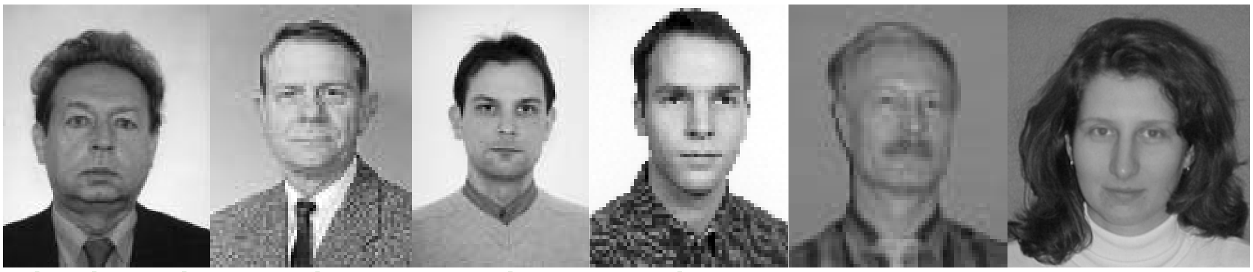
Concrete Structures 2003

DESIGN PROCESS OF THE LOAD BEARING STRUCTURES OF ASIA CENTER



László Polgár | Dr. József Almási | Balázs Karkiss | Péter Varvasovszky | Dr. Csaba Pethõ | Erika Tatai

One of the biggest European structures of the turn of the century is being erected in international cooperation in Budapest. The project is a pioneering example of globalisation. Despite all initial difficulties, there is no doubt that this is the way to go. The scale of the project is best described by the floor spaces of the first and the second phase: 120,000 sq metre and 88,000 sq metre, respectively. The characteristic column grid of 8 m × 16 m, the constant and variable load of 20-25 kN/sq metre beyond the own weight of the load-bearing structure and the approved 71 cm overall depth posed a tough task for the structural designers.

Keywords: ***structural design, Eurocode, pre-fabrication***

1. INTRODUCTION

The ASIA Center, located in the 15th District of Budapest will be one of the biggest structures in Central-Europe. Alone the floor space of approximately 210,000 sq metres is indicative of the scale of the building. The Chinese investor, the application of the principles of Feng Shui, the main contractor (Strabag) and the Austrian-Hungarian designer team make the project all the more interesting. Planning started at the end of 1999, the earth- and foundation works commenced in February 2000 *(Fig. 1.)*.

Fig. 1 A bird's eye view of the construction site

The whole structure was completed by the spring/summer of 2003. The reinforced concrete works alone comprise such a serious task that they deserve to be given an account of in *concrete structures* The process of construction could be followed on a daily basis by means of the photographs published at internet.

2. BIRTH OF THE STRUCTURAL CONCEPT, PRELIMINARYSTUDIES

The Austrian Lackner & Raml Ltd. searching for a Hungarian structural engineering designer

contacted Plan 31 Mérnök Ltd. upon the proposal of Strabag in 1999.
The starting conditions of designing the load-bearing structure of the multi-storey building with a characteristic column grid of 8 m × 16 m were as follows:
l The need for floors with the smallest possible structural depth and of an acceptable price. Considering the scale of the building, it was essential that the entire HVA and services system should be installed under the load-bearing structures so that they do not obstruct the design and the execution of the project.
l The need for a structural concept that facilitates the quick execution, possibly using prefabricated reinforced concrete elements.
At the beginning of the design process we could, to some extent, rely on the Árkád department store whose construction was already going on at that time (another main contract of Strabag), but in that case the designers elected to guide the HVA and services pipelines through the floor beams and thereby to employ a larger vertical clearance (the Árkád department store has a column grid of 10 m × 16.5 m and a structural depth of 1.6 m).
In the case of the ASIA Center we had to effect some serious compromise to achieve a structural depth of 71 cm, which, considering the column grid of 8 m × 16 m (1/22.5) is a very good performance.
Another option could have been an entirely monolithic reinforced concrete structure prestressed by sliding cables, blich was not advantagbews the highly complicated floor plans and the dimensions of the structure.
On the other hand, the concept of a fully prefabricated concrete structure had to be rejected too, because the low vertical clearance made it necessary to employ a monolithic concept.
In the case of the *Lurdy Department Store* and the *Interspar Pesterzsébet Hypermarket* we collected some favourable experience with the simultaneous application of very wide, low depth main beams, pre-stressed double-T floor elements and monolithic top concrete. In the case of these structures we could use prefabricated passing columns, stressed and prefabricated beams and floor elements.
In the case of ASIA Center an optimal solution could be achieved by means of a *monolithic column* + *monolithic beam* (8 m span) + *prefabricated, prestressed concrete T panels* (with a mass limit of 6 tons imposed by the capacity of the tower cranes) + *monolithic top concrete* concept.
Suspending the ends of the T panels could have made the main beams' formwork and reinforcement somewhat less complicated (for suspended beam-ends see Szalay, 1988). After detailed analyses we decided to use the "traditional" method, however: cantilever beams with straight half joint like butts.
After the analysis of the structural concepts and the decision-making process execution planning could commence in February 2001.
April-May 2001 was an especially important period in the structural design of the project. Let us quote from the evaluation written by Mr. Raml at the time:
"As far as the evaluation of the structural concept of the above building is concerned, the most important criteria are as follows:

SOPHISTICATED FLOOR PLANS

The plans supplied by Lengger Architects fulfil the client's needs (Feng Shui spirituality). The arched outlines and the oblique systems of axes result in a structure that is entirely different from other shopping centres. The basic column grid agreed upon with the client is 8 m × 16 m which lends itself for both a monolithic or prefabricated reinforced concrete structure.

BASIC TECHNICAL CONDITIONS

The number of floors planned and the floor heights prevent the use of full-height, prefabricated columns. Theoretically it would be possible to partly prefabricate vertical elements, such as walls, staircases, elevator shafts, columns and combine them with monolithic reinforced concrete.
It would be desirable to build the horizontal structural element, i.e. the floor monolithically. This would enable the construction company to adapt to the floor plans flexibly and to lengthen the prefabricated structural elements floor by floor.
If the floors were constructed entirely from prefabricated floor elements, then a significant number of special elements should be created which would make the lengthening of columns a

lot more difficult.
The biggest problem of the construction of the monolithic slab floor is how to limit deflection. This could be achieved by using large expanded column heads sized in proportion to the large grid of 16 m × 8 m. Deflection could further be decreased by incorporating pre-stressed concrete shuttering panels.

FLEXIBILITY OF USE

During the initial period of planning (1 month before the submission deadline of the building permit plans) the final use of a large number of rooms was not clear. Only the deep-level garage and the spaces related to catering seemed to have taken their final shape.
This meant that the structure to be erected had to be suitable for a flexible use and division of floor bays. Internal partition walls had to be connected to the other load-bearing elements of the structure and there had to be an opportunity for subsequent cutting through the floor for services. Experience has shown that in the case of TT floor element the necessity of subsequent cutting in the ribs must also be reckoned with (Fig. 2.).
INCREASED DEMANDS CONCERNING HVA AND SERVICES
Beyond the partly low structural depth a significant amount of pipelines and cable networks must be accommodated.
In the case of a prefabricated floor structure, which could
best be constructed from reinforced or pre-stressed concrete TT floor elements, there would be a need for a high number of rib cuttings. Mainly due to the large penetrations needed for the voluminous ventilation ducts the double-T panels can not be used and maybe some new columns would have to be added to the structure.
If it was possible to construct a monolithic floor, then only the vertical connecting shafts would have to be specified exactly, the horizontal pipelines could be installed without limitations.

CONTSTRUCTION TIME

During the preliminary discussions we inquired about the available TT floor element manufacturing capacities of the Hungarian prefabricating plants. Based on the construction experience of a project of a similar size (Lurdy Ház Department Store) 5-6 TT elements per production line seemed feasible. With 3 plants this would mean approximately 15 elements a day. This coincides with our findings in Austria and Germany.
This would mean that for the first construction phase comprising approx. 115,000 sq. metres and requiring 2875 TT panels (40 sq. metre per piece) 190 days would be needed, let alone the manufacturing difficulties of the many different elements.
If three plants were commissioned with the production, another drawback would be that there would be no way to have them compete with each other's price quotations.
If we insisted on prefabrication of vertical structural elements, then we could exploit the capacities of more than one prefabrication plant; in the case of a monolithic slab floor several companies could be invited to increase the speed of construction.
Construction could go on at more than one locations simultaneously without being exposed to the capacity constraints of the manufacturing plants.

COSTS

At the state of the design process in 2002 it was very difficult to estimate the costs considering all aspects referred to above. Below is a simple comparison of the costs of the two different construction systems:
Prefabricated TT panels in the 16 m span direction, prefabricated main beams in the 8 m span direction, approx. 0.425 m slab substitute thickness, approx. HUF 30,000/sq metre.
Monolithic slab with point support, approx. 0.55 m slab substitute thickness (slab+column head), approx. HUF 27,500/sq metre.
With all due reservations regarding such rough estimates it can be declared that there are no obvious differences between the two systems. On the other hand, the installation difficulties of HVA and services are a very important aspect.

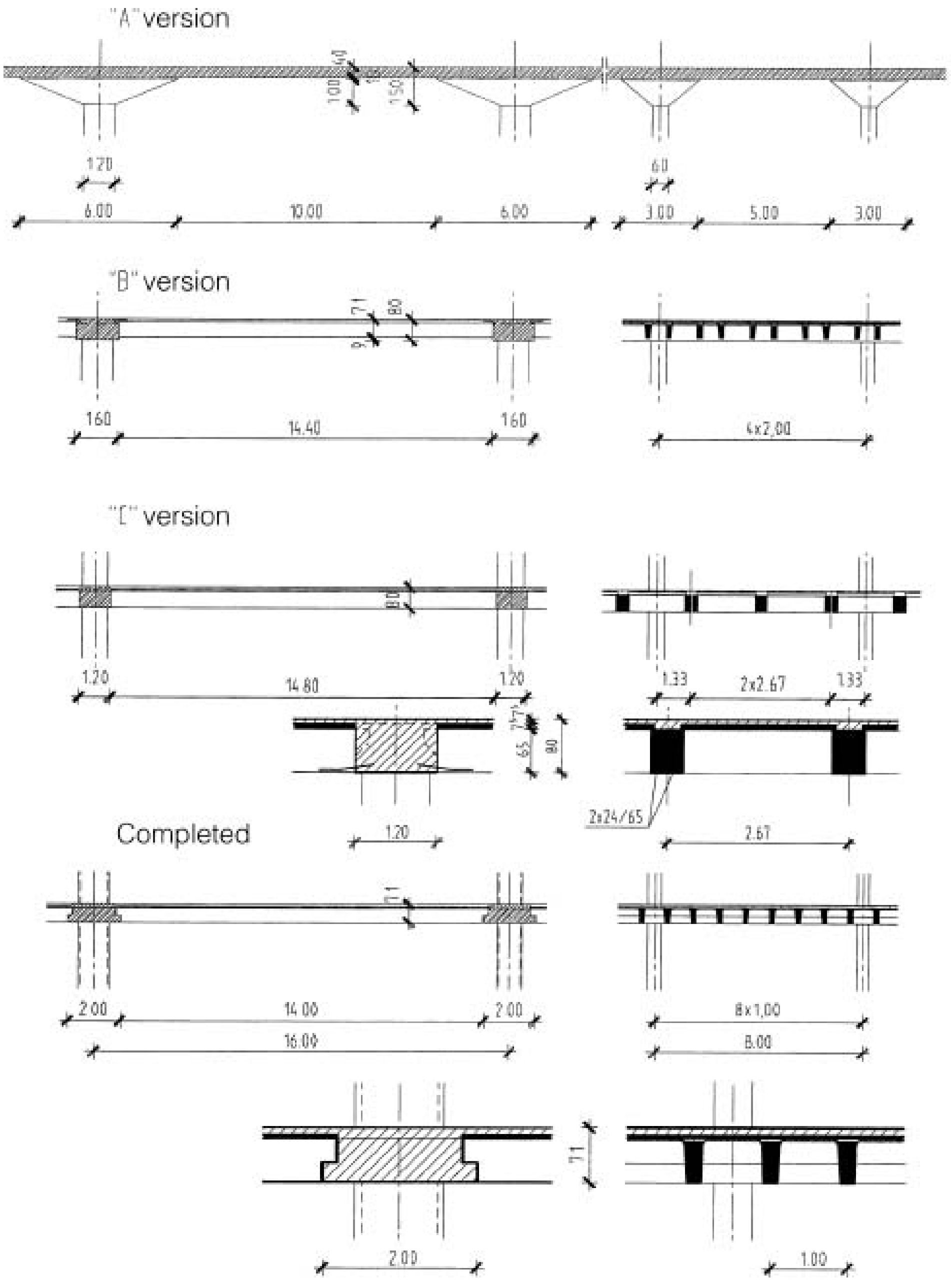


Fig. 2 Various structural elements

PROPOSAL

It is fairly obvious that the most expedient and economical solution would be to combine prefabricated and monolithic elements. The majority of floors could be made of modular floor panels (shuttering panels) or a shuttering system that can be relocated easily. The necessary column lengthenings do not pose serious difficulties. In the case of a monolithic floor slab the floor penetrations and floor edges etc. are not problematic either."

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There were quite a number of such analyses made at that time, both in Austria and Hungary. Back then there were sig
nificant differences between the ideas of Lackner & Raml and PLAN 31 Mérnök Ltd. concerning the structural concept of the building. This resulted mainly from the different building traditions and from the different price structures (these issues will be familiar for anyone who has ever worked with foreign designers). At the time the Austrian partner was evidently reluctant to employ prestressed concrete floor panels. As it turned out during the design meetings this was due to the fact that the specifications of ÖNORM and DIN for prestressed concrete lagged behind modern structural design principles (they were
based on 40-year-old know-how), while our MSZ and especially EC2 are based on crack control. Therefore we have been free in determining the level of prestressing and setting deflection and hogging by varying the ratio between the quantities of the prestressing strands and the normal steel reinforcing bars for at least two decades (and of course we do not determine the concrete cross section on the basis of Magnel's straight line, either).
The use of EC2 - we had agreed upon this at the beginning of the design process, as no international design project could afford not to use it - and the calculations made by the `abacus' software made it highly convincing that using T and double-T panels is the way to go.
In the meantime, Strabag Ltd. had been working on the construction technology. They decided that the maximum mass of any one element must not exceed 6 tons. In the case of a multi-storey building of such a large ground plan area, the placing of the tower cranes is also a factor of prime importance.
It became reasonable to sub-divide the 8 m span into 8 × 1 m. Thus, the optimum length of T-elements turned out to be 14m, which meant the final width of the monolithic beams was 2 m.
Also considering the aspects of HVA the final maximum structural depth was 71 cm (this had to be increased in exceptional cases only, e. g. where the Feng Shui principles required that columns are erected on top of certain beams).
As it was foreseeable, after the basic principles and the structure were finalised, several non-conforming methods had to be employed as well.
The column grid of 8 m × 16 m was at some places modified to 8 m × 8 m. In the floor bays with a span of 8 m the main contractor required that a floor comprising shuttering panels should be constructed. In these bays there will be 8 cm thick floor elements (shuttering panels) + 22 cm monolith reinforced concrete slabs with a span of 6 m between the 2 m wide beams.

The stiffening cores, the staircases and the elevator shafts formed another set of issues where Hungarian and Austrian ideas diverged. In Hungary it is becoming the standard to build such projects with a sliding formwork *(Fig. 3.)*, while in Austria large panel formworks are used for the stiffening cores. Here again, the differences are the result of the diverging cost aspects and work traditions of the two countries.

One big advantage of using sliding formworks is that work-intensive elements are manufactured at the beginning of the construction, and if the flights of stairs are installed at a relatively early stage, then pedestrian traffic is made significantly easier for the rest of the construction time and workers can communicate within the safety of the cores *(Fig. 3.)*.

What is a great advantage during construction, however, can make the design process a lot more complicated. The sliding formworks of the cores quickly reach the top floors, so the joints of all floors must be defined at the beginning of the design process. This, considering the complicated character of the building, posed extreme difficulties in the beginning: monolithic beams, T-panels, floor elements had to be connected to each other, internal staircases and stairheads had to
be created. However, all these efforts paid dividends at during the construction phase.

Fig. 3 An elevator shaft constructed with sliding formwork

3. DIVISION AND ORGANISATIONOF STRUCTURAL DESIGN TASKS

An international design team can only handle the overall design of such a large project involving a foreign client and a foreign main contractor (Strabag International). Lengger Architects (Villach, Austria) coordinate the design of the whole structure. The Hungarian subsidiary of Lengger Architects is Makat Ltd. We are getting used to such design projects without frontiers by now.

Structural design is coordinated by the Austrian Lackner & Raml Ltd. (Villach). Their Hungarian partners are Uvaterv Co (foundation, watertight basin), Caec Statikus Iroda Ltd. (monolithic columns, walls, beams) and Plan 31 Mérnök Ltd. (floors, staircase cores, elevator shafts). Participating Hungarian companies employed further sub-designers.

Coordination between the designing architect and the various participants and the preparation of structural layouts, formwork plans were (and are) the task of Lackner & Raml Architects and involved continuous feedback from their Hungarian partners.

With such large projects, coordination of the teamwork is a huge task in itself. All participants must follow a strict order of positioning, arrangement and documentation.

All statical calculations were made by the participants themselves for their relevant parts of the structure. Dimensioning was based on Eurocode 2, or rather on ENV 1992-1-1 and ENV 1992-1-3, to be precise.

All data supplies were based on reference load values to prevent any errors. It would have been ambiguous to specify calculated values (which is the EC2 equivalent of "critical load" of the MSZ, the Hungarian Standards). For such international design projects the common international language could only be EC2 (and the related EN 206 etc.).

Naturally, all plans were drawn with CAD methods and the Internet played a key role in the flow of information. All plans were delivered on 1 CD ROM and in 3 printed copies in strictly specified (.dwg and .plt) formats. Plans finalised for execution were also published in an extranet for internal use only, so all designers could see on their computers the plans made by other designers. The order of documentation is illustrated by the compulsory "blueprint stamp" to be placed on each and every plan.

The structural design documentation of the "male" and "female" building parts constructed in the first phase (approx. 120,000 sq. metres) consists of some 4000 plan sheets (the design process was completed in December 2001) - this illustrates the absolute importance of a design order.

Below is a summary written by Uvaterv Co. and Caec Ltd. of their respective tasks related to the project.

4. DESIGN TASKS OF UVATERV: FOUNDATION WORKS AND THE WATERTIGHT BASIN

4.1 General description of the foundation

The building is supported by a slab foundation combined with piles. The slab with a thickness of 80 cm (its thickness is
increased at the piles) and the perimeter walls are watertight up to the critical groundwater level. Uvaterv Co. was awarded a contract to design these structures.
4.2 Soil and groundwater
The original site was almost plain, the average ground level was 119-120 m above Baltic Sea. Under the humous surface layer there are very deep fluvial sediments from the Pleistocene and Holocene periods; the sediments are mainly sand soils whose coarseness increases with the depth. Between the granular strata there are lenticular deposits of thin transient and bound strata (silt and muddy rock-flour). Thewater permeability of the grainy measures varies between k = 10^{-3} - 10^{-6} m/s. The surface of the Miocene substratum varies between 101.1-106.0 m above Baltic Sea. Clay and silt soils have good watertight qualities (k = 10^{-8}-10^{-9} m/s). The critical groundwater level is 117.0 m above Baltic Sea. The expected construction water level is 114.0-115.0 m above Baltic Sea.

4.3 Dewatering

The architectural and structural design of the building commenced in September 2000 and February 2001, respectively. The construction of the large underground floors below the groundwater level posed serious difficulties. The underground parking floors and the technical rooms required a building pit with an approximate depth of 11.0 m and we had to be prepared for about 4 m groundwater level difference during construction. For the evacuation of water from the building pit we had to choose the most suitable from a number of different methods. Open channel dewatering is mostly used up to a depth of 2-3 metres; to overcome a depth beyond that requires active interventions. In the case of sump dewatering there is no need to build a separate structure but the high quantity of water removed (15,000 cu. metres/day) would have posed unsurmountable difficulties.
The most reliable method for the evacuation of water from the enormous building pit seemed to be a wall encircling the entire construction site, even though this is costly and time consuming. Finally, the building pit was constructed with a pulp wall enclosure connected into the silt-clay stratum. This structure was designed by Taupe Ltd. and is cheaper than a cutoff wall. As it can be partially removed after it is not needed anymore, it does not inhibit the flow of groundwater. A cutoff wall would not have been suitable anyway because of the varied outline of the building and of the ramps leading to the underground parking floors.
28 sumps were installed for the dewatering of the construction site which facilitated a quick reduction of groundwater level. For the removal of the small amounts of water seeping into the building pit a few sumps are operated intermittently.

4.4 Earthwork

Excavation was performed in three main phases; in the first one, soil was removed to 116.0-117.0 m above Baltic Sea level. This is where the pulp wall encircling the construction site starts. Here, a set-off was made and excavation continued down to the piling level. After the piling work was completed, the subgrade level of the foundation slab was prepared. The deepest level of the final earthwork is 111.03 m above Baltic Sea.
The main contractor (Strabag Co.) required that the sandy gravel bedding layer should be omitted. Even though the subgrade level comprised grainy soils, this was not possible. The coefficient of irregularity of the mainly fine-grained sand found here is low (U = 2.2-2.8). These soils dodge any concentrated loads (e.g. the wheel load of construction machinery), which means that the earthwork needed for the construction of concrete subbase can not be created. Therefore, the bedding layer could not be omitted for reasons of constructability. A layer of 24 cm sandy gravel (Trr = 95%) was placed under the 6 cm thick blinding concrete

layer.

4.5 Piling work

For the foundation of the building SOB piles were manufactured. The load bearing capacity of piles was determined arithmetically and by means of loading tests.
The calculation method employed (Berezantsev) adopts a three-dimensional sliding surface under the tip of the pile; dislocation on this surface is inhibited by the stress region formed around the envelope. The weight of the region acting as a lateral load must be reduced by the friction on the boundary surface of the region. The resulting pile load bearing capacities were used for the preparation of load tests.
During the load tests it posed a problem that the anchoring piles did not have enough reinforcement in them, which meant that the piles could not be loaded up to the breaking point (the anchor steels broke, so this was actually an anchor steel test). The test results were still usable because we defined the load bearing capacity of the piles on the basis of a force belonging to a ~10 mm limit dislocation. The limit load bearing capacity of a 15.0 metre long pile with a diameter of 90 and 60 cm is F_H= 2800 kN and F_H = 1800 kN, respectively.
Piling work was performed by BRK Speciális Mélyépítõ Ltd. and HBM Ltd. There were 837 and 335 piles manufactured with diameters of 90 cm and 60 cm, respectively. There is no structure above the loading areas, therefore it was necessary to use anchoring piles here to prevent the levitation of the foundation slab.

4.6 The foundation slab

The spring constant of the piles was calculated from the ultimate load-bearing capacity of the pile and the associated dislocation. The foundation slab was calculated as a slab embedded elastically, with the springs being more rigid at the pile locations. Individual piles and groups of 2-5 piles were used to accommodate the varying pile loads (max. 18,000 kN) of the first phase. According to our calculations the foundation slab could - without the piles and with reinforcement - accommodate column loads of 3500 kN.
The foundation slab was constructed in accordance with the requirements of the Austrian standard. The standard, which refers to the critical part of the building below the groundwater level as the "weisse Wanne" (the white basin), groups the various structures depending on the height of the water column and the dryness requirements. Then it specifies the cracking limits of the individual classes, the sizes of concrete slabs and walls that can be manufactured in one stage, the minimum amount of reinforcement and the joint arrangement should be made.
The base reinforcement of the 80 cm thick foundation slab had a diameter of 20/20 mm. At column loads exceeding 6000
kN, apart from the piles it was necessary to make the foundation slab thicker. The thickness was 1.04 metre and 1.20 metre at the 2- and 3-5-pile groups, respectively, with a splay of 45°. To accommodate the increased bending moment a reinforcement with a diameter of 25/10/20 mm was used, the cracking limit (0.15 mm) could be achieved with a mesh of 10/15/15 mm diameter. For the purpose of punching reinforcement we used hoops with a diameter of 14 mm and 16 mm.
The foundation slab of the building has no expansion joints; it is separated with displacement joints from the loading areas and the ramps only (the slab thickness is a mere 50
cm here). There are two expansion joints in the structure of the underground floors (along axes 11-12 and 17-18). The foundation slabs of the first and the second phase have an expansion joint between them.
At the -2nd underground floor there will be parking spaces, technical and storage rooms. Because of the different functions of the various areas the foundation slab has different levels as required by the architectural concept. The characteristic foundation slab top levels of the first phase are as follows: ground level: 120.00 m above Baltic Sea (-2.50); parking level: 114.14 m above Baltic Sea (-8.36); storage: 116.45
m above Baltic Sea (-6.05); service tunnel: 112.13 m above Baltic Sea (-10.37); lorry goods distribution: 113.9 m above Baltic Sea (-8.60). Naturally, mechanical shafts were also built.
The concrete technological report specified the maximum size of the largest slab section to be poured in one stage at 24.0 metre × 24.0 metre. The average section size is 16 metre× × 24 metre. There were 160 and 87 slab sections poured in the first and the second phase,

respectively. Between the individual slab sections watertight joints had to be created, so the construction joints were filled with expansive rubber and the perimeter was sealed with joint band. At the sinkings and the abutment of the slab sections with different levels it was often not easy to place the joint band. Perimeter walls have a thickness of 40-50 cm, the maximum length to be poured at one stage was 8.0 metres.
One of the difficulties related to the construction of large foundation slabs is the prevention of shrinkage cracks. Even though all provisions of the concrete technology were adhered to, in some sections of the foundation slab 0.1-0.2 mm wide cracks formed, which exceeded the maximum allowable limit. The slab sections most affected were poured during the baking hot periods of the summer and the temperature was above the ideal. In the final form of the building some of these cracks will disappear as the slab will bend under the load. At the present stage of the construction it would not be practicable to fill these cracks. Remedying is to be commenced under favourable weather conditions and after the superstructure has reached 50 per cent completion.
Piling and the dimensioning of the foundation slab were not very complicated tasks. The problems mainly resulted from the following factors:
- because of the protracted decision preparation process the planning and construction times were very short
- the large building is difficult to embrace *(Fig. 4)*
- architectural plans were often modified during the preparation of the reinforcement plans
- coordination of the different structural engineering teams
- design was only a few steps ahead of construction
- all plans had to be prepared in .dwg (AutoCad) format.

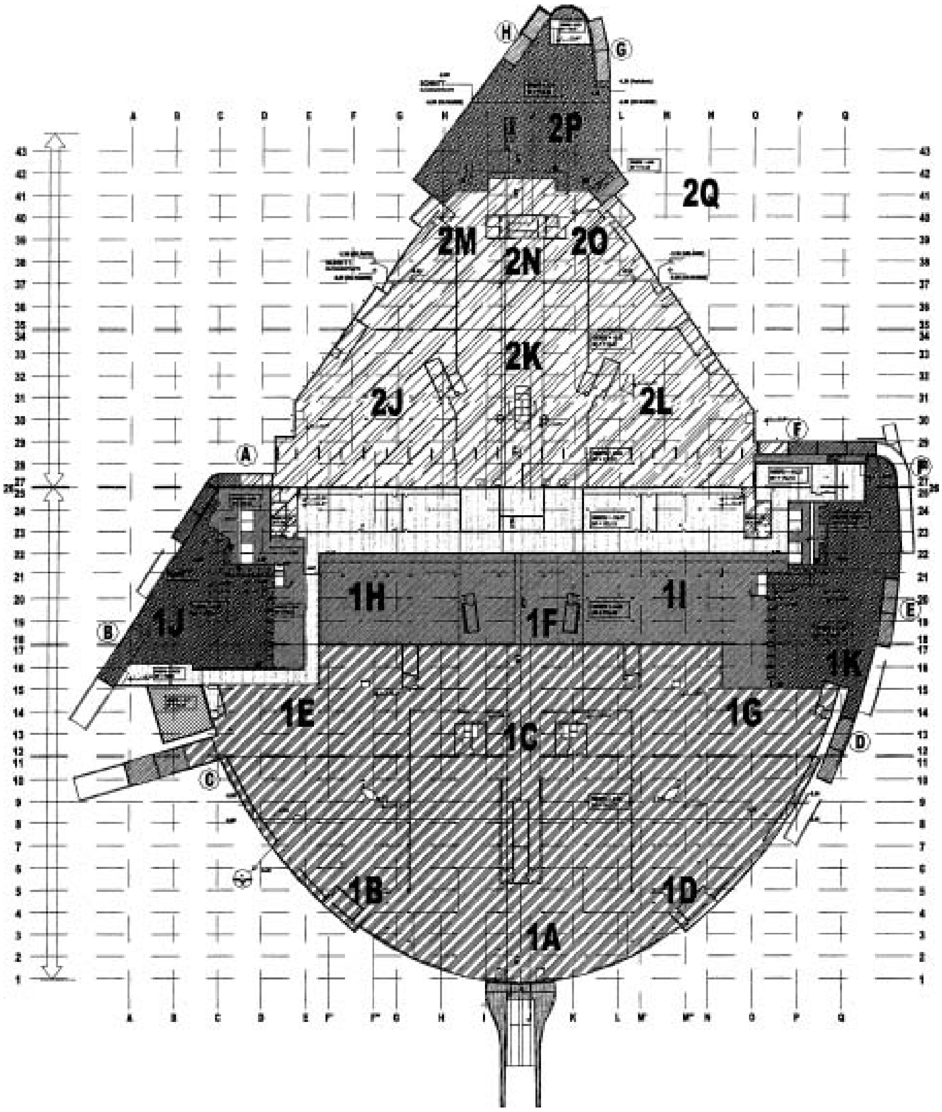

Fig. 4 Ground plan of the building

5. DESIGN TASKS OF CAEC Ltd.: MONOLITHIC COLUMNS AND BEAMS

The office joined the execution design process at a comparatively late stage, in March 2001. In May the first columns of section 1D were already manufactured on the basis of our plans and were followed a few days later by the first reinforced concrete walls and beams. This means that there was only little time left strictly for planning and plotting.
As a result of the lengthy preparations we had a mature load bearing structure concept, which meant that Plan 31 Mérnök Ltd. relieved us of the often time-consuming and tiresome task of coordination with the client.
First we calculated the reaction forces communicated by the superstructure to the foundation, so that Uvaterv Co. could calculate the loads the foundation slab would be exposed to.

5.1 Walls and diaphragms

The individual units of the building between expansion joints
had a sufficient number of staircase and elevator shaft cores necessary for the spatial stiffness - this had been the design task of Plan 31 Mérnök Ltd. The monolithic reinforced concrete walls made with panel shuttering were mainly fire impeding walls at the underground floors and external perimeter walls of the superstructure, whose primary architectural function was to divide the space and had no significant role as load carrying structures.
Certain walls, however, played an important role as part of the load bearing structure, of which the building part extending in a cantilevered manner more than 8 metres long above the lorry ramp in section 1G and 1E is an exciting example.
During the design of the walls we were unfortunately often faced with the fact that at this early stage of the construction the formwork plans did not indicate all the necessary penetrations, and what they did indicate was often modified at a later time. As a result, several plans had to be amended subsequently.

5.2 Columns

In order to expedite the execution and design processes we developed a modular reinforcement system for the columns and beams which had the following advantages:
- *From construction-related aspects:* It enabled us to pre-assemble the majority of the reinforcement which only had to be lifted into place upon delivery. Thereby the time consuming task of steel fixing could partly be "outsourced" from the construction site and arranged at outside locations with larger capacities to expedite the works.
- *From design-related aspects:* The results of the dynamic calculation could be evaluated and prepared for design purposes using the load-bearing levels calculated on the basis of the reinforcement modules. Design could be accelerated by the creation of a computerised modular file system. We employed the "block" features of the AutoCad software. This way the designer had to assemble the plans from the modules specified on the basis of the design analysis (statics) and to supplement the modules with the specific features resulting from the actual location of the module in question.

The general column diameter in the underground levels was 116/50 cm, the columns of the upper levels had a circular cross-section and a diameter of 70 cm or 60 cm.
The reinforcement of the columns was designed under consideration of various aspects in a strict system. Such aspects were: easy assembly, good compaction of concrete, minimum amount and economical use of steel and that the reinforcement of the beams should be able to pass above the columns.
We had prepared detailed plans of the reinforcement joints already at an early stage of the design process in order to coordinate the reinforcement systems of the individual supporting structures. When planning the reinforcement system, every bar had its exact location specified to the centimetre. We defined the ultimate optimum location of every longitudinal bar and reduced the number and/or diameter of the reinforcing bars of columns exposed to smaller loads.

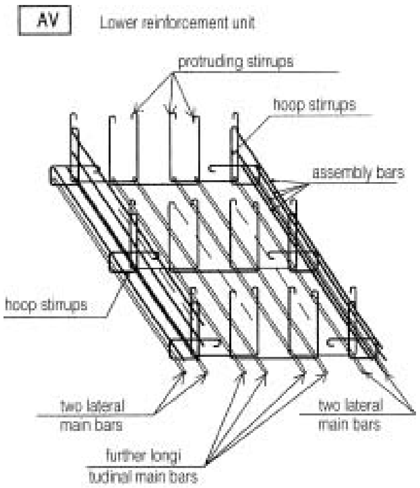

Fig. 5 Scheme of a beam reinforcement assembly

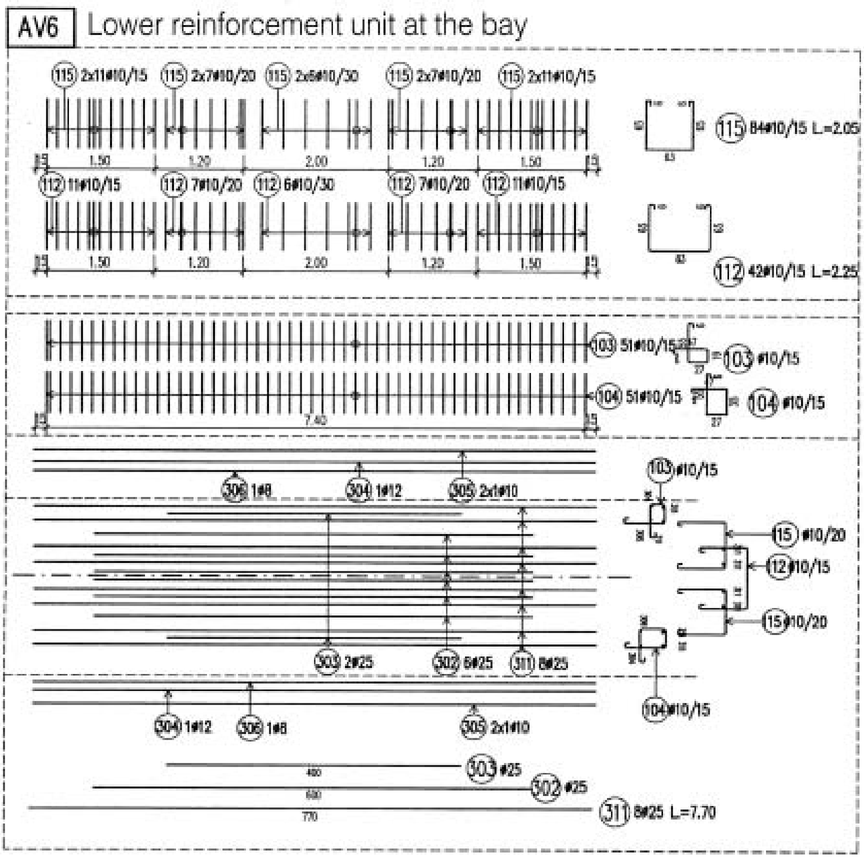

Fig. 6 The lower reinforcement unit

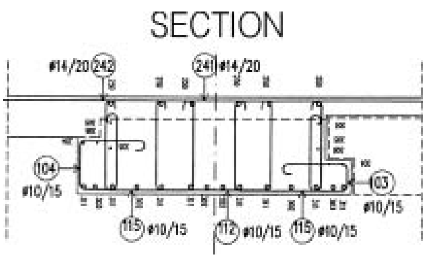

Fig. 7 Beam cross-section within bays

5.3 Beams *(Figs. 5, 6 and 7)*

With the exception of certain 14 metre long perimeter beams at the end of the floor sections

all beams were made of cast in
situ concrete. The beams engaged the majority of our design capacities because a large number of often very complicated beams had to be designed. Not even the most common main beams could be considered as conventional with their width of 200 (162) cm and the depth of 71 cm.
One of the interesting features of the load-bearing structure of the building is that - mainly because of the functional differences between the -1st floor and the groundfloor - the loads of walls and columns carrying the loads of more than one levels had to be discharged to columns located at different locations of the floor-plan, which also posed difficulties during the design of beams.
The reinforcement module system was used for the main beams running along the grid axes. The entire reinforcement of the beam was divided into four reinforcement modules between the supports: lower reinforcement module (AV), upper reinforcement module above the support (FV), upper auxiliary reinforcement module (PV) and upper reinforcement module (TV). These reinforcement modules were saved in separate files. The individual modules were supplemented with further modules by changing the number of pieces and the diameters, which resulted in a module library.
The designer after having drawn the formwork of the beam inserted the modules by means of specific handles - the modules recalled the respective cross-sections as well.
Even though the development of the system and the adaptation to a computerised working environment was rather time-consuming, it was worth the effort because our productivity increased significantly. We managed to exploit the biggest advantage of computers: easy duplication and modification.

Fig. 8 The "male" section of the building at the end of January 2002

6. CHRONOLOGY OF ASIA CENTER

February 2000. First meeting of Strabag International and Plan 31 Ltd. regarding the ASIA Center Project.
March 2000. Contact is made between Lackner & Raml GmbH and Plan 31 Ltd.
April-May 2000. Structural variations: Lenger - Lackner & Raml GmbH. - Plan 31. Ltd. One of the most important phases of structural design: the type of structure is decided upon.
June 2000. Building permit structural designs (Plan 31 Ltd.)
July-Dec. 2000. - Invitation of Uvaterv Co. (foundation works, watertight basin)
- Contract preparation between client
and main contractor (the most time consuming phase of the project)
January 2001. - Invitation of CAEC Ltd. - the protracted preparation resulted in a significant lack of designer capacity
- The structural design team is complete: Lackner & Raml Ltd., Uvaterv Co., CAEC Ltd., Plan 31. Ltd.
February 2001. Contracting for the execution design of the supporting structure. First phase: "male" and "female" building part, floorspace of 120,000 sq. metre.
March-Dec. 2001. Execution design process
May 2001. Critical phase of execution design process; communication problems emerge. Data supply among individual disciplines mutually delayed.
In the meantime, execution starts in full force

June 2001. The designers' team is getting welded together, communication problems are solved (coordination of Internet-based design, of different drawing software and establishment of documentation discipline).
July-Aug. 2001. Extreme efforts of all participants; summer holidays are abandoned.
September 2001. Design process back to " normal". In the meantime it turned out that the decision concerning the structure had been correct: quick execution is feasible, spirits rebounded.
November 2001. The "male" part is completed *(Fig. 8.)*. Construction of the "female" part is accelerated. Decision is made concerning the 2nd phase (the "Father"); preparation of the design process of the 2nd phase, contracting for the execution design of the 2nd phase (88,000 sq metres).
Jan.-Aug. 2002. Structural design of the 2nd phase.
June 2003. ASIA Center opened (projected date).

7. CONCLUSIONS

The design and execution of ASIA Center showed that Hungary already acts as a member of the European Community, at least as far as construction activities are concerned. Eurocode standards are part of our everyday reality, even though they are still not as mature as an EN standard.

8. REFERENCES

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The Authors

László POLGÁR MSc (1943); civil engineer; Technical University of Budapest, Faculty of Civil Engineering; 1966 - Foreman in Hódmezõvásárhely, 31. sz. ÁÉV (State Building Company No. 31.); 1970-71 Structural Designer, IPARTERV; 1971 - Product developing engineer, chief process engineer, head of the Technical Main Depart
ment, 31. sz. ÁÉV; 1992 - managing director, PLAN 31 Mérnök Ltd., managing technical director, ASA Építõipari Ltd.
Activities: design and construction of prefabricated reinforced concrete structures and industrial floors. Chairman of the Concrete Section of the Hungarian Building Material Association; member of the Hungarian Group of *fib*.
Dr. József ALMÁSI MSc (1940); 1964-1966: Mélyépítõ Vállalat (Hungarian State Foundation Engineering Company); 1966-1995: Faculty of Reinforced Structures of the Technical University of Budapest; 1995- to present: managing director of CAEC Ltd.
Activities: execution, education, research, design. A member of the Hungarian Concrete of *fib*.
Balázs KARKISS MSc (1973); static designer of Plan 31 Mérnök Ltd.
Activity: Designer of Óbuda Center, Csaba Center, Metro, Tesco, Praktiker department stores and reinforced concrete structures in Romania and Bulgaria.
Péter VARVASOVSZKY MSc (1977); static designer of CAEC Ltd. (MOM Park, Cora Hypermarket Miskolc, Zollner Elektronic Vác manufacturing hall, gate-keeper's lodge and office block, Szeged Sewage Treatment Plant, Trend manufacturing hall).
Dr. Csaba PETHÕ MSc (1948); 1972: static designer of Út- Vasúttervezõ Vállalat (Hunarian State Road and Railroad Design Company); currently the deputy head of the Underground Railway- and Structural Design Office of Uvaterv Co.
Activities: designer of tunnels, underground railways, reinforced concrete structures, underground structures.
Erika TATAI MSc (1973), Technical University of Budapest, Faculty of Civil Engineering; 1997-1999: Geo Terra Geotechnica Iroda. Currently a design engineer of Uvaterv Co.
Activities: designer of reinforced concrete structures and underground parking lots.