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# DESTRUCTION OF ROCKFILL SLOPE PROTECTIONS BY ICE JAMMING: FIELD AND THEORETICAL STUDIES

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**Abstract:** The relevance of the study is due to the fact that the nature of failure of rockfill slope protections of hydraulic structures under ice loading during water-level fluctuations has not actually been described. At the same time, ice jamming caused by the freezing of the ice cover to the structure during water-level changes should be classified as a dangerous type of ice impact. Therefore, any data on the nature of structural damage and its parameters can contribute to improving the design principles and design substantiation of hydraulic structures. An analysis of failures performed on the drainage berm of the main dam of the Nizhny Novgorod HPP made it possible to thoroughly schematize the recorded defects. To determine the causes of the hydraulic structure failure, a set of stability calculations for the hydraulic structure slope was performed. The complexity of the calculations lay in the fact that they had to be carried out in such a way as to assess the actual stability deficit, i.e., to determine the ultimate pull-out moment as accurately as possible. A comparative analysis of field observation data and calculation results made it possible to establish both the contribution of various factors to the failure scheme of the hydraulic structure and the causes of failure. Convincing evidence is presented of the significant influence of the warming effect of groundwater (seepage water) on the failure of the hydraulic structure. The new data obtained should be taken into account to refine the existing calculation schemes for assessing the stability of slope protections of hydraulic structures in cases where groundwater may emerge onto the slope. Such refinement may be required for slopes of canals in cuts, dam slopes, flood-protection structures, or transport embankments extending into river channels.

**Keywords:** ice jamming, ice cover, slope failure, schematization of defects, shoreline crack

# РАЗРУШЕНИЕ ЛЕДОВЫМ ПРИПАЕМ КАМЕННО-НАБРОСНЫХ ОТКОСНЫХ КРЕПЛЕНИЙ: НАТУРНЫЕ И ТЕОРЕТИЧЕСКИЕ ИССЛЕДОВАНИЯ

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**Аннотация:** Актуальность рассматриваемой проблемы вызвана тем, что характер разрушения каменно-набросных откосных креплений гидросооружений ледовой нагрузкой при колебаниях уровня воды фактически не описан. Вместе с тем, ледовый припай, обусловленный примерзанием к сооружению ледяного покрова при изменении уровня воды, следует относить к опасным видам ледовых воздействий. Поэтому любые данные о характере повреждений конструкций и параметрах этого повреждения могут способствовать совершенствованию принципов проектирования и расчетного обоснования гидросооружений. Анализ разрушений, выполненный на дренажном банкете русловой плотины Нижегородской ГЭС, позволил детально схематизировать зафиксированные дефекты. Для выяснения причин разрушения гидросооружения был выполнен комплекс расчетов устойчивости откоса гидросооружения. Сложность выполнения расчетов заключалась в том, что их надо было выполнить так, чтобы оценить фактический дефицит устойчивости, т.е. наиболее точно определить предельный вырывающий момент. Сопоставительный анализ данных натурных наблюдений и результатов расчетов позволил установить как вклад различных факторов в схему разрушения гидросооружения, так и причины разрушения. Приведены убедительные доказательства существенного влияния отепляющего воздействия грунтовых (фильтрационных вод) на разрушение конструкции гидросооружения. Полученные новые данные необходимо учитывать для уточнения существующих расчетных схем по оценке устойчивости откосных креплений гидросооружений в случае возмож-

ности выхода грунтовых вод на откос. Такое уточнение может потребоваться для откосов каналов, проходящих в выемке, откосов плотин, противопаводковых гидросооружений или транспортных насыпей, заходящих в русло рек.

**Ключевые слова:** ледовый припай, ледяной покров, разрушение откоса, схематизация дефектов, береговая трещина

## INTRODUCTION

The practice of hydraulic structure operation involves the cases of slope protection failure caused by ice loads resulting from the freezing of the ice cover to the structure during water level changes. Such ice impact is commonly referred to as ice jamming, the mechanism of which is described in ICOLD Bulletin 105 and in reference [1]. In particular, when ice freezes to rockfill, this lifts the frozen mass as the water level rises. If the water level falls, the ice sheet, acting as a cantilever, displaces the rockfill protection from its design position.

Slopes of canals [2], flood protection embankments along riverbanks [3], bridge approach fills, upstream slopes of dams (e.g., during reservoir drawdown), as well as rock drainage berms in the tailwater of hydroelectric plants in wide sections where the influence of the polynya decreases [4], may be subjected to such loading. At the same time, the ice load caused by the rise/fall of the ice sheet (ice jamming) is the most dangerous among all possible ice impacts for rockfill [4].

Therefore, any field data concerning the destructive effect of ice jamming on hydraulic structure slopes will be valuable for researchers and designers.

For example, reference [5] examined the failure of a rock drainage berm slope of the Nizhny Novgorod HPP main dam. Let us briefly describe the studies previously conducted at the Nizhny Novgorod HPP.

Thus, the rock drainage berm of the Nizhny Novgorod HPP main dam required annual repairs after a two-month winter operation period. According to the original design, the structure has a length of 500 m, an average height of 7–8 m, a crest width of 5 m, and a slope of 1:2. The berm is formed by rockfill with an average stone diameter of 0.25 m. Figure 1 shows a typical cross-section based on depth measurements, illustrating both the reshaping pattern of the berm and its design profile (indicated by a dashed line).

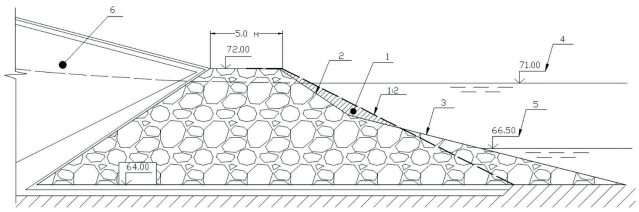



*Figure 1. Scheme of transformation of a slope composed of rockfill under the action of ice jamming: 1) slope failure; 2) reshaping of the upper part of the slope forming a steep slope; 3) reshaping of the lower part of the slope forming a gentle slope; 4) maximum ice-cover level; 5) minimum water and ice-cover level; 6) dam body*

The layout features of the Nizhny Novgorod hydropower complex and its operation exclude any other ice impacts on the structure except for ice jamming. Firstly, the rock berm of the main dam is located at a considerable distance (at least 1 km) from the spillway and the power house, and thus from the polynya. Secondly, no ice is released through the dam.

Evidence that the ice edge approaches the drainage berm (usually in January–February) comes from operation service records and aerial ice surveys [6]. According to these data, the ice thickness in the tailwater near the drainage berm during severe winters does not exceed 35 cm. At the same time, in the lake-like part of the reservoir (near the dam), the ice thickness is 50–60 cm, which is less than in other Volga reservoirs. The modest ice thickness is due to the warming discharges from the Kostroma GRES. Data on anthropogenic thermal pollution and ice thicknesses were obtained from two different sources over different periods: [6] and [7].

Complete ice clearance occurs in April. The formation of ice cover in the tailwater of the Nizhny Novgorod HPP is accompanied by stable frosty weather, low discharges, and constant HPP output throughout the day [7].

Depending on the air temperature and discharges into the tailwater, the ice edge may approach and retreat from the structure several times during one winter. At the same time, water-level fluctuations in the tailwater of the Nizhny Novgorod hydroelectric complex during the day can have a wide amplitude depending on the HPP discharges: from 0 to 2 m [7].

Note that Figure 1 presents an averaged assessment of damage. In reality, the observed parameters vary over a fairly wide range of values:

- the depth of slope failure ranges from 0.44 to 1.18 m (measured perpendicular to the slope);
- the slope steepness in the upper part varies from 1:0.73 to 1:1.77, and in the lower part – from 1:2.78 to 1:5.22;
- the point where the steep upper part transitions into the gentle lower part is located about 2 m from the maximum water level.

There is also a tendency: the steeper the upper part of the slope, the flatter the values of the lower part. In this case, the lower contour of the slope, with an average slope of 1:4, is no longer subjected to failure.

The large scatter of parameters (slope angles and failure depth) is most likely due to the complex thermal regime. Among the factors considered are the influence of snow cover and the warming effect of the reservoir. In particular, relatively warm seepage waters can reduce the weight of the frozen rockfill, which is a resisting force, and shift the center of rotation of the frozen mass towards the shoreline, i.e., reduce the leverage arms of the resisting forces.

The presented observations are important for practice because this allowed recording the actual parameters of the failure of a slope composed of rockfill.

Nevertheless, the cause of the destruction of the hydraulic structure is not obvious. To refine the calculation schemes for assessing the stability of slopes under ice jamming and to improve design methods for hydraulic structures, the following tasks need to be addressed:

- establish the causes of the unfavorable condition of the structure and assess the contribution of various factors to the failure process;
- determine the stability deficit of the failed rockfill embankment;
- schematize/systematize the nature of the failures;
- develop recommendations for improving design solutions.

## METHODS

Perform the stability analysis for the failed slope of the hydraulic structure. In this way, we will seek the actual stability deficit, i.e., we will not include various safety factors.

To determine the moment load most accurately, we will assess stability not on the basis of the ice strength itself, as is common practice, but differently. In particular, we use the expression for the bending moment at the fixed end $M$ (the moment load), which is derived from the differential equation of the deflected axis of the ice cover:

$$E \cdot I \frac{d^2y}{dx^2} = -M_x \qquad (1)$$

Where $E$ is the elastic modulus of the ice cover, $I$ is the moment of inertia of the ice-cover cross-section, and $M_x$ is the bending moment at any section at a distance $x$ from the fixed end (the ice jam). The origin is taken at the ice-jam location, with the $oy$ axis directed downward and $ox$ to the right.

The most accurate solution to equation (1) was proposed by P.A. Shankin, who assumed that the deformation pattern of the ice cover during water-level fluctuations allows a correct determination of the vertical forces, bending moments, and the length $l$ of the deflecting ice sheet [8]. We present the expressions for the bending moment at the fixed end M and the length $l$ of the ice cover over which the load is transmitted to the structure:

$$l = 3{,}58\sqrt[4]{\frac{E \cdot I}{\gamma C}}. \qquad (2)$$

$$M = f\frac{1}{60 \cdot l^2}\{\gamma C(5\psi - 2) + 360 \cdot E \cdot I\}. \quad (3)$$

Where γ is the unit weight of water; *C* is the length of the structure along the front at the ice action level; the numerical coefficient in formula (2) is obtained for the ratio of the water-level rise/fall *H* to the deflection *f* equal to 1 (i.e., $H/f = \psi = 1$).

The bending moment at the fixed end *M* can also be called the ultimate pull-out moment acting in section *x* (Fig. 2). In this section, the first shoreline crack appears in the ice cover.

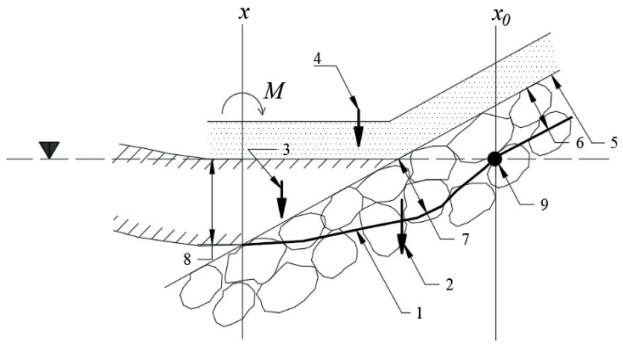



*Figure 2. Force diagram for the stability analysis of a slope composed of rockfill during ice-cover rise: 1 – freezing boundary of the soil; 2 – weight of frozen soil; 3 – weight of ice on the slope; 4 – weight of snow cover; 5 – slope surface; 6 – freezing depth of rockfill above the water level ($H_1$); 7 – freezing depth of rockfill at the water level ($H_2$); 8 – freezing depth of rockfill below the water level ($H_3$); 9 – center of rotation of the frozen rockfill mass*

The length *l* of the ice cover corresponds to the point where the curvature of the ice field transitions to a horizontal position, i.e., when the section rotation angle θ = 0 and the deflection is maximum *f*. The deflection *f* is assigned according to field observation data obtained by geodetic levelling of the ice field under similar ice thickness [8].

For an ice thickness $t = 0.35$ m, $E = 400{,}000$ t/m$^2$ (3922.66 MPa) [9-15]; $I = 0.00357$ m$^4$, $\psi \approx 1$, and $f = 0.025$ m, the length of the ice cover from formula (2) is $l = 22.0$ m, and the pull-out moment from formula (3) is M = 0.44 tf·m (4.31 kN·m).

The applicability of expressions (2) and (3) is limited by the following conditions:

- $B \geq 2l$ (*B* is the width of the water body at the design water level);
- $(\gamma_л \cdot t)/\gamma \geq H \geq f$ ($\gamma_л$ is the unit weight of ice, *t* is the ice thickness).

All the above considerations remain valid for both water-level drawdown and rise, except that during a rise, the overturning moment is clockwise and the vertical force is upward; during a drawdown, the opposite occurs.

Verification stability calculations of the rock berm during water-level rise (as the most dangerous case) will be performed for the actually recorded ice thickness in a severe winter (35 cm). We will also assume different values of snow-cover height δ.

The snow-cover height δ significantly affects the freezing depth of the soil above and below the water level [16], and consequently the magnitude of the resisting forces (weight of the frozen rockfill) and the position of the centre of the frozen mass. In the absence of snow cover and without the warming effect of groundwater, the resisting forces will be maximal. According to operation service data, the maximum average ten-day snow-cover height over the winter does not exceed 53 cm. The maximum snow depths are observed at the end of March, when the ice-jam hazard is already absent.

Evaluate the slope stability for different design cases: for slope angles in the upper part (1:2) and lower part (1:4) and for various snow-cover thicknesses.

In the case of water-level rise, all moments resisting the rockfill are taken entirely within sections $x - x_0$ (Fig. 2), assuming that the frozen rockfill mass does not deform.

To assess slope stability, it is necessary to estimate the freezing depths of the soil: above the water level $H_1$, at the water level $H_2$, and below the water level $H_3$. For the considered calculation scheme (Fig. 2), the values of $H_1$, $H_2$, and $H_3$ can be taken from the formulas:

$$H_1 = H_d - \frac{\delta_s}{\beta_s}\beta_{st1}, \quad (4)$$

$$H_2 = H_d - \frac{\delta_s}{\beta_s}\beta_{st2}, \quad (5)$$

$$H_3 = t. \quad (6)$$

Where $H_d$ is the freezing depth of the rockfill; $\delta_s$ is the thickness of the protective snow medium; $\beta_s$ is the thermal conductivity of snow; $\beta_{st1}$ is the thermal conductivity of the rockfill in the dry state; $\beta_{st2}$ is the thermal conductivity of the rockfill below the water level in the freezing zone; $t$ is the ice thickness.

In determining the weight of the frozen rockfill, the following was taken into account:

- at depth $H_1$, the unit weight of frozen soil containing an unaccounted amount of ice in the pores, with the buoyant effect of water ($\gamma_1 \approx 18.1$ kN/m$^3$);
- at depth $H_2$, the unit weight of frozen soil including the weight of ice in the pores, with the buoyant effect of water ($\gamma_2 \approx 19.1$ kN/m$^3$).

## RESULTS AND DISCUSSION

Example of stability calculation for a slope with design slope angle of 1:2 at a snow depth of 25 cm is given in Table 1.

*Table 1. Stability calculation of a rockfill slope (1:2) under ice-jamming action (ice thickness 35 cm, snow thickness 25 cm, $H_1$ = 0.16 m; $H_2$ = 0.25 m; $H_3$ = 0.35 m)*

| Forces, tf (kN) | Lever arms, m | Resisting moment, tf·m (kN·m) | $K=\Sigma(M_r)_{x0}/M$ |
|---|---|---|---|
| Weight of snow on ice sheet: 0.36 (3.53) | 0.58 | 0.21 (2.06) | 4.31/4.31=1 |
| Weight of snow on slope: 0.07 (0.69) | 0.13 | 0.01 (0.1) | |
| Weight of shore ice: 0.11 (1.08) | 0.82 | 0.09 (0.88) | |
| Weight of frozen rockfill below water level: 0.26 (2.55) | 0.46 | 0.12 (0.18) | |
| Weight of frozen rockfill above water level: 0.06 (0.59) | 0.12 | 0.01 (0.1) | |
| | $\Sigma(M_r)_{x0}$ | 0.44 (4.31) | |

The results of stability calculations for the slope with design slope angle 1:2 at different snow thicknesses are presented in Table 2.

*Table 2. Stability calculation results for the rockfill slope (1:2) at different snow thicknesses*

| Parameter | Snow depth, cm | | |
|---|---|---|---|
| | 20 | 25 | 30 |
| Calculated stability safety factor | 2.77 | 1.00 | 0.48 |
| Weight of frozen rockfill, tf (kN) | 0.97 (9.51) | 0.32 (3.14) | 0.19 (1.86) |

Analyze the results of the calculations and the field observation data. If we do not take into account the warming effect of seepage water, then at a snow thickness of 25 cm the slope with design slope 1:2 should be stable. Thus, failure of the 1:2 slope can occur only when the snow cover depth exceeds 25 cm, i.e., by the middle or end of winter. This is indirect, albeit incomplete, evidence of the influence of the warming effect of reservoir seepage waters on the position of the center of the frozen mass and its weight.

A snow cover thickness of 30 cm creates the most unfavorable conditions for the stability of the slope with design slope 1:2, because a 30-cm snow "cap" protects the slope from freezing. At this snow depth (30 cm), the safety factor reaches its minimum value because the weight of the frozen rockfill under the warming snow cap is negligible. A thicker snow cover would lead to a slight increase in the safety factor due to the additional snow surcharge on the ice sheet and slope.

With a snow cap of 30 cm or more, the slope with design slope 1:2 will fail, but the failure parameters should be identical along the structure length if there are no other factors affecting the collapse. On the contrary, the nature of failure is complex: the actual values of the stone mass displaced from its design position range from 6600 kg (maximum) to 1000 kg (minimum). This means that the failure of the slope with design slope 1:2 is caused by the influence of the seepage flow, which creates a complex temperature field.

Consider the unlikely situation that the snow thickness varies significantly along the slope

length, i.e., it is not uniform in different cross-sections:
1) from 0 to 50 cm;
2) from 30 to 50 cm;
The first case is not confirmed by field observations, because on some sections the structure would not be damaged. In the second case, the failure parameters would be identical, since a snow cap larger than 30 cm no longer affects the position of the soil freezing boundary.
Thus, a second reliable proof of the warming effect of reservoir seepage waters on the position of the centre of the frozen mass and its weight is presented.
The thermal regime of the seepage flow in the dam body cannot but be determined by the water temperature in the reservoir. According to field observations conducted by the operation service, the following is known [17]:
- during the freeze-up period, the water temperature near the bottom in the lower part of the reservoir (near the dam) in November and December is 1–2 °C, and by March it drops to 0.1–0.2 °C;
- in summer, the lake-like part of the reservoir is characterised by a decrease in water temperature from the surface to the bottom; the temperature drop is usually 2–3 °C, but can reach 10–15 °C;
- if the maximum temperature of the seepage flow in the upstream wedge of the dam (up to its axis) is observed in September, then in the downstream wedge (drainage berm), the groundwater has its maximum temperature in December.
The influence of warm groundwater on ice processes has been noted earlier at weather stations, in particular at the Ilyino post (Gorky Reservoir). There, the ice thickness is smaller than at other weather stations on the reservoir. Moreover, the peculiarity of this phenomenon lies in the non-uniformity of ice thickness: even in severe winters, the thickness can vary from 10 to 75 cm (data from the operation service of the Nizhny Novgorod HPP [17]).
The lower part of the rockfill slope with a slope of 1:4 is stable with a good margin even in the presence of snow cover. The stability of such a slope is largely determined by the weight of the shore ice and the significant lever arm of the corresponding force, even if the center of rotation of the frozen mass is for some reason shifted toward the shoreline.
The performed field observations of the failure of the rockfill slope, accompanied by calculations, show that there are two effective ways to combat ice jamming [18–20]: flattening the slope and increasing the stone size. The latter measure is only effective under the warming influence of snow cover or groundwater, because without these factors, the weight of the frozen rockfill, rather than the weight of an individual stone, will be one of the forces ensuring the stability of the slope protection.
The slope angle of the non-failed and stable lower part of the slope is apparently determined by the nature of the failure of the upper part, which, after reshaping, rolls down to the lower part of the structure. Thus, it has been observed that the greater the failure depth and the steeper the upper slope, the flatter the slope in its lower part becomes.

## CONCLUSIONS

1. One of the reasons of rockfill embankment failure under ice-jamming action is the warming effect of groundwater (seepage) flow. This fact should be considered proven for the following reasons:
- according to calculation results, failure of the slope with design slope 1:2 can occur only when the snow cover depth exceeds 25 cm, i.e., by the middle or end of winter, which does not correspond to field observation data;
- calculation results show that if the slope failure were determined solely by the height of the snow cap, its failure parameters would be identical even if the snow thickness/depth varied along the slope length;
- field observations of the failed slope show a wide scatter in slope angles and in the mass of stone displaced from its design position to the lower part of the slope, which is due to the com-

plex thermal regime within the hydraulic structure (the mass of stone displaced during two winter months from the design position varies from 1000 kg to 6600 kg).

2. Systematic observations of the failure of a rockfill embankment under ice-jamming action, together with the stability calculations performed, allow the following conclusions:

- the failure pattern of the design slope with slope 1:2 consists of a steep upper part (reaching 1:0.73) and a gentle lower part (averaging 1:4);
- the gentle slope of the lower, non-failed part of the slope is most likely determined by the extent of failure in the upper part, from which stone is moved to the lower part, forming a new gentle surface;
- under the most unfavorable scenario, a twofold stability deficit of the rockfill slope with design slope 1:2 is observed;
- the existing calculation schemes for assessing the stability of slope protections need to be adjusted and, in particular, to account for the warming effect of groundwater when it emerges onto the slope;
- effective and reliable design measures for slope protections under ice-jamming hazard include flattening the slopes (e.g., to 1:4) and increasing the stone size.

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