phases, using a special formwork resting on specially made steel trusses. The piers at the arch springs were extended by auxiliary steel staying pylons 28 m in height (this is an another detail where a modification with regard to the Maslenica bridge was made: those stay pylons were 23 m high) to facilitate successive cantilevering of the arch. The arch was supported during erection with stays radiating from two levels of the arch spring piers and from tops of the stay pylons, and at each place the equilibrium was maintained by backstays (*Figs 9* and *10*), connected on the other end to the rock anchors via steel transfer beams. All the stays were made of Dywidag-type tendons. The safety factor for the stays and backstays, was 1.8 and 1.9, respectively.

With careful planning, based on 24-hour working days, the arch erection was finished in nine months (July 2003 – March 2004). The actual duration of the construction was even shorter, because strong winds and heavy rains made the work impossible during two weeks. The stay releasing procedure was similar to that used on the Maslenica bridge: hydraulic jacks at the mid-height of outer arch webs, and steel struts at the corners

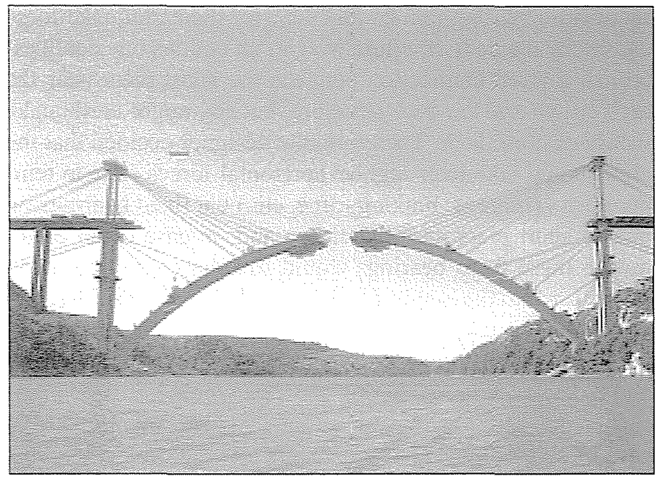


Fig. 10: The three-level system for temporary support of the arch during construction

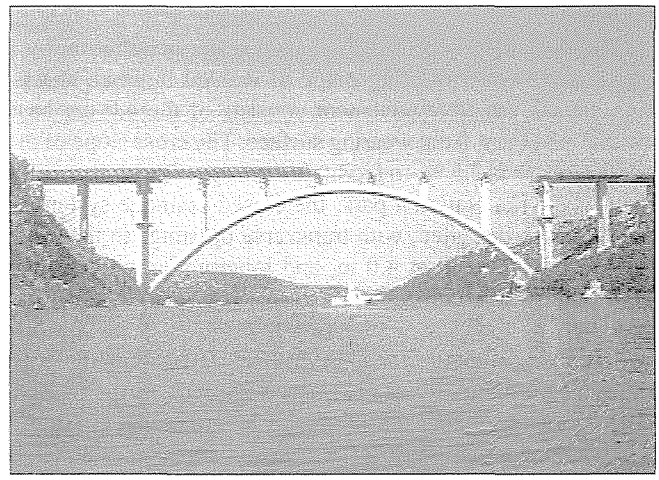


Fig. 11: Launching the steel grid portion above the arch

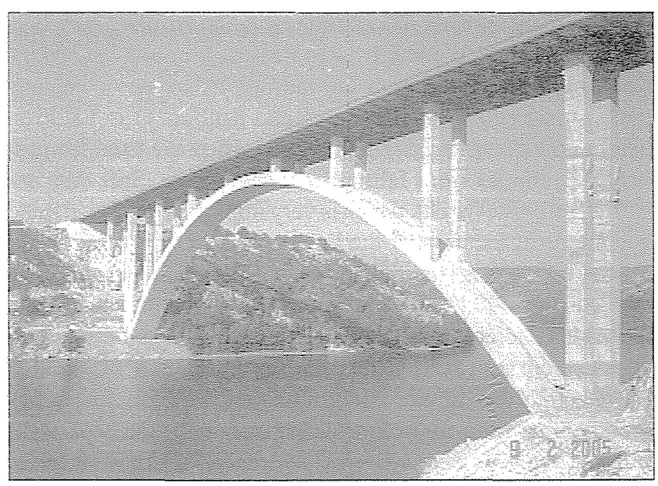


Fig. 12: An overall view of the completed bridge

of the arch box section. The arch axis was designed and executed as a slightly overelevated structure (150 mm maximum elevation in the crown) so that the designed arch shape is to be achieved only after completion of the long term creep and shrinkage.

The steel grillage over the arch was partly assembled in the same area behind the abutment U14 and partly on the approaching deck structure and then launched over the approaching spans until it reached the S11 pier. After that it was pulled over the arch using a launching nose (*Fig. 11*), and was finally lowered onto the bearings. Hydraulic jacks and supports made of halves of oak sleepers were used to properly coordinate the lowering procedure.

The deck slab elements on the steel grillage cantilevers, were installed with the aid of truck cranes moving along the previously completed part of the deck slab between the main longitudinal steel girders. The finishing works were completed in an usual way. An overall view of the completed bridge is presented in *Fig. 12*.

4. PARTIES INVOLVED

The design was made at the Civil Engineering Faculty (CEF) of Zagreb and the main designer was Zlatko Šavor, PhD. The main contractor was Konstruktor-Inženjering from Split; the site managers were Slobodan Brzica and Davor Trlaja. The subcontractor Geotehnika from Zagreb was in charge of rock anchors installation, while a small shipyard from Kaštel Sučurac (near Split) performed all necessary modifications of the steel transfer beams, formwork carriages and stay pylons. However, the main designer provided solutions for carriages, auxiliary stays and back stays, comprising static calculations and elaboration of erection phases. It should be noted that all erection phases, as actually executed, were included into the calculation procedure based on the SOFISTIK computer package. The steel grillage segments manufacturing, assembling and welding as well as the launching and lowering of the grillage units was performed by the Đuro Đaković company from Slavonski Brod. The assembly, installation and stressing of the stays and backstays was done by the main contractor. The monitoring system was designed and installed by Mladenko Rak, PhD, from the CEF, and the supervision was done by the Civil Engineering Institute of Croatia, under the guidance of Zvonimir Marić and Siniša Jakšić.

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THE FIRST EXTRADOSED BRIDGE IN HUNGARY

János Becze

The first extradosed bridge of Hungary was built in the junction of the M7-M70 highwasy (Fig. 1). The superstructure of the prestressed concrete extradosed overpass is a three-supported, hollow slab with elevated edge beams at the two sides. At the middle support pylons were constructed together with the edge beams. The external cables, led through the pylons, transfer the vertical component of their deviation forces at transversal beams. The cable forces of the structure can be checked anytime with the help of force-measuring cells. The extradosed cables show suggestively the statical behaviour of the structure.

Keywords: extradosed bridge, structural height – span ratio, cable strength/force, aesthetics

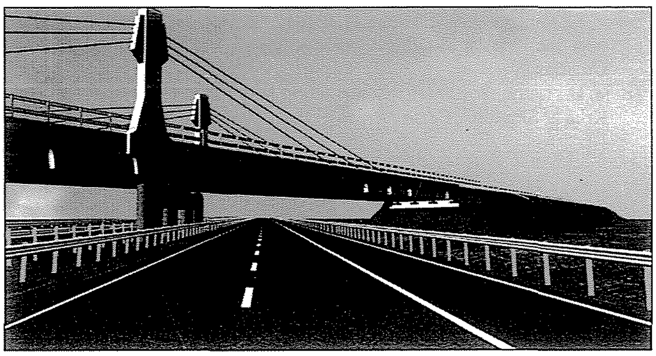
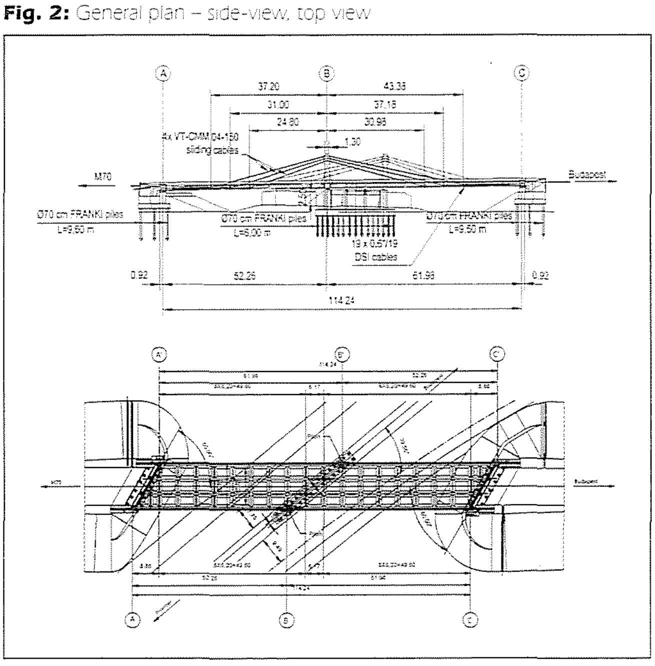


Fig. 1: Presentation in 3D

1. INTRODUCTION

The bridge of the planned junction section crosses the motorway in an angle of $\alpha = \sim 40^{o}$. The originally planned structure was designed as a traditional open monolithic reinforced concrete plate-girder bridge with 5 supports. The implemented underpass has a prestressed concrete superstructure with two spans, stayed with external sliding extradosed cables. In order to have a better visibility on the curved motorway we wanted to build less supports. A superstructure symmetrical to the ichnography point was constructed in this skew junction. The structural height (h = 1.60 m) had to remain unchanged. The ratio of the increased span compared to the structural height is L / h = 38.75. The total length of the superstructure is 116.08 m. The above-mentioned reasons explain the application of the extradosed system (*Fig. 2*).

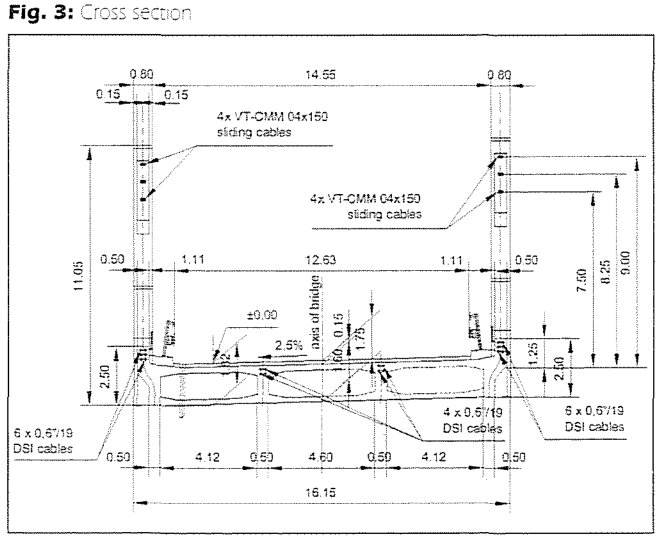
Fig. 2: General plan – side-view, top view



2. DETAILED INFORMATION

The bridge with three supports has got pile foundation. The piles are of franki-system with dia 70 cm, their length at the abutments is 9.50 m, and 6.00 m under the pier. We formed the parallel arm walls of the abutments like a corridor. The external cables can be drawn in through the corridor in the line of the edge beams. This can also facilitate the change of the cables, if necessary later, anytime after the construction. The pier is in the separating lane of the motorway. The columns of the pier are 1.30 m wide due to the nearby clearance. The reactions of the superstructure are taken by round shaped columns under the longitudinal inner beams and by extended round shaped columns under the edge beams. The longitudinal fix support of the superstructure is also placed on the pier. The superstructure has got an open plate cross-section, with two longitudinal inner beams, and two edge beams (*Fig. 3*). The reinforced concrete slab with the longitudinal inner beams is 1.60 m deep, while the edge beams are 2.50 m deep. Transversal girders at each 6.20 m stiffen the deck slab. We de-

Fig. 3: Cross section



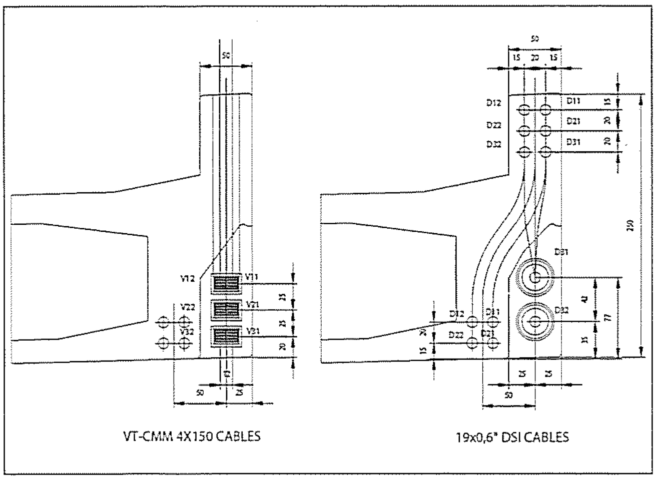


Fig. 4: Cable arrangement in the edge beam

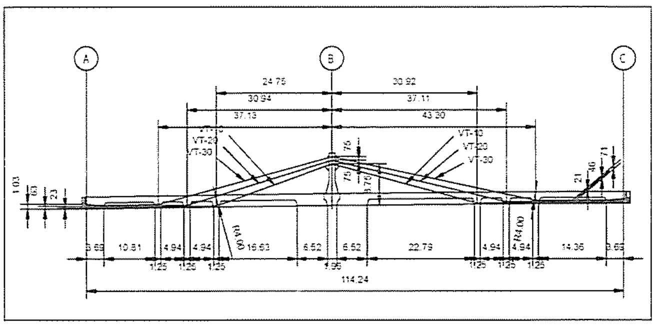


Fig. 5: Tracing of the VT-CMM cables

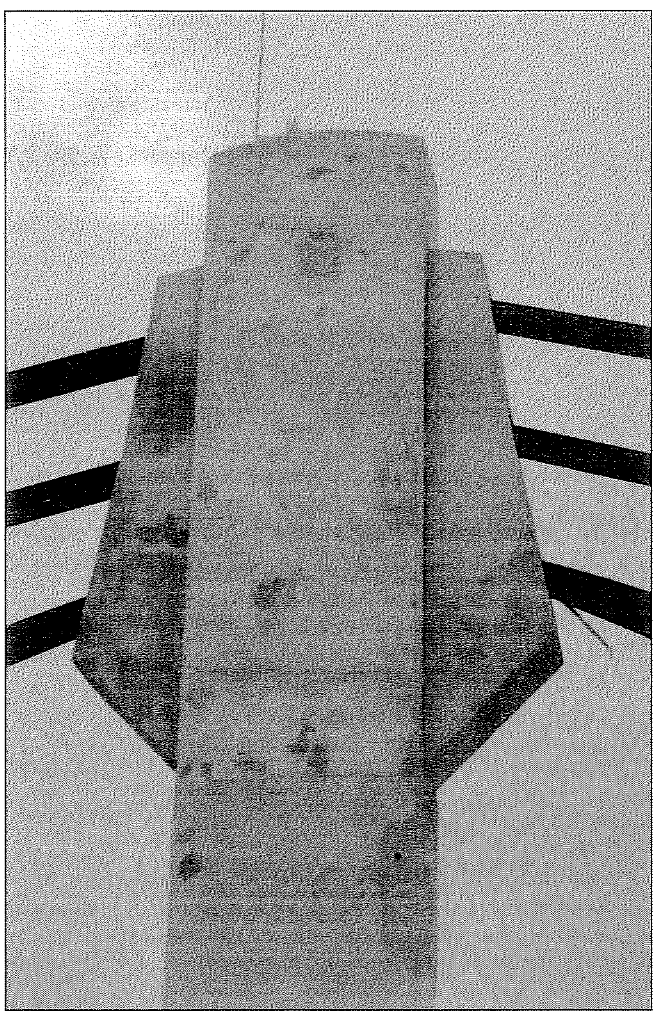


Fig. 6: Leading the cables through the pylon

signed two pylons above the middle support rigidly fixed to the edge beams.

In the longitudinal inner and edge beams a part of the bending moment is taken by 4 – 4 pieces of post tensioned, injected DSI tendons. Each tendon consists of 19×0.6" strands. In addition external cables were also built into the edge beams. These sliding cables are VT – CMM 16-150 type prestressing elements (*Fig. 4*).

The transversal girders lead a part of the vertical loads of the low inner longitudinal beams to the edge beams, therefore their bending load increases significantly. Under the present circumstances the load bearing capacity of the edge beams can be increased to the required extent only with the help of the sliding cables coming out of the delineation of the beam and led up to the pylon (*Fig. 5*). We led the VT – CMM external cables up to the pylon, which can freely slide through the deviators concreted into the pylon (*Fig. 6*).

Coming out of the anchoring block at the end cross girder

Fig. 7 and 8: Steel structures of deviators placed into the formwork, and after concreted in.

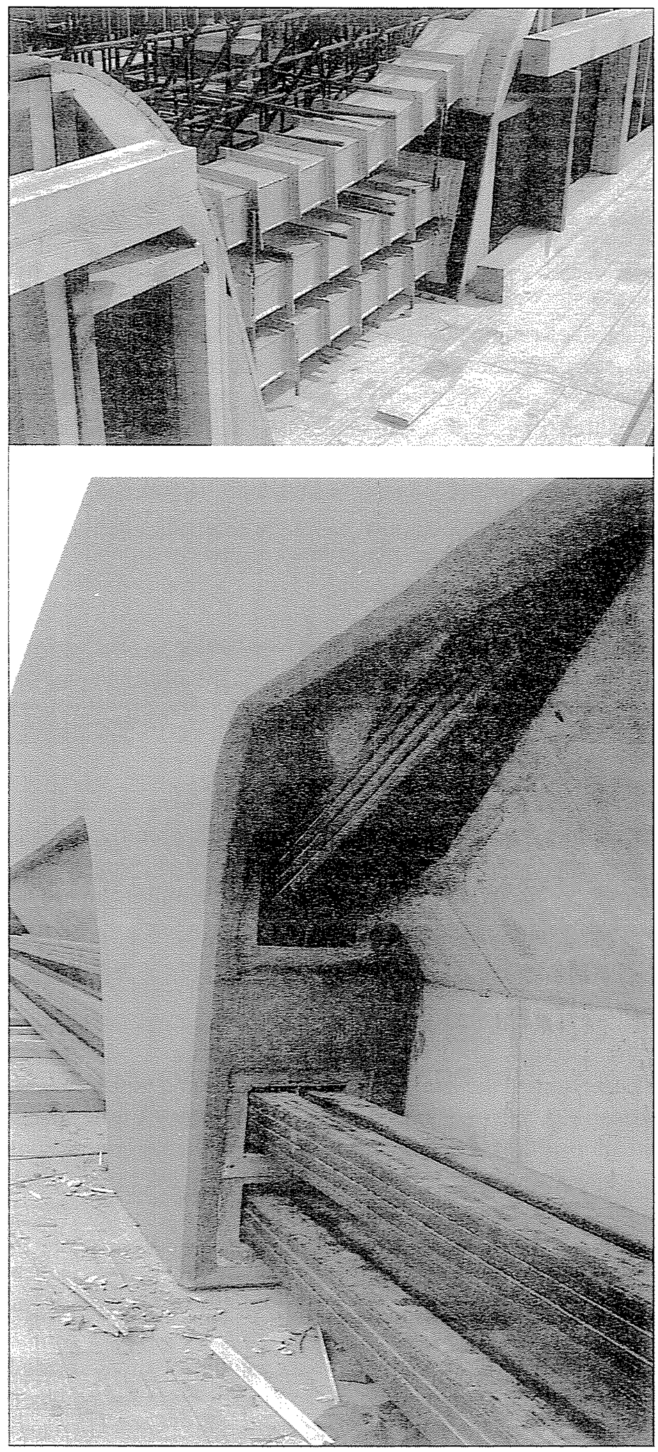


Fig. 9: Force measuring cell – built in

the cables run outside the side plane of the edge beam. They change their direction at deviators in the axes of the transversal girders and progress upwards to the pylon (*Fig. 7* and *8*).

Three times 2 pieces of cables are running in both beams. The cables, led on three different levels, change their direction at three transversal girders. Due to the skew geometry the deviators are located in different transversal girders in case of the two edge beams. We built in force measuring cells at the anchorage of 4 external cables (*Fig. 9*). This enabled us to measure the friction losses during prestressing, which became 3.80 %. Therefore, we considered it sufficient to prestress the 125 m long cables from one side only, naturally changing the active and blind anchorages cable by cable by turns. With time passing the cable force decreases in the sliding cables due to steel relaxation, creep and shrinkage effect. These losses should be checked from time to time. We fixed the acceptable extent of the increase of losses. When reaching this limit value it is necessary to make adjusting of the cables or to change them. The force measuring cells facilitate us to monitor the procedure of cable force decrease and to evaluate this procedure.

3. LOAD TEST

We performed the load test both with static and with dynamic loads (*Fig. 10*). Six loaded trucks stood in one of the spans, weighing 200 kN each, in order to check the deformation of the superstructure. The measured deformations certified the deflection values of the static calculations.

4. PRESENTATION IN SHAPE

Due to the unusual ichnography arrangement of the bridge structure it took significant time to find the harmony between the spatial presentation of the structure and the statical system of the structure. It was the different span ratio of the edge beams at the two sides, and the unusually big skewness that really meant the main problem of the task. The tracing of the extradosed cables, visible from the side, evidently shows the statical system of the structure (*Fig. 11*).

We tried to strengthen this spectacle with the shape of the

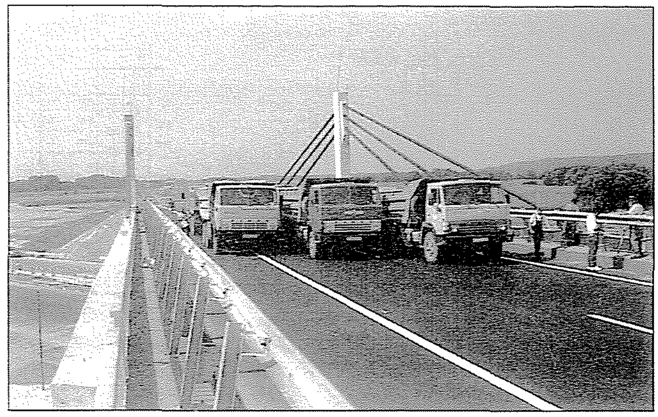


Fig. 10: Static load test

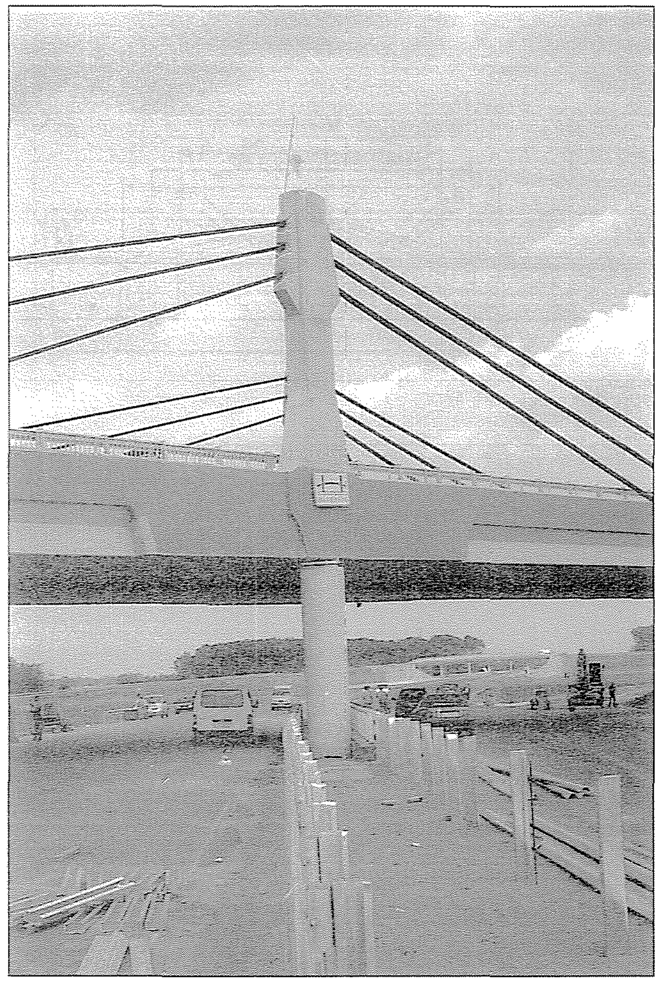


Fig. 11: Pylon with cable passage

Fig. 12: The completed structure

visible deviators. This form appears also on the lower part of the pylon. The quadrangular block of the pylon is eased upwards by the reinforced concrete wall, supporting the deviators, which is narrower, than the prism of the pylon itself *(Fig. 12)*.

5. CONCLUSION

Extradosed bridge technology can be applied successfully in case of constructing slim superstructures. In our case in consequence of the extremely skew ($\alpha < 60°$) supports we faced and had to solve more statical problems than usually. The results of the load test of the bridge proved the accuracy of the calculated deformation values of the structure.

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KŐRÖSHEGY VIADUCT – THE BIGGEST IN SIZE, PRESTRESSED, CONCRETE BRIDGE IN HUNGARY



Péter Wellner – Tamás Mihalek – János Barta

The M7 motorway – leading from the Hungarian capital to the Slovenian and Croatian border, and going beside Lake Balaton in this region – is running among hills at this motorway section. The bridge – unusually in Hungary – is built in a height of 80 m above surface level. Altogether two and a half years time is available for the design and implementation. The bridge is 1870 m long, continuous girder, prestressed concrete structure with more supports, the drawings of which were prepared by us and we have started the construction.

Keywords: motorway bridge, prestressed concrete, balanced free cantilevering

1. INTRODUCTION

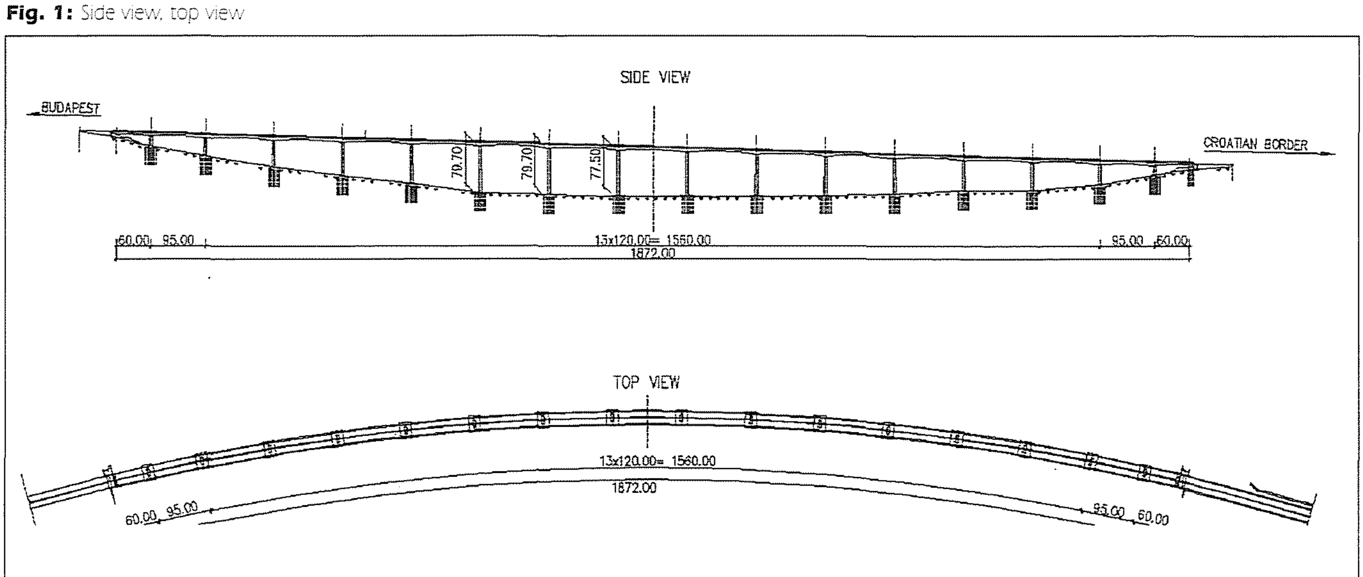
The trace of this motorway section was defined after the elaboration and examination of many alternatives. There were many aspects pro leading this section close to Lake Balaton serving thus the tourism, so important for the region during summer period, and similar number of arguments contra, that is to lead the traffic running towards the Adriatic sea as far as possible from the resort places taking into consideration the recreation aspects as well. The final trace is leading through such kind of section which demands from us a higher level – then usually applied – construction and a viaduct with such a length. The tender, then the order was relating to the construction of a prestressed concrete bridge. Referring to aesthetical reasons the application of spans with 120 m was required. The construction method of the structure is the so-called cast in situ free cantilever technology. This situation demanded from us even the design of the monitoring system – necessary when having multiple spans and building more sections –, of the details of the construction technology, as well as the necessary auxiliary structures simultaneously with the design of the structure itself.

It was an advantage for us that during the passed 15 years we could solve the complex design of significant, modern and huge prestressed reinforced concrete bridges within the company performing the construction itself by using our own taskforce. In case of the Kőröshegy viaduct we prepared the whole scale design of the superstructure such a way. The entire design work of the bridge was shared so that by taking into consideration the extremely short deadline. Pont-TERV Co. participating in the former phases – makes the design of the substructures and of the additional parts as co-designer. This time we discuss about the solution of the superstructure.

2. THE SUPERSTRUCTURE OF THE BRIDGE

The superstructure is built in accordance with the regulations of the technology of the structure being under construction in two directions with free cantilever method. We simultaneously prepare one-one segment in the suspended formwork system in both directions to the already made starting segment above

Fig. 1: Side view, top view



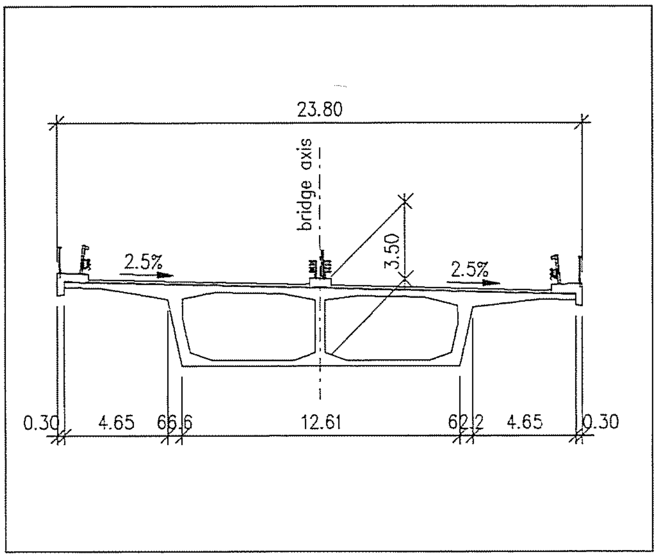


Fig. 2: Cross section of girder in the middle point

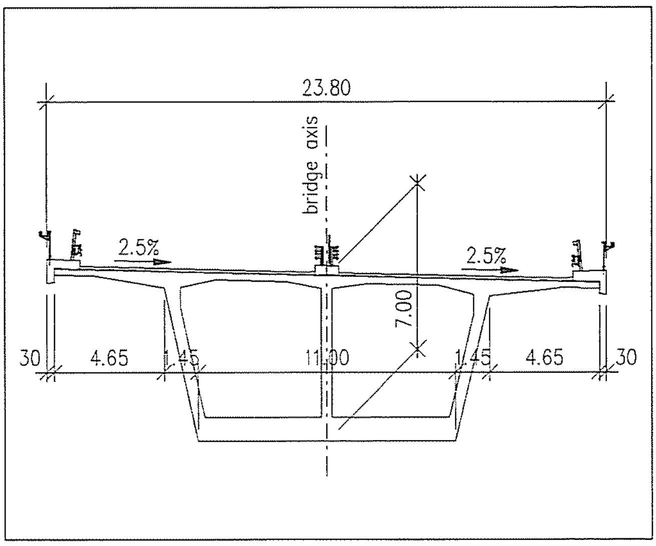


Fig. 3: Cross section of girder above the pier

the pier. These segments are stressed to each other by using cables with tendons led nearby the upper fibre. The construction actually starts in the same time from the direction of the two abutments with a very small difference in time. When the neighbouring bridge sections are ready we close them with the formerly prepared parts. The cables, applied during the construction, made of 19 and 15 tendons, with 150 mm^2 cross section are anchored in *Dywidag* made anchoring heads. We also apply externally led sliding cables in the interior of the box girder. These cables consisting of 19 tendons are usually led through two spans. The cables themselves and their anchorage are *Freyssinet* made. The bearings designed and produced by the company *Maurer Söhne* are suitable, to ensure big motions taking into consideration both the big length of the bridge and the shrinking characteristic features of the concrete. In the interest of solving safely this – unique for us – task the company *Leonhardt Andrä and Partner* performed the independent statical calculations of the structure upon our commission.

The spans of the bridge are *(Fig. 1)*: 60+95+13x120+95+60 m.
The bridge deck has got a 2.68% longitudinal slope.
The bridge is laying in a 4000 m radius curve horizontally.

The structure is 7.0 m deep above the pier *(Figs. 2 and 3)* and 3.5 m deep in the middle of the span. The cross section is

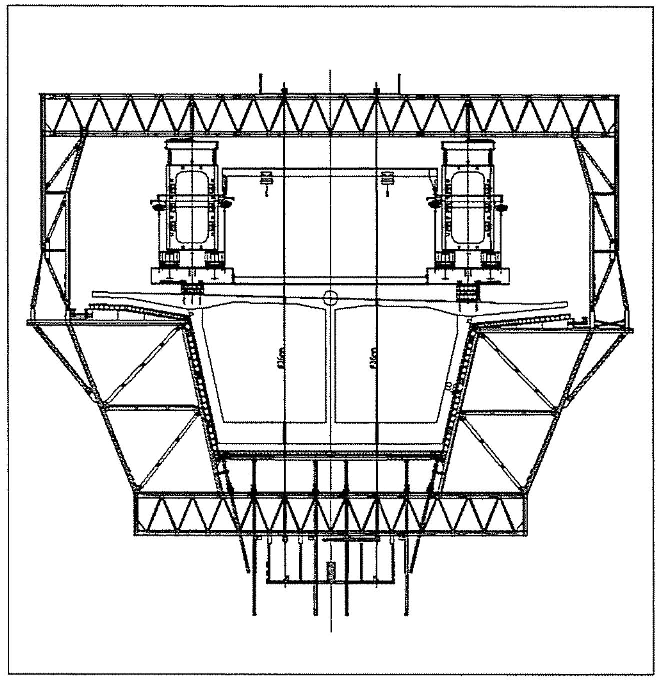


Fig. 4: Cross section of formwork traveller

a structure having two cells and cantilevers in both directions. The width of the deck structure – leading through the entire motorway on one structure – is 23.2 m.

The first segment above the piers is 6 m long. The characteristic length of the further segments is 11.25 m. We use fix bearings on the four middle piers. We place bearings providing only longitudinal motions in one side and bearings providing horizontal motions in both longitudinal and transversal directions in the other side on the rest of the piers.

The planned concrete grade is C 45/55.
The mark of the injected cables is 1770/1570.
The mark of the sliding cables is 1860/1660.
The type of the reinforcement (rebars) is B 500 B.

3. THE METHOD AND STRUCTURES OF THE CONSTRUCTION

In order to apply a cast in situ free cantilever technology the most important thing is to select the means, that are capable to bear the load of the new segment during the concreting as long as we establish the load bearing capacity by the drawn in cables. In this case we have chosen such an auxiliary structure *(Fig. 4)*, which is capable

- to keep/maintain the weight of the formwork system (travelling formwork wagons) of the 11.25 m long and 23.2 m wide reinforced concrete element and of the reinforced concrete itself,
- to move from one section to the other one using its own structures,
- to lift the construction materials from the ground,
- to deliver/supply the construction materials from the already prepared bridge structure to the travelling formwork wagons,
- after finishing one section above the given pier to solve the transfer of the travelling formwork wagons without

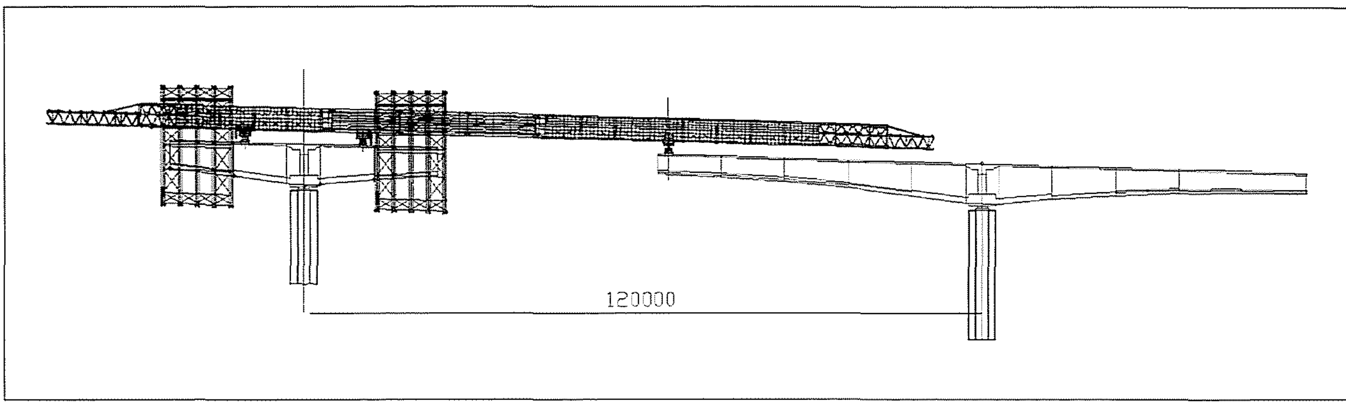


Fig. 5: Longitudinal section of advancing shoring system

disassembling them, placing them to the ground, then assembling them again on the next pier.

We build the certain sections in the following steps *(Fig. 5)*:

- moving forward with the auxiliary structure,
- adjusting the formwork system,
- reinforcement (rebar erection) then concreting of the bottom slab and of the walls,
- prestressing of the cables to be anchored in the walls, after this it is not the travelling formwork wagon any more that bears the load,
- concreting of the deck slab,
- prestressing of the upper cables.

4. ASSURING/MAINTAINING THE SHAPE OF THE BRIDGE, MONITORING SYSTEM

Shrinking, respectively creep is a natural reaction of the concrete based structures with the time passing and it suffers a slow deflection, change in shape due to loading. These phenomena appear continuously as time flies. The bridge structure is under construction during a relatively long time. The parts built in different time are in different phases in a given moment. Therefore their connections must be planned by taking all these factors into consideration. It must be taken into account that a bridge rests on a kind of subsoil. The subsoils in Hungary generally are not very load bearing ones. We also have to count on the sinking of the structure. We wanted to ensure the minimum settlement of the substructures by due stabilisation of the foundation. We are continuously measuring and evaluating the sinking. By the time we can start the construction of the superstructure, we will be aware of the expected – according to our plans minimum – sinking of the substructure.

In order to limit the longitudinal motion of the bridge sections supported movable bearings we anchor the bridge structure back to the abutment cables. On one hand the extent of this anchoring strength/force is given by the horizontal effects, discussed before, and by the effects of the temperature. On the other hand, however, the load bearing capacity of the abutment as such means a limitation. Apart from our calculations we build in a dynamometer to measure the actually operating forces.

The vertical changes in shape of the structure are calculated by the company *Leonhardt Andrä and Partner*, and by evaluating our measurements made at the site, they give the data for adjusting the travelling formwork wagons relating to the next section. We have to apply a high-level geodetic measurement system for the adjustment of the calculated shape, as well as for the measurement of the effects of certain activities. It is necessary to register and evaluate its results continuously.

5. CONCLUSIONS

The application in Hungary of the cast in situ free cantilever method at bridges with such spans took place twenty years ago. The experiences gained at that time could serve only as starting base during the design and construction of the Kőröshegy viaduct. Considering both the sizes and the location of this bridge it is a significant technical challenge.

We thought it practical to use a special technological auxiliary structure, which can minimize the time needed between the constructions after one another of the certain bridge sections. We also expect from this structure to require the shortest possible time for the material supply.

We had to choose such a section length, which serves the shortening of the whole construction time. This is the reason why we chose the 11 m long segments instead of the 5 m long ones usually applied at this technology. Our own design engineers – who have got significant experiences in the construction of the prestressed concrete structures – prepare the construction drawings.

6. REFERENCES

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Péter WELLNER (1933), M. Eng. is Head of Technical Department at Hídépítő. The designing of prestressed reinforced concrete bridges and the associated institutions involved in their technology in Hungary indicates his successful professional background. He received a State Prize for his involvement in the first bridge built using the cantilever mounting method. He also took part in the launching of the method of cantilever concreting in Hungary. The incremental launching technology was initiated in Hungary under his direction. Such structures are now continuously used. He is a member of the Hungarian Group of *fib*.

Tamás MIHALEK (1950), MSc. Structural Eng. He started his designing professionally at Hídépítő and subsequently took part in technological design works beside designing bridges with monolithic superstructures and precast beams. At present he is a leading designer of Hídépítő. In 1988 he took part in the design works of Hungary's first bridge built with the incremental launching technology in Berettyóújfalu. Since 1996 the Technical Department of Hídépítő has been designing the incremental launched bridges (constructed

by the company) under his direction. The main fields of his interest are: the design of prestressed reinforced concrete bridges, the influence of the structural materials and applied building technology on the structures and the consideration of influences on static calculations. He is a member of the Hungarian Group of ***fib***.

János BARTA (1968), MSc. Civil Eng. is design engineer at the Technical Department of Hídépítő Co. After graduating at the Civil Engineering Faculty of the Budapest University of Technology he worked for a statical design company for five years. There he designed (as co-designer) building structures consisting mainly of office buildings and blocks of flats. In 1997 he moved to Hídépítő. There he took part in the design works of the sub- and superstructures of several bridges and a pier structure of the Port of Ploèe, Croatia. Some of the designed bridges had prestressed reinforced concrete superstructures and were constructed with the incremental launching technology such as the Viaducts on the Hungarian-Slovenian railway line. Meanwhile, between 1992 and 1998, he gave lectures in English for foreign students at the Department of Building Materials of the Budapest University of Technology. He is a member of the Hungarian Group of ***fib***.

SYMPOSIUM
KEEP CONCRETE ATTRACTIVE

23-25 May 2005, Budapest, Hungary

21 Sat.	*fib* Council + Steering Committee meetings (only on invitation)
22 Sun.	*fib* Technical Activity Workshop including presentation of *fib* Diplomas for younger engineers (participation is free of charge)
23 Mon.	9:00 Opening Session p.m. Technical Sessions
24 Tue.	a.m. Technical Sessions p.m. Technical Sessions
25 Wed.	a.m. Technical Sessions p.m. Technical Sessions + Closing Session

Concrete is an attractive structural material. Just think on its constructability, unlimited form, economy and aesthetics. The *fib* Symposium "Keep Concrete Attractive" intends to summarize all aspects of its attractiveness.

Abstracts submitted show a very strong interest on the Symposium. Almost 250 Abstracts were submitted from 41 countries of 5 continents (*Algeria, Argentina, Australia, Austria, Belarus, Belgium, Brazil, Canada, China, Croatia, Czech Rep., Egypt, Finland, France, Germany, Greece, Hungary, India, Iran, Italy, Japan, Korea, Lebanon, Netherlands, New Zealand, Pakistan, Poland, Portugal, Romania, Russia, Serbia and Montenegro, Slovakia, Slovenia, Sweden, Switzerland, Taiwan, Thailand, Turkey, UK, Ukraine, USA*).

Following keynote speakers will promote the success of the Symposium:

Topic 1 Attractiveness of concrete structures
Prof. Hugo Corres Peiretti
Prof. Jun Yamazaki
Mr. Jean-Philip Fuzier

Topic 2 Innovative materials and technologies for concrete structures
Prof. Joost Walraven

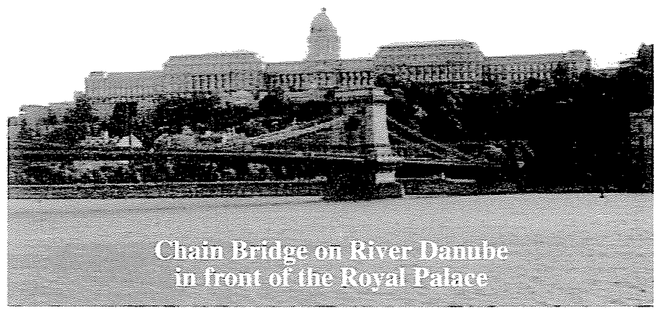
Topic 3 Modelling of structural concrete
Prof. Peter Marti

Topic 4 Sustainable concrete structures
Dr. Steen Rostam

Topic 5 Prefabrication
Mr. Gunnar Rise

Topic 6 Fire design of concrete structures
Mr. Niels Peter Høj

Budapest:
Budapest is the capital of Hungary in the heart of Europe, city which is beautifully situated on both sides of the Danube River has a history dating back over 2000 years. Its main characteristics reflect the atmosphere of the end of the 19th century when the millennium of the Hungarian State was celebrated. As Budapest is a traffic junction in Eastern Central Europe, it can be easily reached by air, train, car or hydrofoil. May is generally pleasant and sunny in Hungary with occasional short periods of rain. Temperature ranges from 15 to 25°C.



View from the Symposium Venue

Symposium Venue:
The Symposium Venue is located in the heart of Budapest, at the Central Building of the *Hungarian Academy of Sciences*, with an amazing view of the Danube river bank and the hills of Buda.
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Social events:
For delegates, younger delegates and accompanying persons interesting and pleasurable events will be organized.

Accompanying Persons' Programme:
Monday: Opening Session + guided visit in Budapest, Tuesday: one day trip to the spectacular Danube bend North of Budapest, Wednesday: visit of the Parliament + Closing Session

Post-Symposium tours:
Post-Symposium tours are offered on the Symposium homepage.

Symposium Secretariat:
"Keep Concrete Attractive" Symposium Secretariat
Hungarian Group of *fib*
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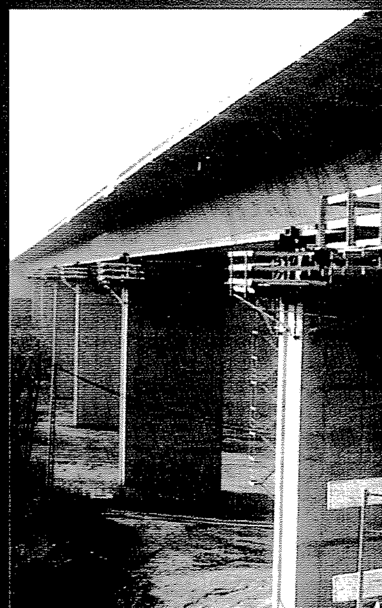

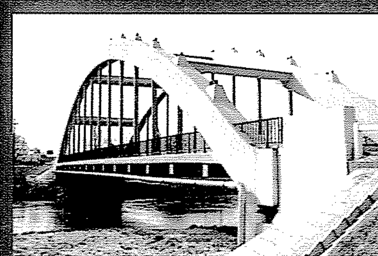

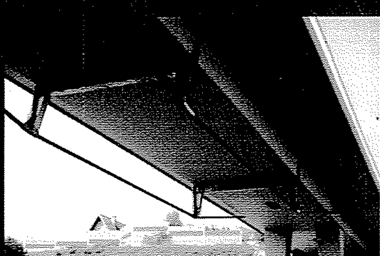

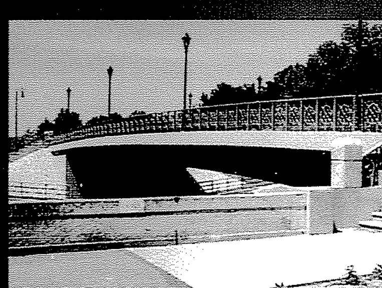

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ÉPÍTŐ IPARI NÍVÓ DÍJ 2000

HÍDÉPÍTŐ

ÉPÍTŐIPARI MESTERDÍJ 1996

DIN EN ISO 9001

HÍDÉPÍTŐ RÉSZVÉNYTÁRSASÁG

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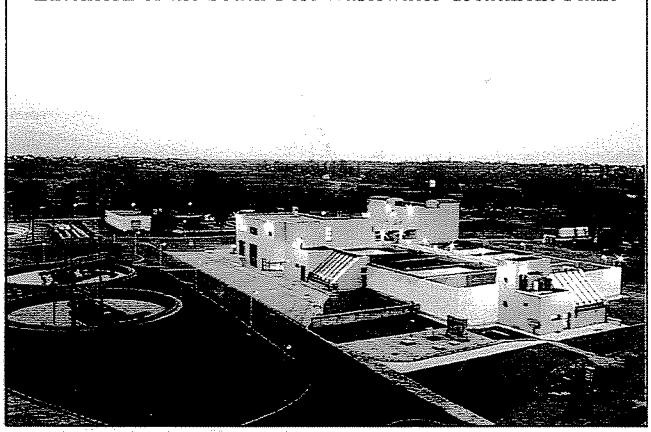
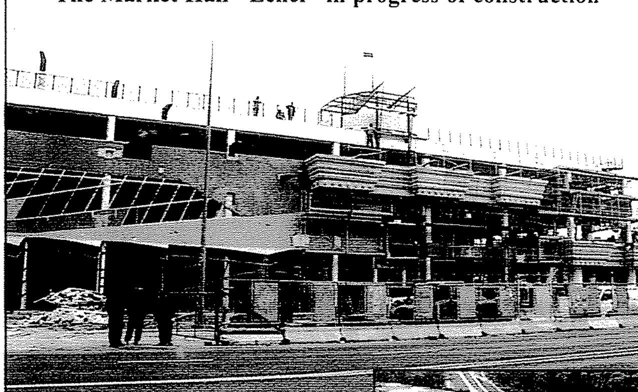
Extension of the South-Pest Wastewater Treatment Plant



The State-owned Hídépítő Company, the professional forerunner of Hídépítő Részvénytársaság was established in year 1949 by nationalising and merging private firms with long professional past. Among the professional predecessors has to be mentioned the distinguished Zsigmondy Rt., that participated, inter alia, in the construction of the Ferenc József (Francis Joseph) bridge which started in year 1894.

The initial purpose of establishing Hídépito Company was to reconstruct the bridges over the rivers Danube and Tisza, destroyed during the Second World War, and this was almost completely achieved.

The Market Hall "Lehel" in progress of construction



The next important epoch of the "Hídépítők (Bridge Builders)" was to introduce and to make general the new construction technologies.

Even among these can be judged to outstanding the bridge construction by balanced cantilever method, the experts having participated in it were awarded the State Prize. By this technology were constructed five bridges in the region of rivers Körösök and this was applied at the flyover of Marx square (today Nyugati square) in Budapest, still the most up-to-date two-level crossing in the capital requiring the minimum maintenance works.

The next big step was the introduction of the so-called cast-in-situ cantilever bridge construction method. This technology was applied for four bridges constructed over great streams, among them can be found the bridge with largest span (120 m) in Hungary made of stressed reinforced concrete, the road bridge over river Tisza at Szolnok.

An important result of the technologic development in the bridge construction was the introduction of the so-called incremental launching method. In the period from year 1989 up to now yet 22 bridges were constructed by this method, mainly on the base of the designs prepared by the Company's own Technical Department.

Among them distinguishes itself the viaduct made of stressed reinforced concrete in length of 1400 m on the Hungarian-Slovenian railway line, near the Slovenian State Border, constructed in one year using the incremental launching method,.

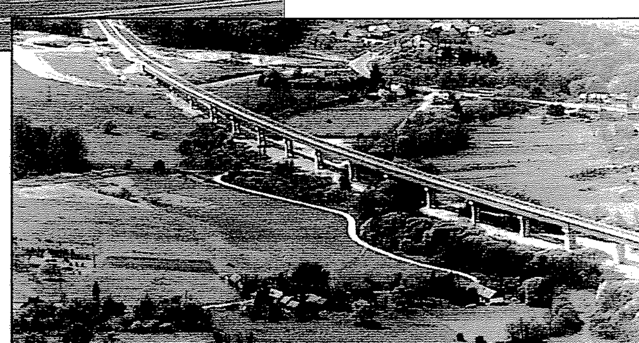
Beside the bridge construction, important results were achieved by the "Hídépítők (Bridge Builders)" in the field of foundation's technological development as well, in the introduction and general use of the bored piles with large diameter, of jet grouting and of CFA (Continuous Flight Auger) pile preparation, further also a new method, subject of patent protection, was developed for very quick and economic constructing bridge piers in living water.

The Company was privatised (bought by the French Company GTMI) in year 1993. Following the multiple merger of the foreign interest parent Company, today Hídépítő Rt. belongs to the multinational Company "VINCI".

In the recent two years a great number of important professional recognitions were awarded the high level activity in the fields of bridge construction and bridge designing.

- High standard Prize of Building Industry for designing and constructing a bridge in length of twice 187 m on the section accessing Budapest of the motorway M5 (2000),
- Innovation Grand Prix for designing and constructing in record-time (one year) viaducts in length of 1400 m and 200 m on the Hungarian-Slovenian railway line at Nagyrákos (2001),
- Prize of Concrete Architecture for designing the viaducts at Nagyrákos (2001).

Nowadays, beside the high level activity in the field of bridge construction, the Company has extended its scope of activity by taking part in winding up the backwardness in infrastructure, construction of drinking water treatment plants, wastewater treatment plants, solid waste spoiling areas and sewers as well as by the introduction of the architectural engineering profile.



The 1400 m long viaduct on the Hungarian-Slovenian railway line at Nagyrákos

By working in good quality the Company makes efforts to inspire the confidence of the Clients. For this purpose have been introduced and operated the Quality Assurance and Environment Controlling Systems meeting the requirements of the international Standards ISO 9001:1994 and ISO 14001:1997, justified by international certificates.

The Company is hopefully awaiting the new tasks in order to enhance the reputation of the "Hídépítők (Bridge Builders)".