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REINFORCED CONCRETE CHECK DAMS

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Summary

A methodology for the design of angle-shaped reinforced concrete check dams has been worked out. It aims at optimizing the external geometrical dimensioning both under different working conditions and for various strength characteristics of construction materials. This methodology is supported by an automatic procedure and performs the dimensioning for weirs of given heights, of all possible foundation designs. A check dam is designed which employs minimum volumes of reinforced concrete for a required external stability degree. At the same time, the most stressed inner sections work as tensions approach the maximum values allowed for concrete and steel. In addition, results point out some interesting criteria as to the most suitable design depending on external forces, namely: hydrostatic pressure, hydrostatic pressure and uplift, mixed water-ground thrust, debris-flow thrust.

Riassunto

Viene presentata una metodologia per la progettazione delle briglie angolari in cemento armato e per l'ottimizzazione delle dimensioni esterne sia nei riguardi di differenti condizioni di esercizio che delle caratteristiche dei materiali impiegati. La metodologia è supportata da una procedura di calcolo automatico e fornisce, per differenti altezze dell'opera, tutte le possibili configurazioni della fondazione. Si ottiene così una briglia che impiega i minimi volumi costruttivi per il grado di stabilità esterno richiesto ed in cui al contempo le sezioni più sollecitate lavorano con tensioni prossime alle massime consentite per calcestruzzo ed acciaio. I risultati forniscono inoltre i criteri di progettazione più convenienti in relazione alle forze esterne considerate: spinta idrostatica e sottospinta, spinta mista terreno-acqua, spinta da debris flow.

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1. Introduction

Self-standing reinforced concrete check dams are now largely used as mountain torrent control works. Their construction profile is overturned dissymmetric T-shaped or also simply L-shaped and allows to make use of the water or soil weight over the upstream projection as a stabilizing strength. The result is a rather slim structure and a considerable concrete saving if compared with the common solid gravity dam.

The reliability of this work has suggested to define a procedure of automatic computation with the double purpose of optimizing the check dam design for usual conditions and to evaluate if this structure can be designed so properly as to resist debris-flow impacts.

2. Schematic description of the computation procedure

The self-standing check dam taken into consideration (fig. 1) favours the most easily feasible building technique. It requires a vertical bracket with parallel upstream and downstream faces and a constant thickness of the ground support base.

The design is performed according to a bidimensional static scheme of the work and it considers a piece of unit length directly in the notch.

Fig.1 - Diagram of the bracket check dam and symbols used
After firstly assigning: the types of steel and of concrete, soil carrying capacity, friction coefficient of soil with foundation, storage height, overflow height, and eventually the kind of external forces, the computation is continued determining, in the order:

a) bracket thickness (b): search for it is made in such a way that the section of combined compressive and bending maximum stress (anchoring section at the foundation base) shall be the most suitable for the external forces taken into account. Optimum thickness is the one which allows to reinforce the concrete section so properly that maximum compression and traction stresses may be both close to the ones admitted for the materials used. This thickness is called "normal height" for the section. When concrete compression stresses are inferior to those admissible while steel traction stresses are next to the allowable ones, normal height is exceeded and thickness can be decreased. Conversely, if it is concrete that works close to the maximum admissible, while steel works below the allowed limit, section height must be increased. In any case, the thickness measure is never set inferior to 0.6 m in order to provide the top crest with a sufficient safety of resistance against possible impacts by coarse sediment flow.

b) foundation base height, through the relation:

$$z_f = 1.1 \, b$$

this figure is suggested by the necessity to warrant a proper bracket anchoring. It can be increased in the case the most stressed deflected section is inferior to "normal height" or when it is necessary to make the check dam heavier to meet the controls of external stability (slip and upsetting).

c) All the designs of the base with upsetting safety coefficient equal to 1.5; the possible designs are represented by two dimensionless ratios:

- relative foundation width \( J \): \( J = l_f/z \);
- design parameter \( K \): \( K = (l_m-l_v)/(l_m+l_v) \); with \(-1 \leq K \leq 1\).

For \( K = -1 \), an unusual design is found where the base projection is given only downstream; for \( K = 0 \), the check dam is symmetric T-shaped and for \( K = 1 \) it is only L-shaped (upstream projection).

d) The equilibrium design chosen out of those previously determined, having the shortest total length of the base (\( J = J_{\text{min}} \)).
e) The possible increase of the downstream projection length, in the case the maximum stress transmitted to the ground exceeds the assigned carrying capacity.

f) Moments and shears acting in the anchoring sections of the bracket and of the upstream and downstream projections. The distribution of ground stresses normal to the foundation is supposed to be linear.

g) Minimum necessary longitudinal and shear reinforcements.

3. Results of the computation procedure

As it can be seen from the proposed computation procedure, the external designing of the check dam and the determination of reinforcements do not occur in two distinct stages, but the resistance of materials to be used is already accounted for external designing.

In order to provide general information and, at the same time, analytical relations for a ready determination of optimum dimensions, only those results are illustrated which were obtained taking into account steel rods with an admissible stress of 1900 kp/cm² and concrete with maximum work stresses of 60 kp/cm² or 85 kp/cm².

Besides, the ground was supposed always to support transmitted stresses in order to prevent these from hampering the work designing.

In fact, in this case, the optimum design chosen (point d, previous paragraph) would be partly modified by the computation procedure adopted (increase of downstream projection length).

3.1 Hydrostatic thrust and possible underpressure

The determination of the optimum thickness for the vertical bracket in case of hydrostatic thrust does not depend on the presence of underpressure. The semi-empirical formula proposed by FATTORELLI – MAZZALAI (1985):

\[ b = 0.2(z+h) \]

can be improved taking into account the resistance characteristics of the materials. The pattern of "normal heights" (with 0.04 m concrete thickness between exterior and iron) for the two concrete working stresses examined (fig. 2) shows that the minimum value of 0.6 m is sufficient for check dam heights of about 4 m for maximum compression stress of 60 kp/cm² and of 5 m for stress of 85 kp/cm².

The interpolating formulae obtained are expressed by the following linear relations:
- compression stress of 60 kp/cm²

\[
\begin{align*}
\text{h = 1 m} & \rightarrow b = 0.208 z - 0.35 \quad \text{for } z \geq 5 \text{ m;} \\
\text{h = 2 m} & \rightarrow b = 0.216 z - 0.24 \quad \text{for } z \geq 4 \text{ m;} \\
& \quad b = 0.6 \text{ m} \quad \text{for } z \leq 3 \text{ m.; }
\end{align*}
\]

- compression stress of 85 kp/cm²

\[
\begin{align*}
\text{h = 1 m} & \rightarrow b = 0.160 z - 0.29 \quad \text{for } z \geq 6 \text{ m;} \\
& \quad b = 0.6 \text{ m} \quad \text{for } z \leq 5 \text{ m.; }
\end{align*}
\]

\[
\begin{align*}
\text{h = 2 m} & \rightarrow b = 0.165 z - 0.21 \quad \text{for } z \geq 5 \text{ m;} \\
& \quad b = 0.6 \text{ m} \quad \text{for } z \leq 4 \text{ m.}
\end{align*}
\]

Fig. 2 - Thickness b required for the bracket as a function of height z: hydrostatic thrust, steel with admissible stress of 1800 kp/cm² and concrete with admissible stresses of 60 or 85 kp/cm².

Instead, as regards the pattern of parameter J depending on the designing ratio K, curves were obtained which are defined "stability curves" (figs. 3-4). In fact, these represent all the designs which secure the check dam an upsetting safety coefficient equal to 1.5.
Fig. 3 - Stability curves for a check dam having a height \( z = 4 \) m and subject to hydrostatic thrust.

Fig. 4 - Stability curves for check dams with different heights \( (z = 2\text{ to } 7 \) m) where \( h = 1 \) m and subject to hydrostatic thrust.
These curves remain in minimum J values in the range:

\[ 0.6 \leq K_{\text{optimum}} \leq 0.8 \]

that is

\[ 4 \leq \frac{l_f}{l_f} \leq 9 \]

Within this range, the lengths of the base slab \((l_f)\) undergo very slight variations and they can be approximated with the (in any case precautionary) expression:

\[ l_f = 0.72 \, z + 0.55 \, h \]

which was obtained through the replicated implementation of the automatic computation procedure. If also an underpressure is supposed to act below the foundation slab with a maximum pressure value equal to 50\% of the upstream pressure, the base designing formula becomes:

\[ l_f = 0.93 \, z + 0.67 \, h \]

and the range of optimum \(K\) values remains the one already obtained for the only hydrostatic thrust.

Slip control does not affect designing for the ordinary values of friction coefficient between ground and foundation \((0.70 + 0.75)\); slip safety coefficient always exceeds 1.2 (for optimum designing).

Maximum stresses trasmitted to the ground, \(\sigma_{t,\text{max}}\), can be determined with some approximation and only for hydrostatic thrust through the interpolating formulae:

\[ \begin{align*} 
  h = 1 \, \text{m} & \rightarrow \sigma_{t,\text{max}} = 0.49 \, z + 0.8; \\
  h = 2 \, \text{m} & \rightarrow \sigma_{t,\text{max}} = 0.53 \, z + 0.7; 
\end{align*} \]

where \(\sigma_{t,\text{max}}\) is expressed in \([\text{kp/cm}^2]\) and \(z\) in \([\text{m}]\).

As regards the distribution of the principal longitudinal reinforcements, those of the bracket anchoring section are the greatest, the ones of the upstream projection (upper face) anchoring section are reduced by 10\% on an average, while the reinforcement for the downstream projection (lower face) anchoring section never exceeds 25\% of the same. However, if in addition to hydrostatic thrust also underpression acts (with a maximum pressure of 50\% of the upstream pressure), then the reinforcement of the upstream projection is increased by + 10\% with respect to the one necessary for the bracket.

3.2 Soil thrust

By computing soil pressure behind the check dam after Rankine's theory, it was possible to find that - for up to 50\% siltation \((z_a=z_t)\) - it is profitable on an average to design the check dam for:
0.7 ≤ K_{optimum} ≤ 0.8

where K values = 0.8 are obtained for a 50% siltation. For a self-standing 50% filled check dam, bracket thickness and foundation base width could on an average be reduced by 22% with respect to the values required for mere hydrostatic thrust conditions.
If siltation is increased any more (≥60%), optimum designing is always the simple L one (K=1) without the downstream projection. The choice of this last design, which proves also to make construction easier, is supported by the results obtained for the only hydrostatic thrust, where stability curves (fig. 3) for K=1 deviate very little from optimum conditions (K=K_{optimum}).

4. Structure for debris-flow

Most debris-break check dams built in these latest years are found to be rather bulky and to have got a structure which poorly matches with the mountain landscape where they are located, under the environmental point of view. Thus, it may be suitable to realize self-standing check dams which can withstand debris-flow, though they are not specific debris-break works.
Use was made of the procedure described for designing an angle-shaped solid wall check dam invested by debris flow having a specific weight of 1800 kp m^{-3}, the pressure of which was amplified by 7 times that of the hydrostatic thrust. In fact, this magnifying coefficient proved to be the greatest admissible for the design of this type of check dams; above this value, concrete volumes become too big for works to be realized in a mountain torrent. The extent of involved stresses suggests to take into consideration only the use of concrete with possible stress of 97.5 kp/cm² and of steel with maximum bearable stress of 2200 kp/cm².
As further assumptions, debris-flow was supposed to invest the not yet filled (z_f=0) check dam and to overflow the crest by the height of 1 m.
The friction coefficient between ground and foundation was assumed to be unitary (it can be easily reached providing some offsets under the foundation slab). In fact, a lower value is a rather heavy constraint and increases base height (z_f) too much.
The designs found are all simple L shaped, and stability curves actually point out the range of optimum values for K:

0.8 ≤ K_{optimum} ≤ 1.0

Unlike what happens for hydrostatic thrust, the foundation height (preliminarily computed with the relation: z_f = 1.1 b) is increased to some extent, since this value turns out to be inferior to "normal height" in the flexure
computation of the base upstream projection anchoring section.
Based upon these planning assumptions for designing the work, the automatic computation led to the following formulae of external designing (with K=1):

\[
\begin{align*}
    b &= 0.307 z - 0.25; \text{ for } 3 \text{ m} \leq z \leq 6 \text{ m}; \\
    b &= 0.65 \text{ m}; \quad \text{ for } z < 3 \text{ m}; \\
    z_f &= 0.380 z - 0.32; \text{ for } 3 \text{ m} \leq z \leq 6 \text{ m}; \\
    z_f &= 0.80 \text{ m}; \quad \text{ for } z < 3 \text{ m}; \\
    l_f &= 1.71 z + 0.82; \quad \text{ for } z \leq 6 \text{ m};
\end{align*}
\]

These relations secure upsetting and slip safety coefficients which are exactly equal to 1.5 and 1.0, while maximum stresses transmitted to the ground show a pattern which increases almost linearly with height:

\[\sigma_{t,\text{max}} = 1.08 z + 0.64; \text{ for } z \leq 6 \text{ m};\]
with \(\sigma_{t,\text{max}}\) expressed in [kp/cm²] and \(z\) in [m].

It is not advisable, in the case of debris flow pressure, to extend the desining of this type of check dams above 5.5±6.0 m, since ground-transmitted stresses become too strong and their reduction requires a base slab which is out of proportion to bracket height.

5. Conclusions

The computation options taken into examination have allowed to define some interesting designing standards for a quick design of the angle-shaped check dam under different operation conditions.

1) The dissymmetric T profile statically proves to be the most advantageous in the case of hydrostatic thrust and also of possible underpressure. In addition, it leaves some choice to the designer as regards the ratio between the lengths of the foundation base upstream and downstream projections. Actually, a wide range of ratios was determined where the volume required for the base keeps within minimum values when a same volume for the bracket is given.

2) In the case of thrust and if the work height does not exceed 4 m, no appreciable material volume saving occurs when concrete is used with a working stress superior to 60 kp/cm², unless a bracket is chosen which has a thickness inferior to 0.6 m.

3) If a short-term check dam siltation is expected or the check dam is to be designed to withstand debris-flow stress, the simple L design with upstream foundation
projection is always the most favourable as to upsetting stability.

4) The computation made for a debris-flow with a pressure 7 times greater than the hydrostatic thrust, where no check dam siltation is assumed, is only intended as an example. However, it shows that a check dam of this kind can be conveniently built also in watersheds affected by debris flow. In any case, attention should be paid not to go too far with the total height of the work, to increase friction coefficient between ground and foundation and to use concrete and steel with suitable resistance characteristics. This last condition is essential in order to avoid making structure heavier. This, in fact, already has a remarkable volume and weight because of the external force considered.

6. References
