CHAPTER 104

Dynamic Response of Vertical Structures to Breaking Wave Forces - Review of the CIS Design Experience-

J.G. Marinski[1]; H. Oumeraci[2]

Abstract

The necessity of a dynamic analysis and the dynamic approaches available in the CIS, formerly Soviet Union, for the stability of vertical structures subject to breaking wave impact loads are first briefly discussed. The different steps of the dynamic method recommended by the Russian Design Guidelines VNIIG-77 are presented. The results obtained by using the different static and dynamic methods for a numerical example are compared. Finally the effect of the nonlinear behaviour of the foundation of the structure under impact loads is discussed.

Introduction

A large experience is available in the CIS, formerly Soviet Union, on prototype measurements, hydraulic model investigations and dynamic analysis of vertical breakwaters. The stability of these structures has long been recognised as being a purely dynamic problem when subject to breaking wave impact loads. In this case, the widely accepted (particularly in Japan and western countries) static approaches using static loads and static stability analysis is not sufficient and should be supplemented or replaced by dynamic approaches.

It is the main objective of this paper to review and discuss the CIS design experience in this field. Emphasis will particularly be put on dynamic analysis, as compared to the commonly used static analysis.

Necessity of Dynamic Analysis

The failures experienced all over the world by vertical breakwaters have clearly shown that the traditional design approach (static stability analysis) can neither explain nor predict the most relevant failure modes and mechanisms observed in the field (OUMERACI et al., 1991).

Some of the further reasons for accounting for the effect of impulsive loading due to breaking waves in the stability analysis of vertical structures are given below.

In Fig. 1, wave loadings and accelerations of a caisson breakwater simultaneously measured in large-scale model tests are shown (OUMERACI et al., 1991).

[1] Dr.-Ing., Senior Researcher, Bulgarian Academy of Sciences, Institute of Water Problems, Sofia, Bulgaria

[2] Dr.-Ing., Senior Researcher, Franzius-Institut, University of Hannover, Germany

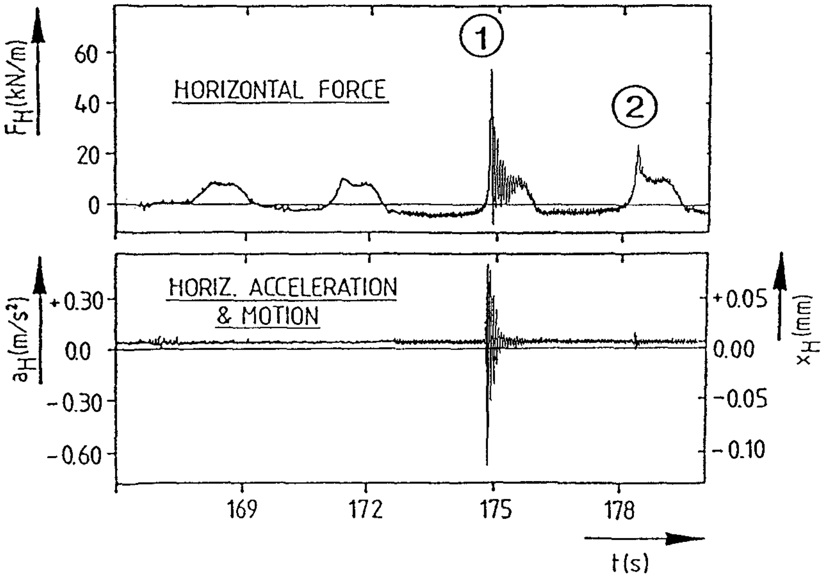

FIG. 1: EFFECT OF IMPACT FORCE ON CAISSON BREAKWATERS

By comparing the effect of impact force 1 and that of the quasi-static force 2 on the response of the structure, it is seen that the commonly suggested opinion that only quasi-static pressure forces are relevant for the stability of vertical structures cannot be confirmed.

On the other hand, the rocking motions of the caisson breakwater are transmitted to the rubble mound foundation and to the seabed which may result in an accumulation of irreversible deformations, and thus in the initiation of failure. These rocking motions are expected to be particularly high for breaking waves with large entrapped air pockets, since this generally results in force oscillations with periods in the range of the natural period of oscillations of the structure (OUMERACI et al, 1992).

Furthermore, the local impact pressures with high magnitude and relatively short duration may be important for the structural stability of the components of the structure in the impact zone.

Brief Review of Methods for Dynamic Analysis of Vertical Structures

In the CIS, the application of dynamic methods for the stability analysis of vertical breakwaters subject to breaking wave loads already started in the fifties (PETRASHEN, 1956).

Most of the methods developed in the CIS for the dynamic analysis of vertical breakwaters are generally based on a lumped parameter model of a rigid body on a homogenous, elastic and isotropic half space. The difference between the various methods mainly consists in the type of soil parameters used in the conceptual model.

By briefly reviewing the available literature in this field, three schools of thoughts appear to emerge which are represented by PETRASHEN, SMIRNOV and LOGINOV, respectively.

PETRASHEN (1956) was certainly the first to suggest dynamic methods for the stability analysis of vertical breakwaters. His first suggestion concerns a rigorous mathematical formulation which is difficult to apply to a practical problem. His second suggestion, however, was almost fully empirical (PETRASHEN, 1956). Since the latter was essentially based on the results of very small-scale model tests (empirical design diagrams), it was not accepted in the design practice.

In the model of SMIRNOV & MOROZ (1983), the vertical structure is considered as a rigid body with three degrees of freedom, and the elastic half space is described by the JOUNG Modulus E_S and POISSON's ratio ν of the foundation soil beneath the structure. This method has also found no acceptance in the design practice, although it generally leads to much larger stress and deformation in the soil than the static approach.

The model of LOGINOV (1962) is the only one which has been recommended for design practice by VNIIG-77. Since design guidelines and standards generally reflect to a great extent the state of the art in the related field and country, this method (called here "VNIIG Method") will be discussed below in more detail.

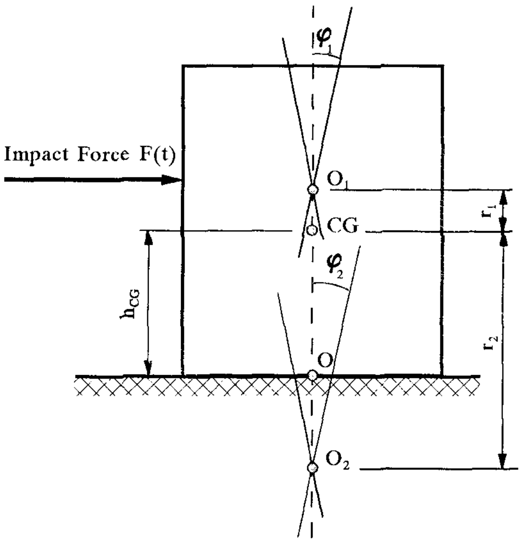

FIG. 2: DYNAMIC SYSTEM CONSIDERED BY VNIIG-77

VNIIG Method for Dynamic Analysis of Vertical Structures

As already mentioned, the dynamic analysis recommended in the Design Guidelines VNIIG (1977) is principally based on the method developed by LOGINOV (1962). The latter makes use of the large experience available in the field of the dynamics of maschine foundation, particularly the methods introduced by SAVINOV (1955) and BARKAN (1948). In these methods, the subsoil is considered as an elastic half space. This means that the structure on an elastic foundation described by coefficients of subgrade reaction according to SAVINOV (1955) exhibits horizontal and rocking motions. The vertical motions which is assumed to be uncoupled is not considered.

In addition, the damping is neglected in the equations of motion, since only the first maximum amplitude is considcred to be of interest for the stability of the structure. The dynamic system considered is given by Fig. 2, showing that the structure may rotate around point O_1 and O_2 located on the vertical axis through the centre of gravity C of the structure; i.e. the rotating and swaying motions have been replaced by the rocking motions around O_1 and O_2. The different steps of the procedure recommended by VNIIG (1977) are described below.

Step 1: Evaluation of Force Impulse and Load Durations

The typical impact pressure history is schematised in Fig. 3 where t_r is the rise time up to p_{max}, t_D the duration of the impact pressure and p_s the maximum quasi- static pressure.

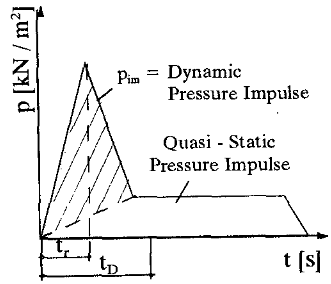

FIG. 3: TYPICAL PRESSURE HISTORY

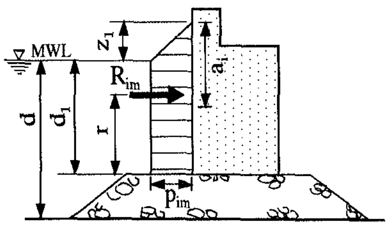

FIG. 4: PRESSURE IMPULSE DISTRIBUTION

The distribution of the pressure impulse on the structure front is given in Fig. 4 where:

$$z' = \left(0.55 d_3 + 0.1 H\right) \tag{1}$$

$$p_{im} = \frac{k_p}{\pi^2 \cdot g} \cdot \gamma_W \cdot a' \cdot v = 0.065\, a'\, v \tag{2}$$

with g = acceleration of gravity [m/s^2]
k_p = 6.17 empirical coefficient [-]
γ_W = specific weight of water [t/m^3]
v = velocity of the impinging wave [m/s] according to the following formula:

$$v = 1.2\sqrt{g \cdot d_3} \tag{3}$$

a' = height of the wave impact zone [m] which is defined by the relationship:

$$\frac{a'}{H} = 1.6 \tanh\left(\frac{2H}{d_3} - 1.34\right) \sin\frac{8\pi H}{L} = \leq 1.1 \tag{4}$$

H = wave height [m]
L = wave length [m]
d_3 = water depth at the wall [m]

The force impulse R_{im} [ts/m] is then obtained by:

$$R_{im} = k_a' \cdot p_{im}\left(d_2 + \frac{1}{2}z'\right) \tag{5}$$

where k_a' = empirical coefficient which accounts for the irregularity of the distribution of the pressure impulse along the wall (length l_C):

$$k_a' = \frac{k_a \cdot a' + 1.3\,k_a\left(d_z + z' - a'\right)}{d_2 + z'} \tag{6}$$

$$k_a = 0.55 + 0.15 \tanh\frac{H}{l_C} \tag{7}$$

l_C = length of the caisson [m]

The point of application of the resultant force impulse R_{im} is located at a distance r_{im}:

$$r_{im} = \frac{d_2^2 + z'd_2 + z'\cdot\frac{2}{3}}{2d_2 + z'} \tag{8}$$

from the caisson base (Fig. 4).

The relative rise time t_r and the relative impact duration t_D/T of the resultant force corresponding to the impulse R_{im} is obtained from Fig. 5 as a function of the relative depth d_3/a (T = wave period).

Step 2: Calculation of Natural Periods of Oscillations

The swaying and rotating motions of the structure are combined to give rocking motions around O_1 and O_2 located above and under the centre of gravity C, respectively (Fig. 2).

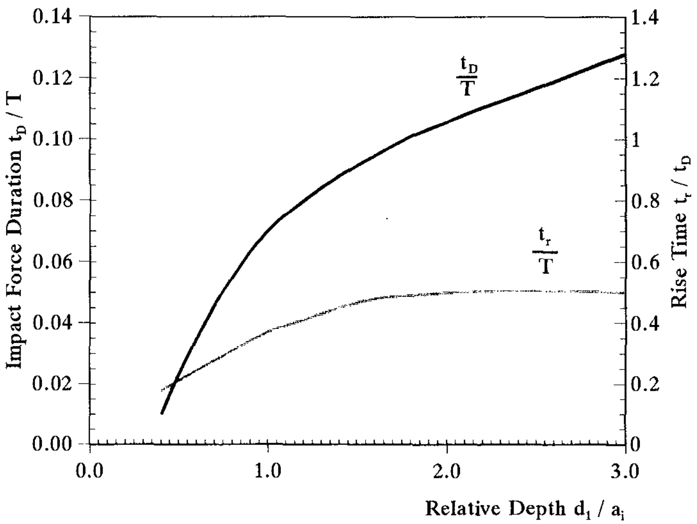

FIG. 5: IMPACT DURATION AND RISE TIME OF HORIZONTAL FORCE

The points O_1 and O_2, located at a distance r_1 and r_2 from the centre of gravity C, are determined according to the following formula:

$$r_{1,2} = \frac{h_0\ \omega_x^2}{\omega_x^2 - \omega_{1,2}^2} \tag{9}$$

where: h_0 = distance of the centre of gravity from the caisson base

$\omega_{1,2} = \frac{2\,\pi}{T_{N_{1,2}}}$ angular frequency of the free oscillation around O_1 and O_2, respectively [rad/s] which is determined by:

$$\omega_{1,2} = \frac{1}{2\,\vartheta}\left[\left(\omega_x^2 + \omega_\varphi^2\right) \pm \sqrt{\left(\omega_x^2 + \omega_\varphi^2\right)^2 - 4\,\vartheta\left(\omega_x^2 + \omega_\varphi^2\right)}\right] \tag{10}$$

where:

$$\vartheta = \frac{\theta_C}{\theta_0} \tag{11}$$

θ_C = mass moment of inertia around the centre of gravity C [tm^2]

θ_O = $\theta_C + mh_O^2$ = Mass moment of inertia around the centre of caisson base O [tm^2]

$$\omega_\varphi = \sqrt{\frac{C_\varphi \cdot I}{\theta_O}} \tag{12}$$

$$\omega_x = \sqrt{\frac{C_x \cdot A_f}{m}} \tag{13}$$

where: m = mass of the structure
A_f = $a \cdot l_C$ = Area of the caisson base [m^2]
l_C = length of the caisson [m]
I = moment of inertia of surface A_f [m^4]
ω_φ = natural angular frequency of rotation around centre of caisson base O [rad/s]
ω_x = natural angular frequency of swaying motion [rad/s]
C_x = coefficient of subgrade reaction for swaying motion [t/m^3] which is defined according to SAVINOV (1955):

$$C_x = 0.7 \cdot C_z \tag{14}$$

with

$$C_z = C_O \left[1 + 2\,\frac{a + l_C}{A_f}\right] \sqrt{\frac{W'}{2a}} \tag{15}$$

where: C_z = coefficient of subgrade reaction for vertical motion [t/m^3]
a = width of the caisson base [m]
l_C = length of the caisson [m]
W' = submerged weight of the caisson [t/m]
C_O = coefficient of subgradc reaction determined from field measurements or from Tab. 1 as a function of the thickness of the rubble mound foundation d_r, the width of the caisson base a and the type of the subsoil [t/m^3]
C_φ = coefficient of subgrade reaction for rotational motion [t/m^3] according to SAVINOV (1955):

$$C_\varphi = C_O \left[1 + 2\,\frac{a + 3l_C}{A_f}\right] \sqrt{\frac{W'}{2a}} \tag{16}$$

Step 3: Calculation of Maximum Amplitude of Oscillations

The angle of rotation around O_1 and O_2 are φ_1 and φ_2, respectively. These angles and the resulting horizontal motion δ at the base of the caisson are shown in Fig. 6.

$$\varphi_1 = \frac{2\,k_{d_1} \cdot M_{im,1}}{\vartheta_{O_1} \cdot \omega_1^2 \cdot t_D} \quad [\,rad\,] \tag{17}$$

	Characteristics of foundation	C_o [kN/m³]
1	· Rubble with small thickness $d_r/a \leq 0.25$-0.30 on sandy subsoil, silty clay, peaty clay, peat or very soft clay · on silty sand or very soft clay	1250 - 1500
2	· Rubble with small thickness $d_r/a = 0.25$-0.30 on sand or relatively stiff clay · Rubble with medium thickness $d_r/a = 0.35$-0.40 on soft soil (clay and sand)	2000 - 3000
3	· Rubble with medium thickness $d_r/a \approx 0.40$ on relatively compact subsoil (sand and clay)	2500 - 4000
4	· Rubble with large thickness $d_r/a \geq 0.45$ on subsoil with medium stiffness (sand and clay)	4000 - 6000
5	· Rubble mound with large thickness $d_r/a \geq 0.45$ on compact soil (gravel, compact sand, hard clay)	6000 - 8000
6	· Concrete bags or concrete blocks	11000 - 13000
7	· Rock	30000 - 50000

TAB. 1: EVALUATION OF COEFFICIENT OF SUBGRADE REACTION C_O

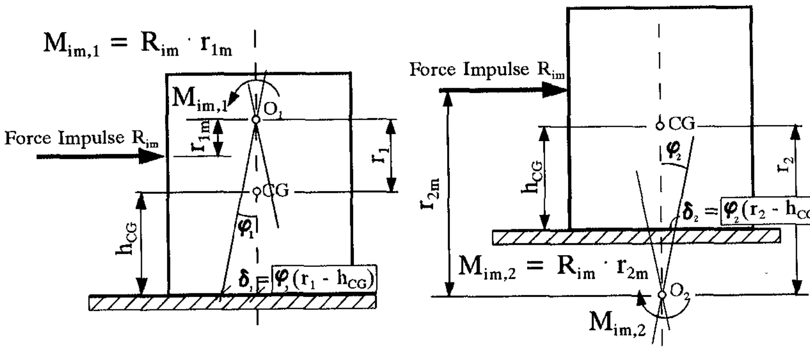

FIG. 6: DEFINITION OF ROTATION ANGLE φ_I AND φ_{II} AND DISPLACEMENT δ

$$\varphi_2 = \frac{2\,k_{d_2} \cdot M_{im,2}}{\vartheta_{O_2} \cdot \omega_2^2 \cdot t_D} \qquad [\,rad\,] \tag{18}$$

The horizontal motion at the base of the caisson is given by:

$$\delta = \varphi_2 (r_2 - h_o) - \varphi_1 (r_1 + h_o) \tag{19}$$

where $M_{im, 1}$, $M_{im, 2}$ = moment of the force impulse around O_1 and O_2, respectively [tsm/m]

The dynamic coefficient k_{d1} and k_{d2} are obtained from response curves like those shown in Fig. 7 as a function of the ratio $t_D/T_{N1,2}$ and t_r/t_D.

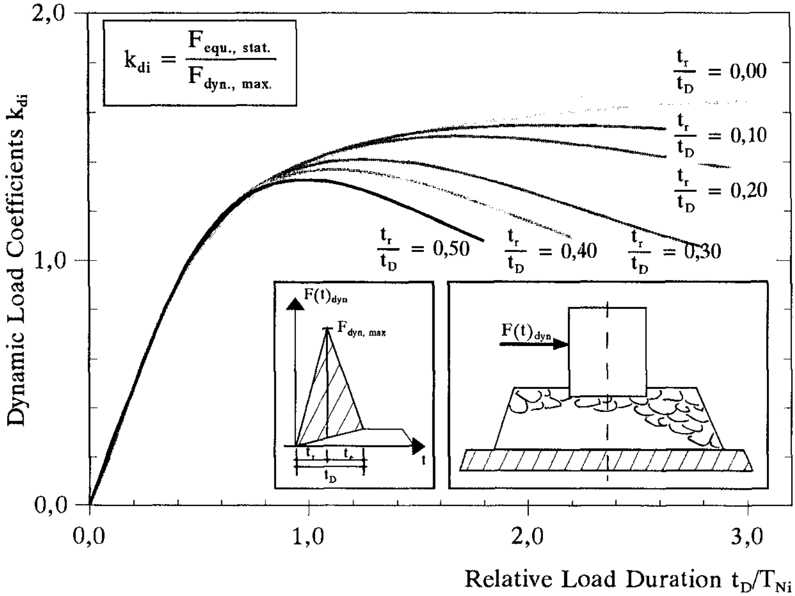

FIG. 7: RESPONSE CURVES AND DYNAMIC COEFFICIENT k_D

Step 4: Evaluation of Stability against Sliding

The safety coefficient η_{SL} against sliding of the caisson is given by:

$$\eta_{SL} = \frac{(W' - n R_u)\,\mu}{R_s + n R_u} \tag{20}$$

where μ ≈ 0.6 friction coefficient (concrete - rubble mound)
R_u = uplift force [t/m]
W' = submerged weight of the caisson [t/m]
R_s = shear resistance [t/m]:

$$R_s = C_x \cdot \delta \cdot a \tag{21}$$

$n = {}^{t_1}/_{T_N} \leq 1$

t_1 = time of occurrence of the maximum amplitude of motion [s], t_1 is obtained from Fig. 8 as a function of ${}^{t_D}/_{T_N}$ and ${}^{t_r}/_{t_D}$

t_D = impact force duration

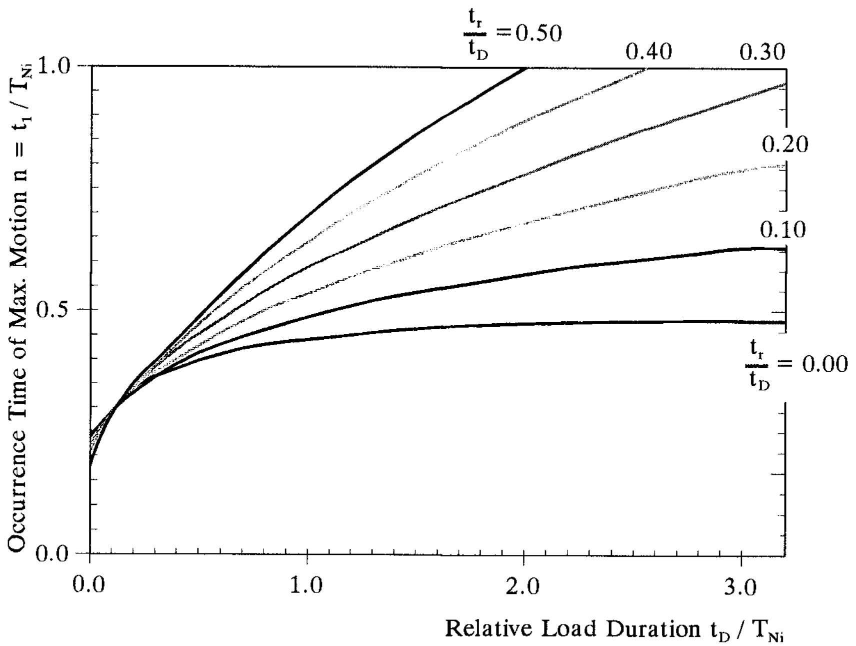

FIG. 8: TIME OF OCCURRENCE OF MAXIMUM OSCILLATION

Step 5: Evaluation of Maximum Normal Soil Stress

The normal soil stress induced by the oscillation of the caisson in the foundation is given by:

$$\sigma_{1,2} = \frac{W' - nR_u}{a} \pm \left[(\varphi_1 + \varphi_2) \cdot \frac{a}{2} \cdot C_\varphi + \frac{n \Sigma M}{W_r} \right] \tag{22}$$

where: W_r = $a^2/6$ [m^3]

$\sum M$ = moments around the centre O of the caisson base due to W', R_u and R_p

n = ${}^{t_1}/_{T_{Ni}}$ from Fig. 8

$\sigma_{1,2}$ = max. normal soil stress under the caisson edges (σ_1 = shoreward and σ_2 = seaward)

Discussion of the Methods

a) Comparison of Existing Standard Design Methods

In order to compare the existing standard methods for the analysis of the stability of vertical breakwaters, the numerical example and the structure shown in Fig. 9 are considered.

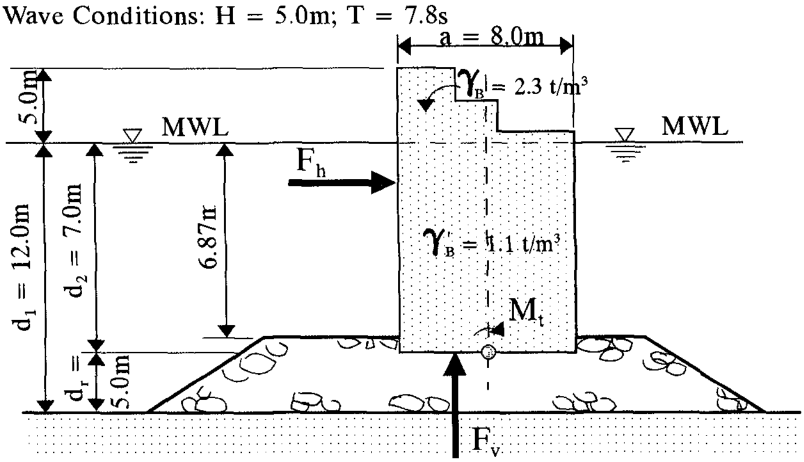

FIG. 9: STRUCTURE FOR COMPARISON OF STANDARD DESIGN METHODS

The results of the calculation by using the methods of SNIP-82 (static approach), VNIIG-77 (static and dynamic approach), PETRASHEN (dynamic approach) and GODA (static approach) are summarized in Tab. 2 showing that:

- GODA method appears to be more conservative than the existing standard methods in the CIS with respect to the bearing capacity of the rubble mound foundation.
- The static approach of SNIP-82 appears to be the most conservative method with respect to the stability against sliding.

b) Comparison of Linear and Nonlinear Calculation

A model which accounts for the nonlinear behaviour of the foundation of a vertical structure subject to breaking wave impact loads has been suggested by LOGINOV (1969). In order to compare the results obtained by this model and the dynamic approach of VNIIG-77, the structure and the numerical example shown in Fig. 10 are considered.

Method	SNIP-82 (1982)	VNIIG-77 (1977)	PETRASHEN (1956)	GODA (1974)
Dynamic Force [kN/m]	-	316	1045	-
Rise Time [s]	-	0.39	0.075	-
Static Force [kN/m]	610	496	620	495
Uplift Force [kN/m]	120	140	71.4	140
p_{max} [kN/m^2]	76.5	62.4	220.0	52.0
M_h [kNm/m]	3254	2730	-	3400
M_u [kNm/m]	640	746	380	746
M_t [kNm/m]	3894	3476	-	4166
η_{sl} [-]	1.07*)	1.30*) 1.58**)	1.10*)	1.30*)
η_{ov} [-]	1.24*)	1.39*)	-	1.17*)
σ_1 [kN/m^2]	509[1)]	320[1)]	-	646[1)]
σ_2 [kN/m^2]	-165	32	-	-380
φ [rad]	-	$1.69 \cdot 10^{-3}$	-	-
δ_{max} [mm]	-	3.5	-	-
T_N [s]	-	0.56 0.14	1.35	-

*) According to static analysis
**) According to dynamic analysis
1) Admissible stress 500 kN/m^2
η_{ov} = Safety coefficient for overturning stability

TAB. 2: RESULTS OF COMPARISON BETWEEN STANDARD DESIGN METHODS

Wave Conditions
H = 4.5 - 4.8m
L = 75m
T = 8s

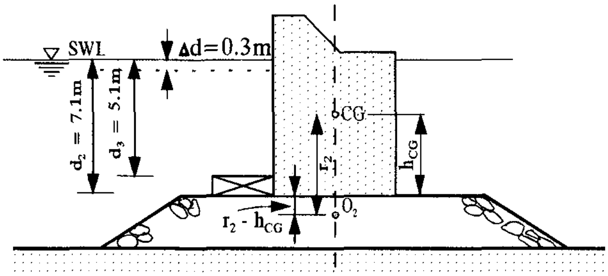

FIG. 10: STRUCTURE FOR COMPARISON OF LINEAR AND NONLINEAR CALCULATIONS

The results of this comparison are summarized in Tab. 3, showing that much larger amplitudes of oscillations of the structure (and thus larger soil deformations) and slightly larger periods of oscillations are obtained by using a linear model instead of a nonlinear one.

Description of Parameters		RESULTS		
		Prototype Measure-ments	Nonlinear Calculations (LOGINOV, 1969)	Linear Calculation (VNIIG 77)
R_{im}	kNs/m	130	102	102
p_{im}	kNs/m	14.5 - 16.5	15	15
$F_{h,max}$	kN/m	715	497.3	497.3
p_{max}	kN/m^2	160	62.4	62.4
$(r_2 - h_{CG})$	m	1.50	1.67	1.34
Periods of oscillations				
T_1	s	-	0.155 **)	0.158 **)
T_2	s	-	0.50 **)	0.55 **)
t_D	s	1.0	1.0	1.0
t_r	s	0.4	0.5	0.5
φ	rad	-	$0.541 \cdot 10^{-3}$ **)	$0.733 \cdot 10^{-3}$ **)
δ	mm	1.0 - 1.1	1.2 *)	1.63 *)

*) By using static calculations δ = 4 mm
**) linear description yields overestimated results for deformations (by +35%) and oscillation periods (by up to +10%) as compared to nonlinear description

TAB. 3: RESULTS OF LINEAR AND NONLINEAR CALCULATIONS

Concluding Remarks

As the stability of vertical breakwaters against sliding and the bearing capacity of the rubble mound foundation are concerned, GODA method and further standard static methods used in the CIS are more conservative than the dynamic approach recommended by the Russian Design Guidelines VNIIG-77. However, this so-called "dynamic approach" appears to have some limitations which may be due to a) uncertainties in the impact load characteristics used for the calculations and b) uncertainties of the measurement (prototype and model tests) of the structure motions used for model validation (low natural frequency of the accelerometers).

Furthermore, it is suggested that nonlinear behaviour of the foundation of vertical structures should be accounted for in the case of breaking wave impacts for which soil deformations larger than 0.1 mm are expected. However, the use of linear model appears to yield conservative results with respect to soil deformations.

Acknowledgements

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