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MECHANICAL STATE ANALYSIS AND AN ORIGINAL SEQUENCE OF CALCULATIONS OF A HUGE BAY ARCH UNDER STATIC AND DYNAMIC ACTIONS

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Abstract. In the paper the simulation ideas of steel 255 m bay arch are reviewed. For modelling of wind, temperature and flexible guys mechanical work an original sequence based on a variational procedure have been prepeared. Linear static, linearized dynamic and stability, geometric non–linearity analyses are described, different support conditions in the numerical model are considered. The results of these numerical investigations were applied for the basic structural design of the building.

Keywords: sequence, algorithm, variation, arch, stadium, dynamic action, design.

Introduction

Development of sports and enhanced interest to public entertainments predetermines growth of such kind of buildings. Among these constructions the Olympic and National stadiums characterized by their versatility are the most important. Not less important are questions of sports technologies and comfort of spectators and as an aspect of this problem protection of the stands for spectators (and sometimes the whole arena) from atmospheric exposures. This can be achieved by using roofs, visors or coverings.

Building of covered stadiums has started in the 90s of XIX century and it extended especially rapidly in the 20s of XX century. The priority in this field belongs to the USA. This initiative was taken up by the European designers in the 30s of XX century. A rapid development of the sports movement in the 50s of the XX century initiated a wide scope of building different covered stadiums in Europe, North America, South America, and Asia. For example, in the former Soviet Union there were only 3 covered stadiums in 1956, but in 1970 their number has increased up to 30 (CNIISK Report 1988).

Design engineering of the optimum shape of the stands and the stadium contour was stipulated by the desire to place the majority of the spectators in the most convenient areas for the centre of the oval arena. This results in the necessity to put the roof over the stands into an elliptic shape. It should be kept in mind that the sports standards require to have the possibility of using exactly an open arena.

A traditional classification of the structural design decisions as for local roofs and closed general coverings of the stadiums as well is presented by three types:

– suspension systems (membranes, guy ropes, bars or combination structures);
– dome or arch systems;
– beam systems of a variable cross section (may be with openings in the walls) or systems of trusses.

On 9 March 2007 the famous arch has been erected over a stadium in England, Wembley. Since that time such a monumental though not always rational decision became rather popular all over world. At present, there are about 7 stadiums with arch system in the world. It should be noted, that the arch is not only an expressive element of architecture for the construction as itself and for the whole city but in addition it carries the systems of the stadium illumination and means of temporal covering.

In 1985 six design versions of the Central stadium in Vilnius have been exhibited for a competition. In one of the versions it was supposed to erect an arch (Fig 1). But the conqueror of this competition turned out to be design of the stadium having a steel membrane covering– its building has been started in 1988 but was stopped in a year. In 2007 the architects, who have been designing the stadium in 1985, have prepared a new basic design (Nasvytis *et al*. 2007) of the arched stadium (Fig 2).

It was planned to put the stadium into service in June 2009 and name it in honour of thousand years since the moment of mentioning Lithuania in literature. The stadium has been designed for 25 thousand of sitting spectator seats and 5 thousands of added seats. In case of concerts or national festivals the arena could be filled with 50 thousands of participants. The basis of creating a unique sequence for the arch design, its realization and some results are described further.

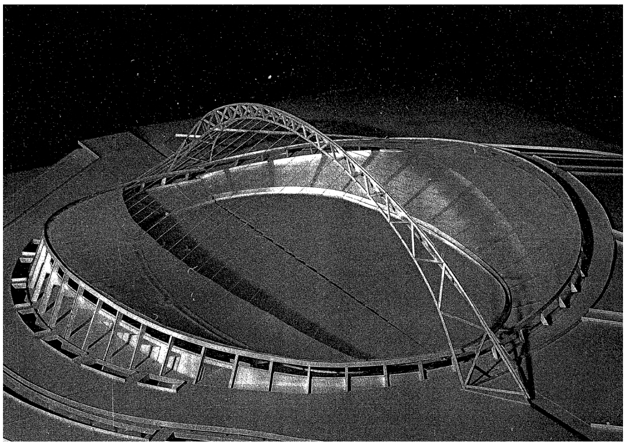


Fig 1. One of the design versions of proposals for building of the Central stadium with the arch, 1985

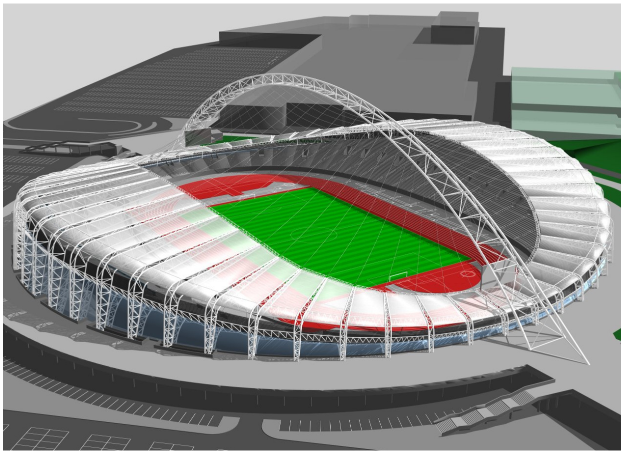


Fig 2. Visualization of the arched stadium in Vilnius, designed in 2007

General description and support conditions

The arch represents a trussed structure consisting of three chords (two bottom and one top). The arch supports are placed beyond the external contour of the main building around arena (Fig 3).

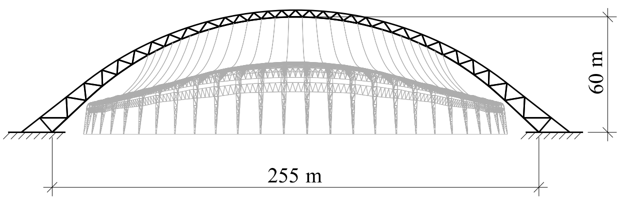
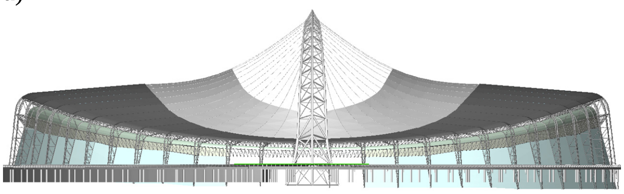


Fig 3. The arch general view

Distance between axes of the arch inner chords at the level of supports is of 255 m. The arch cross section is a true equilateral triangle slightly distorted near support zones. The cross–sectional height (distance between axes of the chords) at the edge (minimum) is of 3,5 m, near supports (maximum)– 10 m. The arch chords are made of round steel pipes of 1,0 to 1,4 m diameter, the lattice – of pipes of 0,4 to 0,8 m.

In the design requirements for the arched stadium service a temporal thin flexible tent is provided. It covers space over the arena in summer time and can be partially or completely rolled out over the cables (Fig 4).

a)



b)

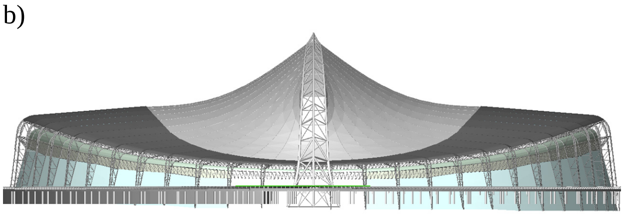


Fig 4. The tent roof, stretched over the stadium arena: by a half of the cable length (a); by the whole length of the cables (b)

As the stadium roof consists of 56 symmetrically arranged cantilever frames, the arch is jointed with the roof internal contour by means of 56 cables. This system of cables stabilizes mechanical work of the covering and arch uniformly distributing horizontal displacements of the arch and cantilever truss sags under the spectators' stands. Geometrically non–linear work of the cables serves as a damper in this case. It is rather difficult to predict deformation of the cable systems of such a structure. Even a more complex problem is an investigation of the mechanical state of the structures when bulging of the tent with the cables upwards is taken place– this problem is not considered by authors in the presented paper.

The sequence of the stadium arch calculation is complicated due to basically different types of influences on the structure and a respective reaction of the construction. According to types of deformation three cases are being considered:

– statics and kinematics (all loads with the exception of wind pulsation and thermal actions via deformations, respectively) by an approach of the *finite element method* (FEM) are presented by formula

$$\left[\boldsymbol{K}\right]\boldsymbol{u} = \boldsymbol{F} \quad (1)$$

– dynamics (pulsation of wind) expressed by the FEM approach in the same manner

$$\left([\boldsymbol{K}]-\omega[\boldsymbol{M}]\right)\boldsymbol{\psi}=\mathbf{0} \tag{2}$$

– stability (arch as a general structural system) in the FEM way

$$\left([\boldsymbol{K}]-\lambda[\boldsymbol{G}]\right)\boldsymbol{\phi}=\mathbf{0} \tag{3}$$

In the above presented expressions the variables are marked: $[\boldsymbol{K}]$ is a stiffness matrix; $[\boldsymbol{M}]$ – a mass matrix; $[\boldsymbol{G}]$ – a geometric stiffness matrix; $\boldsymbol{F}$– a load vector; $\boldsymbol{u}$ – a displacement vector; ψ – a mode shape vector in the dynamic analysis; ϕ – a stability critical shape vector; ω – a value of a natural frequency; λ – a non–dimensional critical load factor in the stability problem.

Despite of different kinds of the system reaction an elastic stage of the steel deformation is being considered. Deformation of the soil foundation and a reinforced concrete slab is treated as occurring step–by–step, i. e. absolutely rigid supports at short–term actions and elastic ones at long–term actions. When simulating mechanical work of the structure the following decisions have been used:

– linear (selfweight of bearing constructions, weight of the engineering equipment, thermal actions);
– linearized (dynamics and stability problems);
– non–linear (calculation of geometrically variable flexible cables).

Due to considerable support reactions of the arch foundations are provided by means of a base plate resting upon uniformly distributed piles. Because of presence of essential thrust reactions, the piles are placed at an angle of 15 degrees with respect to the vertical. In case of an action of the long–term loads (mainly selfweight of constructions) transmission of forces to the soil is simulated by means of a system of vertical and horizontal elastic springs of each pile. With an action of the wind pulsation, the soil cannot react instantly (to become deformed) in response to the exposures and manifests itself as a much more rigid support (8 to 20 times). With regard to the above described features, two kinds of support variants have been simulated (Fig 5)– a piled plate on the elastic springs and absolutely rigid supports (Fig 6).

a) b)

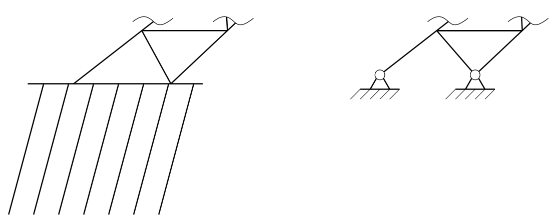


Fig 5. Kinds of the arch supporting: a piled plate on the elastic springs (a) and absolutely rigid joints (b)

The soil resistance directly under the foundation plate (an area of contact between the plate and soil) has not been considered– such an assumption is taken to a safety side.

a)

b)

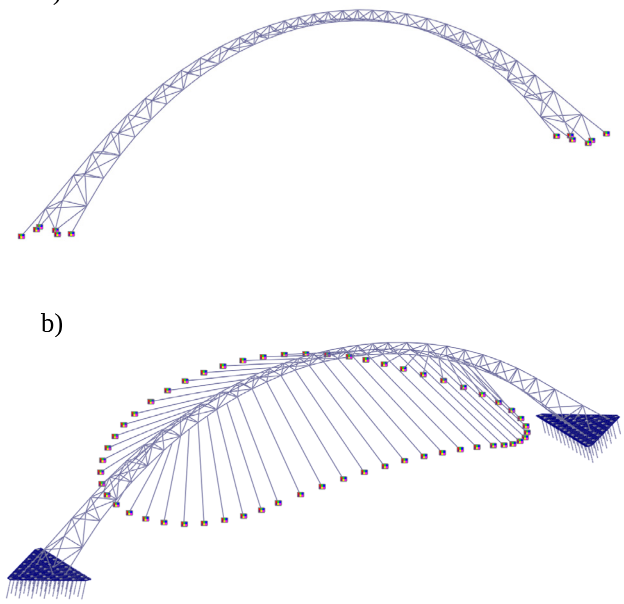


Fig 6. Arch calculation models: rigid joints during mounting (a) and piled plates at the service stage (b)

The designs assume also that the arch will be assembled rapidly (in 2 months) in comparison with the service life of the whole building (100 years) and will have no essential effect on setting of the foundations.

Design situations during mounting and service processes

According to the used Lithuanian and the EU codes (STR 2.05.03 2005; LST EN 1990 2004) the class of responsibility of the building is RC3, class of possible consequences– CC3, thus all values of the effects are increased by the safety factor of 1,10.

In accordance with common engineering practice all acting loads are subdivided into dead loads:

– selfweight of soil, concrete and steel structures;
– weight of engineering networks and technological equipment;
– preliminary tension force of the cables;

life loads, some of which (above presented by percents) are considered as long–term actions in case of an estimate of crack propagation of reinforced concrete plate:

– snow (20 %);
– climatic temperature (for a summer season is about 60 %, winter– 80 %);

life load short–term part:

– wind (static and pulsation components);
– temporal thin tent;
– useful load due to visitors or operation staff;
– load arising while mounting.

The safety factors at dead loads are assumed to be of 1,35, for life loads– 1,50 (STR 2.05.04 2005; LST EN 1991 2004). The pre–tension of the cables is also considered as a long–term action, though in reality it is not always so. In contrast to other effects, the preliminary tension forces are an artificial action and, therefore, deviations of the designed values are controlled beforehand. Monitoring of the cables behaviour is also provided in the service manual of the building. The above mentioned arguments have confirmed that in case of the cables the safety (overload) factors not always contribute to the structural safety in general.

The structure selfweight is given, as a rule, with allowance for density of the materials (STR 2.05.05 2009; LST EN 1992 2008; STR 2.05.08 2007; LST EN 1993 2009; STR 2.05.04 2005; LST EN 1991, 2005), weight of engineering networks and basic equipment– by the customer agreed design requirements.

Actions of forces due to a pre–tension of cables on the arch bearing structures are problematic in two aspects: selection of different (the most rational) values of pre–tension forces for a long operation stage; tasks of the tension sequence for a mounting stage.

A load due to the snow weight is set with account of areas: chords and horizontal lattice of the arch, operating bridges, basic equipment. Load value has been calculated by expression:

$$s = \mu \cdot s_0 \tag{4}$$

where s_0 is a characteristic value of the snow cover weight by 1 m^2 of a horizontal ground surface, μ– reduction coefficient of the snow cover.

Thermal effects acted on the steel arch and cables are taken with account of temperature drops which can occur in the climatic zone (RSN 156–94 1995) of building in warm and cold seasons, respectively:

$$\Delta t_w = t_w - t_{0c} \tag{5}$$

$$\Delta t_c = t_c - t_{0w} \tag{6}$$

where t_w and t_c are characteristic values of average temperatures in the warmest summer month (June) and in the coldest winter month (January), t_{0w} and t_{0c}– respective temperatures during the mounting process of the structures. Increase in temperature as a result of direct exposure to sun rays on the structure surface is also taken into account in these formulas. A long–term part of the temperature influence is specified by its difference during the year, short–term– during the day.

Loads due to wind action are the most complex in this problem. First, the wind has an effect on the tent and efforts via the cable system are transmitted to the arch. A value of the load depends on its distribution over surface, i. e. on the aerodynamic coefficient $c_e(\beta)$. A distribution function c_e depends of the wind direction angle β. Data of wind effects on the tent have been acquired as a result of investigation of the structure features in the wind tunnel (Samofalov *et al.* 2008). The wind also directly acts on the structural bars in two main directions: along and across arch. The wind diagonal directions are considered as a linear combination of longitudinal and transversal ones. The wind action area of the arch segments has been selected applying engineering methods described in the design codes (STR 2.05.04 2005). Eight key directions of the wind (specified by the whole stadium design) were studied: two opposite directions along the arch and across it, four diagonal ones. The wind force relative to cardinal point (azimuth) was corrected by the coefficients of 0,80 to 1,00. The wind velocity:

$$v_{ref}(\beta) = c_{dir}(\beta) \cdot c_{tem} \cdot c_{alt} \cdot v_{ref\,0} \tag{7}$$

where $v_{ref\,0}$– wind velocity, c_{alt}– coefficient allowing for a global altitude above the sea–level, c_{tem}– coefficient of duration of an actual situation to be considered (value 1,000 when designing the long–term service stage of the building, 0,806– mounding stage by STR 2.05.04).

A characteristic value of the wind load calculated by the formula

$$q_{ref}(\beta) = \frac{\rho}{2} \cdot v_{ref}^2(\beta) \tag{8}$$

where ρ is an air density. It should be noted, that in this formula the squared wind velocity provides one third decrease during mounting load in comparison with the service one. The static component of the load is determined by the formula:

$$w_{me}(\beta, h) = q_{ref}(\beta) \cdot c_h(h) \cdot c_e(\beta) \tag{9}$$

where h is the height of a point to be considered above ground surface, c_h– coefficient allowing for distribution of wind pressure by height.

In addition, to chords and lattice of the arch the wind load is applied to the cables when the tent is not tightened. Such action is considered to be the main one in the non–linear deformation of the cables.

The temporal tent over the stadium arena can be stretched only in summer season, so this loading is not compatible with loadings of snow and winter temperature. A load from the tent weight is directed downwards and is considered as a short–term load.

Under service conditions the arch is supposed to be visited by the spectators. This possibility is provided but this kind of load is taken to be similar to that one produced by maintenance of the technological equipment of the arch (illumination of the arena, audio and video techniques and so on).

The mounting loads on the arch are considered in two ways: action with a direct uplifting of the segments to the designed position; short–term existence of the arch during mounting of the roof while there are no cables.

Preliminary investigation of the structural features

The advance software and hardware enable to create and simulate the complex models of the buildings (Samofalov *et al*. 2007; Перельмутер and Сливкер 2007a; Perelmuter and Fialko 2005). While considering a set of detailed decisions form these models one main (global) error can be overlooked. Usually, for bearing construction of multifunctional buildings simulation of their mechanical work only on the basis of conventional linear static analysis ideology is insufficiently as a traditional approach provides correction of aspects of a stage–by–stage character of assembly, becoming apparent physically non–linear features of the materials, geometrically non–linear working members etc (Jankovski and Atkočiūnas 2008; Kalanta *et al*. 2009). Taking into account these properties of design models one should thoroughly elaborate a plan of the numerical test in advance. The experience of planning such experiments has been accumulated (and can be borrowed) from practice of full–scale natural and laboratory investigations of the structures (Касаткин *et al*. 1981; Wu and Hamada 2000).

For our specific case of a huge bay arch of the stadium the following stages of numerical simulation have been provided (Fig 7).

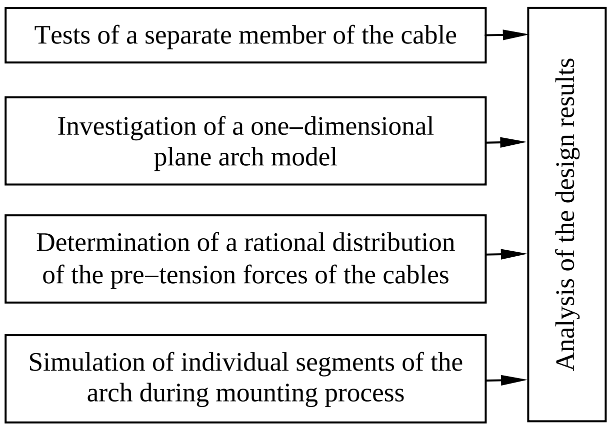


Fig 7. Stages of the features investigation for separate structural members and arch state

Design of a separate member of the cable is performed in order to study peculiarities of solution of such a problem being used by the software (Карпиловский *et al*. 2004). The last cable–type structural member is specified by a formula in the analytical manner

$$\Delta l = \frac{\Delta H \cdot l}{EA} - \frac{1}{2}\left(\frac{D}{H^2} - \frac{D_0}{H_0^2}\right) + \alpha \cdot \Delta T \cdot l \tag{10}$$

where the shear actions are expressed by the integrals:

$$D = \int Q^2(x)\,\mathrm{d}x \tag{11}$$

$$D_0 = \int Q_0^2(x)\,\mathrm{d}x \tag{12}$$

In the above mentioned formulas the following variables are employed: l is length of a chord; Δl – variation of the chord length; $Q_0(x)$ and $Q(x)$ – shear forces due to action of initial $q_0(x)$ and final $q(x)$ transversal distributed loads respectively; function argument x points the longitudinal coordinate along a chord of the cable; H_0 and H – initial and final axial forces; EA – tension stiffness; ΔT – variation of the thermal action; α – coefficient of materials.

The equation shows that the temperature action on deformations is a linear one. The shear forces act in two manners: linearly and non–linearly. A non–linear component of deformations due to the shear forces action is corrected by a transversal load. The tests of one cable for the simple system (Fig 8) have shown that there is a full convergence even with one iteration step.

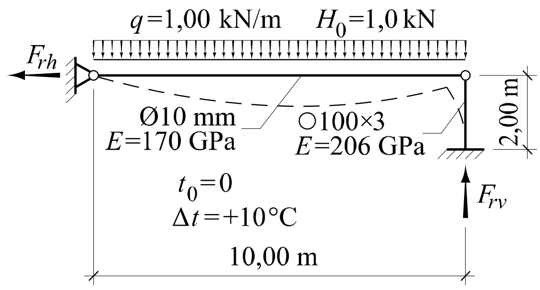


Fig 8. The test model of a separate cable

The static linear analysis of one–dimensional plane arch model under a principal load enables to investigate the distribution of stress/strain state parameters (Fig 9). At this stage of investigations one makes analysis of the state in cases of vertical symmetrical load (meaning, first of all, selfweight) and radial non–symmetrical load (wind action along the arch). Influence of cables on the arch was excluded in the model. A linearized dynamic analysis was also made. It was aimed at definition of natural frequencies and respective shape mode of the plane arch. Values of the lowest frequencies (Hz): 0,75; 1,89; 3,57. In addition, a linearized stability analysis is carried out. The aim of this analysis is to get a plane mode shape of arch stability.

Determination of the rational pre–tension forces of the cables was considered as an individual simplified optimization problem (Корнилов 1978; Лихтарников 1979). In this case the space arch cables were fixed at the points which were located on the internal contour of the roof under spectators' stands. The first effect to be investigated is thermal action.

In winter season the arch ridge due to action of temperature is moving downwards and the cables must be tensioned to a minimum level, in summer season the cables should not overload the arch excessively hindering its movement upwards under action of the temperature (displacement due the temperature deformations of the longest cable is 2…3 times less than that one of the arch ridge). While selecting the pre–tension force of the cables the wind load on the arch and cables was considered when three versions can be available: without tent; the tent is tightened over a half length of the cable; the tent is fully lifted. In all cases the wind directions along the arch and across it, including the pulsation component of a wind stream, have been taken into considerations. With longitudinal and transversal wind directions the arch structure with cables works symmetrically but differently (due to temperature action– equally).

The main criterions when selecting the pre–tension forces should be as follows: unification; a cable must be always able to be stretched; designing of the arch bars. The deformed shape is also very important– as a result of the action of cable pre–tension forces the arch should deflect uniformly without local distortions. The problem of the cable strength is not urgent and therefore the diameter is selected to suit design considerations. The ridge displacements make up 1/1000…2000 of the arch bay. Finally, a following assumption was made: minimal possible pre–tension forces should be distributed linearly depending upon a length of the cables (Table 1).

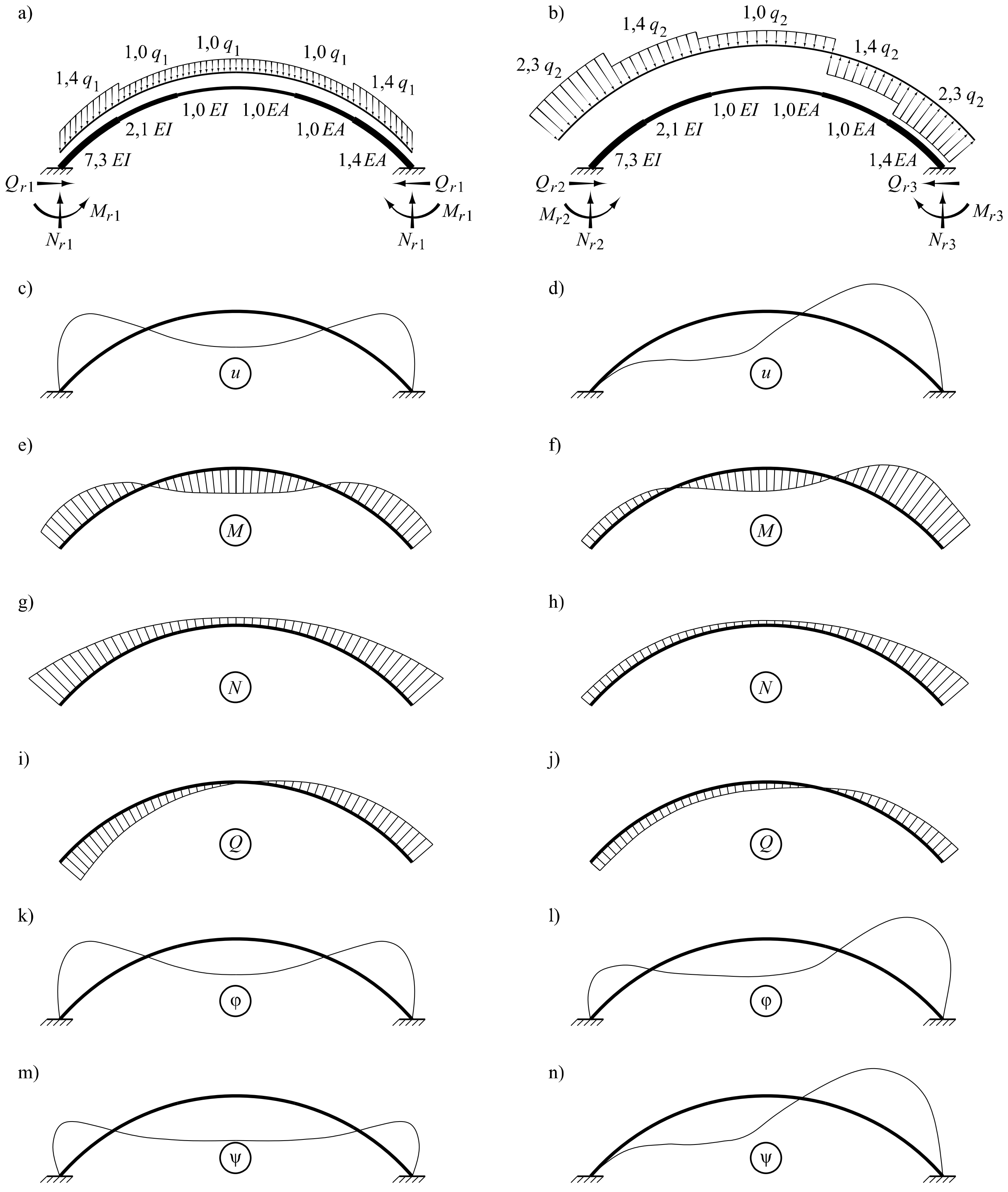


Fig 9. One–dimensional model, loaded by symmetrical (a) and non–symmetrical (b) loadings, respectively: deformed mode shapes (c, d); diagrams of internal forces (e, f, g, h, i, j); the first mode shapes of natural frequencies (k, l); the first mode shapes of stability (m, n)

Table 1. Values of the length and pre–tension forces of cables

Axis No.	Relative length at geometric chord	Relative pre–tension force
1	0,47	0,13
2	0,54	0,20
3	0,62	0,27
4	0,70	0,33
5	0,77	0,40
6	0,83	0,47
7	0,87	0,53
8	0,91	0,60
9	0,93	0,67
10	0,95	0,73
11	0,97	0,80
12	0,98	0,87
13	0,99	0,93
14	1,00	1,00

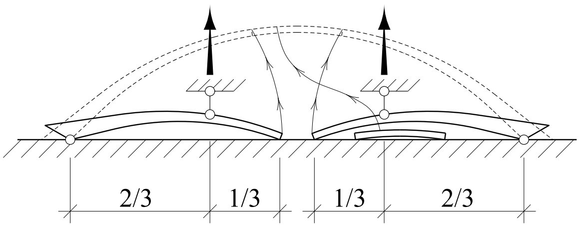


Fig 10. Arch mounting scheme

Design of individual segments of the arch is important when considering variants of raising separate parts to the service position under mounting. In our case the variant of raising two halves has been adopted (Fig 10). For this a hinge was provided on the arch foundation plates. A crane of the least carrying capacity (i. e. the cheapest one) can be employed during lifting a half of the arch by its free edge, but verification of the design parameters of the chords gave negative results. So, a “safe” point was selected (without increase in cross sections of the structure). The crane of a large carrying capacity has been chosen in this case.

General algorithm of calculations

Analysis of the arch structure in itself is considered to be oriented, as a final and complete design is accomplished by using a model of the whole stadium. As a matter of fact, namely such a stage will be conclusive one as influence of the arch on the structure covering over spectators’ stands and vice versa will be taken into account. Nevertheless the analysis of the arch as an individual structure is very tedious and so it would be unreasonably to investigate the arch as a structural part of the large design model.

After review of the loadings and preliminary analysis of the arch structure unique features different versions of the design have been created. The design results of separate models were reasserted and accepted while checking defined steel cross sections and reinforced concrete (Fig 11).

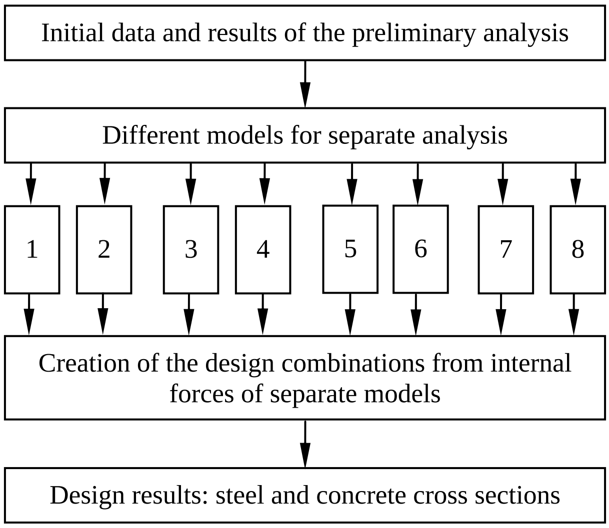


Fig 11. General algorithm of calculations

Each of the models is twin and properties of its geometry do not allow to design it together with the next one. This fact complicates the design but provides the opportunity to consider situations different in principle.

Stress/strain state of the models is studied under action of the temperature or a wind load during mounting or service. As the temperature is predetermined by two independent extreme values (in summer and in winter) and wind– by orthogonal longitudinal and transversal directions– then one gets 8 independent design situations. Each one can be characterized briefly:

1. The arch is loaded by selfweight, it is subjected to action of a positive temperature. As the actions are of a long–term type, the model supports on a piled plate on elastic springs. A mounting stage when there are no cables is being considered. The static linear analysis is used.
2. The same as in 1, but only the temperature is negative.
3. The arch is loaded by selfweight, weight of engineering networks and basic equipment. The arch is subjected to action of the temperature in summer season, the tent is tightened. The model supports on a pile plate. It is one of the service load versions. The static analysis is “poorly” non–linear as there are cables loaded with weight of the tent.
4. The same as in 3, but in winter season and therefore the tent is not available.
5. The arch, besides selfweight, is undergone to action of the wind in the transversal direction when mounting is being performed and cables are not available. The arch supports are absolutely rigid because of instant wind pulsation effect. This analysis is linearized dynamic for

the wind and stepwise "highly" non–linear– for the cables.

6. The same as in 5, but the wind is directed along the arch.
7. The arch is loaded by selfweight, weight of engineering networks and technological equipment, the tent is tightened. The wind blows across the arch. The supports are rigid. The service stage is considered. The analysis is dynamic linearized for the wind and stepwise "highly" non–linear for the cables.
8. The same as in 7, but the wind blows along the arch.

After analysis of each model one gets internal forces for separate loadings (selfweight, weight of engineering networks and equipment, temperature, wind) with respective preset coefficients taking account of mounting and service stages. At the variational calculations stage the design combinations are being formed from internal forces of separate models due to individual loadings. For example, for the service stage the selfweight, weight of equipment and action of the temperature are taken from the model Nr. 3 on the elastic foundation and combined with the internal forces from a model Nr. 7, which was supported on the rigid supports. In this example the selfweight from the model Nr. 7 is excluded, masses due to weight are used only in calculation of natural frequencies. It is evident, that the algorithm is based on original ideas of software authors (Перельмутер and Сливкер 2007a).

Actually according to the above explained algorithm not only internal forces are combined but also all other parameters of the stress/strain state.

Nevertheless each of the models is quite efficient and is interesting for investigation. Therefore, for each model separately the system is additionally checked for stability (Перельмутер and Сливкер 2007b).

Results

As the main practical result of simulation of the arch mechanical state is the conclusion whether the steel and concrete cross sections are initially appropriate. Not less important for the building of this kind are displacements due to static (Fig 12) and dynamic (Fig 13) actions.

In calculations the assumption is taken into account that the arch natural frequencies do not depend on the availability of the cables. Values of the first three frequencies (Fig 14) differ to a large extent: the first 0,58 Hz (a flexible mode shape out of arch plane); the second 1,06 Hz (flexible in–plane); the third 1,40 Hz (torsional out–of–plane).

Taking into account an original shape of the arch it is useful to know about actions of wind dynamics, expressed by formula:

$$\eta = \frac{u_{st} + u_{dyn}}{u_{st}} \qquad (13)$$

where u_{st} and u_{dyn} are displacements due to actions of static and dynamic components of the wind load. For the wind along the arch irrespective of availability of the cables the dynamic factor will be of the order 1,3. For the transversal direction: without cables 2,5, with them– 1,7. for the diagonal directions the factors were calculated as a linear combination of longitudinal and transversal directions.

Investigations of the arch global stability have shown positive results– a stability coefficient in all cases exceeds a value of 10. The stability mode shape– flexible out–of–plane (Fig 15).

The essential result of the arch features investigation is the analysis of the support reactions values by means of which foundation plate, piles, strength and rigidity of elastic soil have been examined.

a)

b)

c)

d)

e)

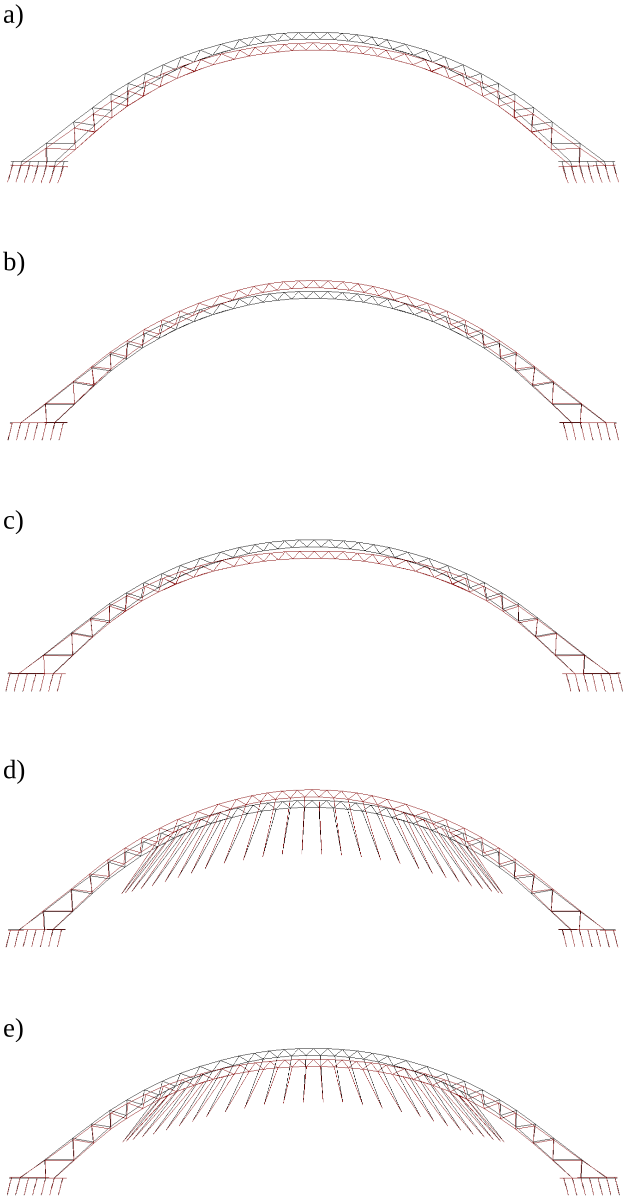


Fig 12. Deformed shape as a result of actions: selfweight (a); temperature in summer (b) and winter (c) under mounting; the summer temperature at a stage of service (d); pre–tension of cables (e)

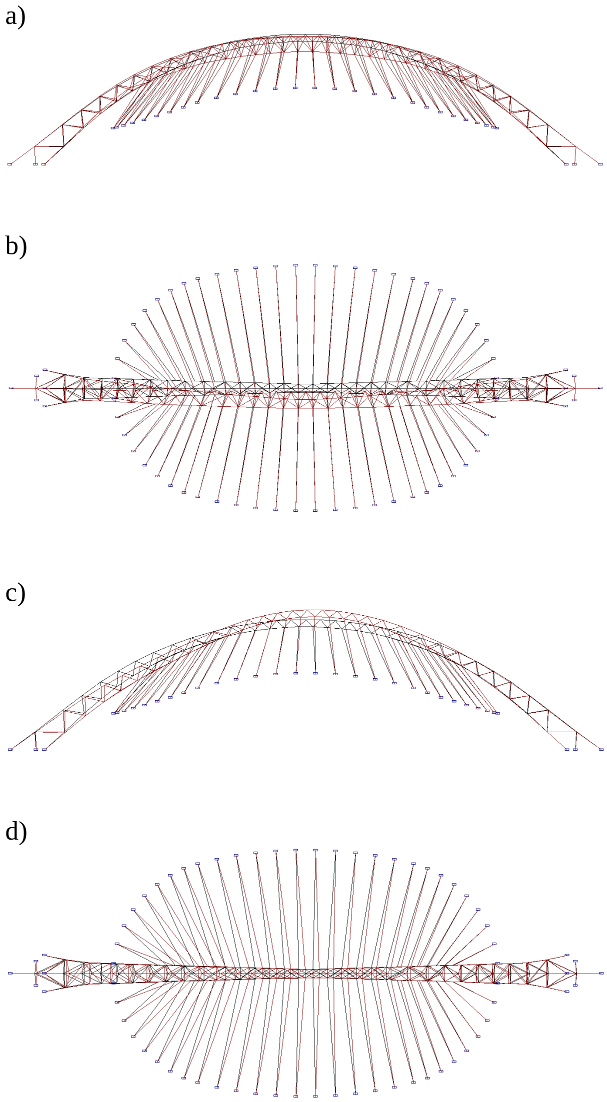


Fig 13. Deformed shape due to wind actions at a service stage: across the arch (a, b); along the arch (c, d)

Conclusions

As a result of initial data analysis for stadium arch designing as well as preliminary calculations and main solution the following conclusions are made:

1. For the unique buildings one should elaborate the untraditional design methodology, depending on individual properties of the construction and specific conditions of a real problem (including experience and technical conditions, available traditions in engineering at this region and so on).
2. For development of simulation algorithms of complex structures scientific supporting and substantiation of decisions are quite necessary. Information about the design sequence of real designed buildings is the important and useful knowledge for scientific researchers and practice engineers in the sense of improving their competence.

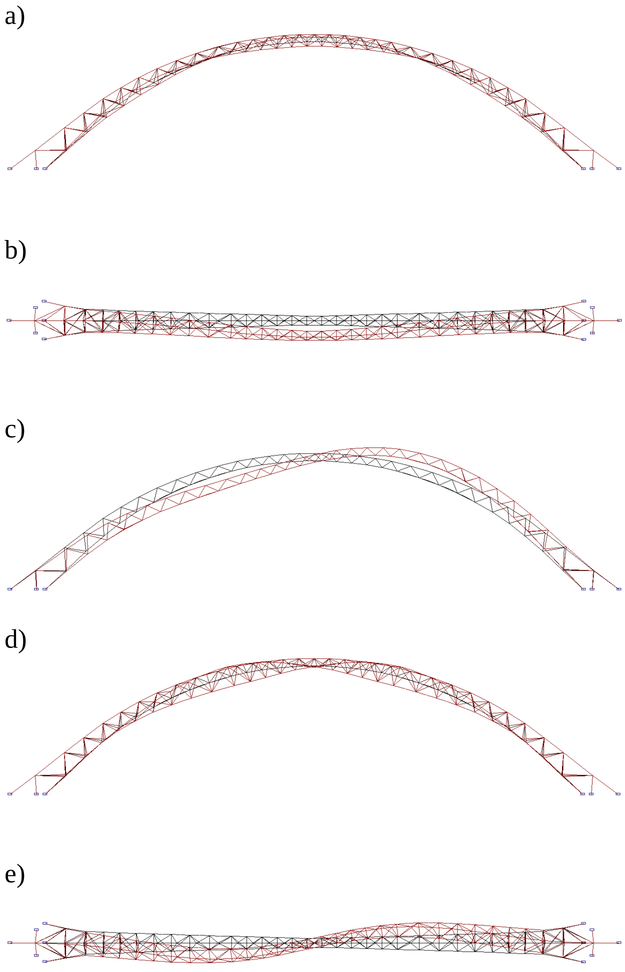


Fig 14. The natural mode shapes of the space arch of the frequency: 1st (a, b); 2nd (c); 3rd (d, e)

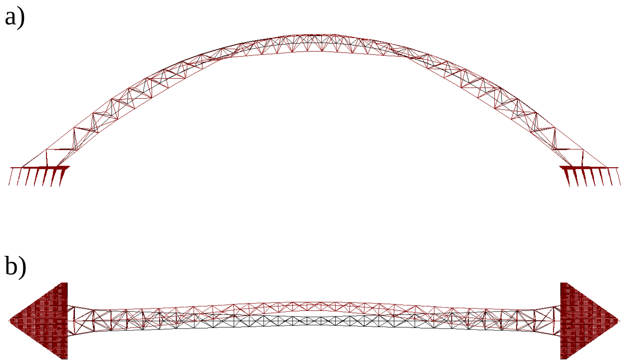


Fig 15. Views of the first stability shape mode: side (a); plane (b)

3. During the structural analysis of the design of original buildings there should be provided a preliminary stage of stress/strain state investigation for individual structural members, joints as well as specific conditions. A sequence of such kind of numerical experiments should be planned.

4. A sequence of research of the structural features and a final calculation algorithm must consider and use peculiarities of the software to be employed.
5. In optimization problems the attention should be paid to the fact that the most important result is just a last one– a design of the real reliable building.

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