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Autor: Seliverstov, Vadim
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Bridge Foundations Design Practice - Codes Development in Russia.

Vadim SELIVERSTOV
Civil Engineer, P.E.
Giprotransmost J.S.Co
Moscow, Russia

Vadim Seliverstov, got his civil engineering degree in 1981 at Automobile and Road Technical University of Moscow. He has more than 17 years professional experience in bridge engineering. Vadim Seliverstov is now with a consulting firm and is involved in major bridge projects.

SUMMARY
This paper presents an overview of bridge foundation design practice in Russia. General principles specified in the current design codes regarding bridge foundations are briefly discussed. Limit states principles adopted for design of bridge foundations are summarised. A comparison with some provisions of codes of other countries is briefly discussed. Also some information on methods of scour assessment at bridge piers including consideration of influence on construction sequence is given. Some general notes to improve design procedures are introduced in light of the recent change of codes in Russia.

1. INTRODUCTION

The reliability of any bridge and its economic viability is not based solely on the choice of superstructure type. The proper selection of the substructure system including the details of the elements for that system plays an important role also. The cost of bridge foundations is normally about 30% of the cost of the bridges. Along with this construction time and labour intensity related to bridge foundations give about 40% of the time and labour intensity required for the whole bridge. In complicated geological conditions and where foundations are needed to be constructed at a large water depth, the cost of substructures may reach up to 60% of the total bridge cost. Therefore selection and design of the effective foundation for bridge piers is an important consideration and depend upon many aspects. These are loading conditions, bridge pier geometry, geotechnical and hydrologic conditions at the site.

2. MAIN PROVISIONS OF FOUNDATIONS DESIGN

2.1 Methods and design codes

Considering the required high reliability of foundations, in some countries the design is based on permissible stress procedures. In the Russian practice the limit state principles were adopted for design of bridges since 1962. The code requirements are specified for two groups of limit states. The design of bridge foundations is based on the requirements of codes: СНиП 2.05.03-84* «Bridges and culverts» {1}, СНиП 2.02.01-83 «Foundations of buildings and structures» {2}, СНиП 2.02.03-85 «Pile foundations» {3}. The workmanship levels are specified by the other code.

The Bridge code СНиП 2.05.03-84* {1} has single volume and covers design of new and rehabilitation of existing highway, railway, pedestrian and combined (highway - railway) bridges and culverts in Russia. Bridge foundations are designed to withstand loads stipulated by the Bridge code . Also this code provides the requirements for structural detailing of bridge foundations. The current Bridge code does not specify qualitative and quantitative criteria of limit states for particular structure types, but contains them in general form only. Generally in connection to bridge foundations the first limit state relates to a loss of bearing capacity of soils, stability of foundation due to overturning or sliding, strength and stability of structure and its structural elements. The second limit state covers deformation of bearing soils below the foundation (settlements, tilting, horizontal displacement), crack resistance of reinforced concrete foundation structures.

The following Table 1 summarises the types of calculations to the 1st and 2nd limit states. The objective of calculations to the code is that the abovegiven limit states should not occur within the expected lifespan of the structure. This is ensured by the use of a system of coefficients applied to nominal loads and strength characteristics of materials.

Types of calculations	Shallow foundations of		Deep foundations of	
	abutments and piers at bank slopes	piers	Abutments and piers at bank slopes	Piers
Limit state I				
Bearing capacity of soil (rock)	+	+	+	+
Stability of foundation against overturning	+	+	-	-
Stability of foundation against sliding	+	-	+	-
Stability of foundation against deep shear	+	-	+	-
Strength and stability of foundation structural members	+	+	+	+
Limit state II				
Deformation of bearing soils (settlements, tilting, horizontal displacement)	+	+	+	+
Crack resistance of reinforced concrete foundations	+	+	+	+
Crack resistance of concrete foundations	-	-	+	+

Table 1. Types of calculations to limit states

Geotechnical design parameters to be used for the analysis of capacity of bearing material are determined in accordance with the requirements of the other code - СНиП 2.02.01-83 «Foundations of buildings and structures». For the cases not covered by this code the geotechnical design parameters are determined in accordance with the approach established in the Bridge code. Pile foundations are analysed to the methods stipulated in СНиП 2.02.03-85 «Pile foundations».

2.2 Analyses of foundations

2.2.1 General considerations

When computing foundations (e.g. determination of load effects acting in a cross section of members, pressure on soil, horizontal and angle displacements) the surrounding foundation soil is allowed to be considered as linearly-deformable system {1}. This linear-deformable system is characterised by coefficient of deformation, increasing proportionally with depth.

Computation of structural strength is made {1} using reliability coefficients of dead loads $\gamma_f > 1$, in case these loads increase a design action (e.g. selfweight of substructure when calculating section strength or resistance of bearing material). In case dead loads reduce the design action, the reliability coefficients are taken as $\gamma_f = 0.9$ (e.g. selfweight of substructure when calculating pier to stability against overturning).

In the Russian and other countries practices to optimise foundations of various types, load tests of rock or soil and individual structures are normally conducted. The most widespread are plate bearing tests, pile tests (trial piles or test piles). These tests are performed to assess the bearing capacity and modulus of the ground, to investigate performance, to check quality of construction.

2.2.2 Bearing capacity of founding material

Design resistance of founding soil (axial capacity) below shallow foundation or caisson is determined {1} from the equation

$$R = 1.7\left\{R_0\left[1 + k_1(b+2)\right] + k_2\gamma(d-3)\right\} \qquad (1)$$

In the above equation R is the design resistance of founding soil, kPa; R_0 is the conventional resistance of soil (the recommended values are given in the Bridge code), kPa; b is the width (the lesser side or diameter) of foundation, m (when the width of foundation exceed 6 m, b is taken as 6.0 m); d is the depth of foundation founding, m; k_1, k_2 are the coefficients depending on the soil type, m^{-1}; γ is the design specific gravity of soil layered above the bottom of foundation (γ may be taken as 19.62 kN/m^3).

According to the AASHTO specifications for highway bridges the ultimate bearing capacity of soil is recommended to be estimated using the following formulae

$$q_{ult} = c\,N_c + 0.5\gamma B N_\gamma + qN_q \qquad (2)$$

The allowable bearing capacity is determined as

$$q_{all} = q_{ult} / FS \qquad (3)$$

where c = soil cohesion, N_c, N_γ and N_q = bearing capacity factors based on the value of internal friction of soil, B = width of footing, q = effective overburden pressure at base of footing.

Design resistance of non-weathered rock (axial capacity) is determined {1} from the equation

$$R = R_c / \gamma_q \qquad (4)$$

where R = design resistance of rock, kPa; R_c = strength of rock samples under uniaxial compression, kPa; γ_q = reliability coefficient of rock material, normally taken as 1.4.

According to the AASHTO recommendations the ultimate bearing capacity of footings on rock is estimated as

$$q_{ult} = N_{ms}\,C_o \qquad (5)$$

where N_{ms} = coefficient factor which depends on rock mass quality and is given in AASHTO in the table form, C_o = compression index, which is normally determined from the results of laboratory testing of rock core. According to the AASHTO recommendations a minimum factor of safety is taken as 3.

Compared to the Russian practice the AASHTO Standard Specifications for highway bridges stipulates a more differential approach to determine an allowable contact stress for foundations on rock. E.g. the allowable contact stress below foundation on rock is determined from the results of laboratory testing of rock and the RQD (rock quality designation) values or other rating system. A direct comparison of these two approaches is rather complicated but conventionally based on a review of reliability factors, the allowable bearing pressures (design resistance in the Russian terminology) on soils and rocks obtained using the Russian code approach are larger in some cases by up to 30%.

2.2.3 Other aspects of foundation design

Normally the foundation members are designed with non-prestressed reinforced concrete. These members are analysed to specified in the Bridge code crack resistance category. The maximum specified by the Bridge code {1} crack opening is 0.30 mm. A more precise ultimate value of crack opening is taken depending on condition of the member behaviour in a foundation structure. E.g. in the zone of ice drift, the crack opening is limited to 0.15 mm. And for structural members within the water reservoirs (formed by dams), if a number of freezing / thawing cycles exceeds 50, the value of crack opening should not exceed 0.10 mm. In the BS 5400 the maximum design crack width is limited by 0.25 mm and depends on the environment regarding 4 categories. A comparison has shown that in {1} a more detailed consideration for various conditions has been provided.

One of the controversial questions in foundation design practice is the differential settlement criteria. The opinion on an acceptable value of differential settlement differs between design offices, and particularly for foundations of continuous bridges. According to the American practice {4} it is recommended at a stage of preliminary design to assume differential settlements equal to a fraction of the average of adjacent span lengths for pile foundations – 1/500, for spread footings on soil – 1/1000, for spread footings on rock – 1/2000. However the values to be used for the final design are not specified, they are recommended to be determined from the project soils report or by consultation with the geotechnical engineer. The AASHTO Standard Specifications for highway bridges require to consider differential settlement in the analyses and that its value should not exceed the tolerable movement of the structure. The same approach is stipulated by BS 5400 (part 2). In the Russian bridge code {1} the differential settlement is limited by a bend angle between adjacent spans caused by pier settlements, being 0.2 %.

The deck designed to accommodate large differential settlements is likely to be more expensive since the differential settlement may govern the design. On the other hand this cost can be negligible compared to provision of very stiff foundation designed for a small amount of differential settlement. Therefore the final choice of foundation have to be based on a review of alternative solutions supported by technical and cost comparison.

3. ASSESSMENT OF SCOUR

3.1 General

One of the most important aspects in bridge foundation design is an assessment of scour. The types of scour at bridges is normally divided into three main categories: natural, contraction and local. Natural scour relates to fluviomorphological process in rivers and occurs irrespective of whether the bridge is there or not. Contraction scour occurs because of the contraction of the waterway by the bridge. Local scour is caused by the interference of the piers and abutments with the flow.

The local scour effects at piers, abutments, training works and temporary works for bridges over rivers have attracted the interest of many engineers and researchers. However the local scour problem resulting in bridge pier failure and inadequate foundations still exists and is actual for the current practice. The present discussion will concentrate on methods of assessing local scour.

In the recent years the engineers have used various methods for local scour prediction which may lead to essential variability in resulting values. Based on the results of researches, generalisation of theoretical, experimental and field data a new code of practice for local scour assessment has been recently developed in Russia. This code of practice СП 32-102-95 "Methods of local scour calculation" {5} have regulated the principal approaches and methods of local scour calculation taking into account type of bridge structures, their structural features and various geological conditions.

The code {5} covers assessment of local scour depth for the following elements of bridge crossing: piers; abutments; approach fills at floodplains; guide banks and groynes. The given in the code methods allow to estimate scour effects in cohesionless and cohesive materials. For cohesionless material scour analysis is stipulated for two cases: sediments transporting condition and clear water condition. Also a special consideration is given to pier foundations on piles, where analysis of scour depth is dependant on location of pile cap relatively river bed after occurred contraction scour.

3.2 Estimating Local Scour in Cohesionless Soils

To predict the depth of scour in cohesionless soils adjacent to a pier (in a form of single pile etc), having permanent width of section within water depth, two cases are considered in {5}: sediments transporting condition and clear water condition. The following equation is recommended for sediments transporting condition

$$h = 0.77H^{0.4}b^{0.6}\left(\frac{V}{V_B}\right)^{\frac{1}{2}} MK \qquad (7)$$

In the above equation h is the depth of scour measured below river bed level *after contraction scour*, m; H is the depth upstream of pier, m; b is the width of pier, m; V is the approach flow velocity, m/s; V_B is the turbid (characterising suspended sediment presence) velocity for the soils under consideration, m/s; M,K are the coefficients of shape and angularity.

The established methods allow to analyse the local scour effect at piers of any configuration. E.g. the pier, having a variable section within the stream depth, is divided into elements of constant width and the «input» of each element into formation of the local scour depth is determined. In this case for sediments transporting condition the following equation is recommended:

$$h = 0.77H^{0.4}\left(\frac{V}{V_B}\right)^{\frac{1}{2}} F(b) \qquad (8)$$

where $F(b) = \sum_{i=1}^{n} b_i^{0.6} M_i K_i f_i$ (9)

In the above equation $F(b)$ is the parameter, taking into account pier geometry, m $^{0.6}$; b_i is the width of each pier section composed of n variable structural elements, m; M_i, K_i are the coefficients of shape and angularity of each variable pier element; f_i is the conditional volume coefficient.

From the European practice it is known {6} that estimation of local scour at piers (non-cylindrical shape) may be obtained e.g from formulae:
scour depth = d_s f_2 f_3
where d_s is the scour depth at cylindrical pier, f_2 is the *factor to account for pier shape*, f_3 is the factor to account for oblique flow.

To calculate the scour depth at the cylindrical pier, a number of empirical formulas for various conditions is suggested {6}. But in general all of them account for the two parameters: stream velocity and pier width. Furthermore it may be concluded that the methods stipulated in the Russian code of practice {5} consider more than two parameters. In this light it also should be noted that some engineers consider a practice to account for many variable parameters, when assessing the local scour effect, in reality has not proved to be more reliable.

3.3 Influence of Scour on Temporary Structures

Typically the construction of foundations requires initial placement of sheet piling. When designing temporary structures within the river, it is important to take adequate account of the effect of scour. In some cases the depth of scour at sheet piling may exceed the predicted value of scour at the permanent pier. Therefore special measures is needed to be adopted before sheet piling are removed.

Based on the recent model studies the most rational sequence of sheet piling construction may be determined {7}. To control the minimum scour depth, the construction have to be commenced at longitudinal axis of sheet piling from downstream. Parameters and sequence for the outlined rational placement of sheet piling of cylindrical shape are given in Table 2.

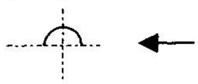
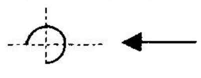
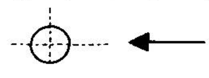
Sequence	1	2	3	4
Cross section				
M	1.70	1.12	1.02	1.00
K	0.80	0.63	1.00	1.00

Table 2. Parameters of effective construction sequence. *Note*: for notations M and K see sub-chapter 3.2

Similar investigations were conducted for sheet piling of non-cylindrical shape. Based on the results of model study, the most rational scheme of construction have also been outlined. The placement of sheet piling have to be commenced at one side only and be proceeded in the upstream direction.

4. RENEWAL OF CODES

A new system of normative documents for construction was put in power in 1995 in Russia. This new system establishes three levels of normative documents: 1 – Federal codes and standards (Building norms and regulations, State standards, Codes of practices); 2 – Regional codes; 3 - Standards of branches of industry (standard of enterprise, etc).

Based on the previous experience, the codes for bridge construction sector have to be reworked normally every 7-10 year period. However in the current practice a number of relevant design and construction codes have become obsolete. E.g. the Bridge code was issued in 1984. Due to that fact a fundamental revaluation is required, including researches and generalisation of national and foreign experience. Some of this work have commenced and further is currently under planning.

One of the first steps towards the renewal of bridge codes was the development of Regional standard TCH 32 {8}. This new regional standard was drafted in 1997. The main objective of this new standard is to reflect the specifics of bridge design in Moscow and to improve reliability and durability of bridge structures. Section related to foundations contains more hard terms (compared to the Bridge code) to concrete class, its frost resistance and watertightness. The draft TCH 32 was being studied and reviewed by the appropriate authorities and is expected to be revised during 1998 to take account of the comments.

In the light of codes renewal some general notes to improve the existing design procedure are introduced below.

For the design to limit states principles a system of coefficients have been established. These coefficients consider reliability on the basis of structures importance classification and working conditions. However the bridge comprises various structural elements which act a different role in the whole structure. When the ultimate state is reached by one of the elements, their failure may have different consequences. Therefore the reliability of elements have to be differentiated in the whole structure.

The recent study {9} of design requirements currently in use for determination of loads, having hydrologic and meteorological nature, has shown inadequacy of existing codes. This study have concentrated on the aspects related to temporary structures, however the main results are also applicable to permanent structures. E.g. the reliable functioning of temporary structures within the rivers demand hydrologic (hydraulic) justification. The worked out recommendations suggested to widen the existing range of the design flood return period in the directions of lower and higher probabilities of exceedance. Thus the design have to be elaborated in the range from a 100-year to 2 year return period. The choice of an optimum range of probability of exceedance have to consider importance classification of permanent structures and probability distribution of hydrologic characteristic.

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