A REVIEW OF METHODS FOR MODELLING DRYSTONE RETAINING WALLS

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Drystone walls are durable structures, due to their intrinsic ductility, permeability, and the strength of the used materials. Nevertheless, they can be subject to slow deterioration due to the weathering of the materials, application of loads for which they were not designed, impact or inappropriate repair methods. It is then necessary to assess the condition of the structure, and to repair or to replace the construction. This paper aims to bring together advances made by researchers in France and in the United Kingdom over the last two decades on drystone retaining walls. Three methods which are used to evaluate the stability of drystone retaining walls are summarized and compared: distinct element method, limit equilibrium analysis and yield design analysis. Charts which enable the preliminary design of drystone retaining walls are presented, which derive from the understanding obtained from these models and confirmed by full-scale testing.

Keywords: drystone retaining wall, distinct element, limit equilibrium, yield design, homogenization, design chart

1. INTRODUCTION

Drystone walls are used around the world as castle walls, field boundaries and retaining walls for roads and terracing. They are an essential part of many monuments which have been designated as World Heritage Sites, such as Great Zimbabwe National Monument in Zimbabwe (1986) or the Historic Monuments of Ancient Nara in Japan (1998). Within France and the United Kingdom, a substantial proportion of retaining walls on roads are constructed of drystone: about one-sixth of road gravity retaining walls in France, Odent 2000, and about half of highway retaining walls in Great Britain, O'Reilly *et al.* 1999. These structures have a long life. The drystone walls at Great Zimbabwe Monument were constructed in the 11th century, and most road dry-stone retaining walls in France and the UK date from the 19th and the early 20th centuries. However, the constituent materials are subject to weathering, especially to frost damage in colder climates, and so do deteriorate over long periods. Poor repairs, especially pointing and grouting, can accelerate the deterioration by holding water within the structure, leading to accelerated weathering or even catastrophic collapse if significant pore pressures build up. Imposed loadings can be much higher than in the past due to increasing axle loads of modern vehicles, and even impact damage (Gupta *et al.* 1982). Repair methods or even full reconstruction may be needed. In the case of reconstruction, engineers need a design method which allows economical construction by avoiding over-conservative design assumptions. Such methods need to be based upon a proper understanding of drystone retaining wall behaviour, which in some respects can differ from the behaviour of conventional mortared masonry or mass concrete gravity retaining structures. These methods can allow efficient use of materials and resources, to produce structures which are both sensitive to the local environment, and sustainable.

Substantial studies of drystone wall construction and performance have been carried out over the last 25 years, and these studies are ongoing, with the aim of guiding the maintenance, repair and new construction of these structures. Three main methods have been used for modelling drystone retaining walls:

- Limit equilibrium method (LEM)
- Yield design method (YDM)
- Distinct element method (DEM)

Each method has advantages as well as disadvantages. These methods will be summarized, and their use described. Design charts will also be presented.

2. DISTINCT ELEMENT METHOD

This numerical method was developed to analyse the deformation of jointed rock in rock mechanics (Cundall, 1971). In the field of drystone construction, it was first used by Walker and Dickens (1995) to simulate the behaviour of the free-standing and retaining walls of Great Zimbabwe. It was also used by Harkness *et al.* (2000) and by Claxton *et al.* (2005) to model the well-known four drystone wall tests by Burgoyne (1853).

The principle of the distinct element method is to set up and to solve the equations of motion for the elements. According to the distinct element method, the system to be analyzed consists of discrete elements, which may be rigid or deformable; deformable elements are discretized into triangular sub-elements. The discrete elements touch at contact points which may change, and transmit forces between them at these points. If the forces acting on an element are not in equilibrium a displacement will result, and if moments are not in equilibrium a rotation will result. The analysis proceeds in a series of time steps, chosen to be small enough so that in a single step an element can interact only with its immediate neighbours. For each time step, a force-displacement law is used to determine the contact forces, whilst applying Newton's second law using the out-of-equilibrium forces and moments defines the instantaneous acceleration of the element, which is integrated to give the velocity. Once the relative velocity between the contact points is known, the relative displacements must be calculated and new contact forces deduced. This cycle is repeated until movements cease, when equilibrium is achieved. At this stage, the structure being analysed may or may not be standing: it is possible for the elements to be rearranged completely. If the structure is standing still, then the equilibrium deformations and stresses at the contact points may be obtained. If not, then the movement of the individual elements during the analysis illustrates the failure mechanism. The question then naturally arises as to when the time stepping process begins. Commonly the analysis traces a construction process, with layers of material being added, and the analysis run through to equilibrium. However, how the Walker *et al.* (2007) reported a faster approach of defining the full geometry from the outset, then progressively applying gravity to the model. This approach was feasible for back-analysing structures which were known to have stood, and would work for a structure which can reach a stable equilibrium.

Figure 1 shows a distinct element model established by Walker *et al.* (2007) using UDEC (Universal Distinct Element Code) for wall D in the series of tests carried out by Burgoyne (1853). In this example, the stone, the soil and the bedrock were all considered as elastic/Mohr-Coulomb plastic materials. The cross sections of the wall, the backfill and the rock foundation were divided into meshes of discrete elements. The backfill soil was treated as deformable whilst wall blocks were defined as rigid in order to reduce the running time. Meshes used by Harkness *et al.* (2000) were about 2 times denser, and all elements were defined as deformable; it took them 7 days to run an analysis using an RS6000 workstation. Claxton *et al.* (2005) spent only 60-80 minutes to carry out a calculation using a Pentium II. Whilst the more powerful computer contributed to the substantial time saving, increasing the greater number of elements considerably increases, the time required for the calculation. The value of the greater precision of the result is very questionable, given the uncertainties in the input data.

The distinct element method is useful for exploring aspects of drystone behaviour, and for investigating the sensitivity to variations of the input parameters and the geometry. However, it is not useable for routine design because of its complexity. As with Finite Element Analysis, it is easy to produce a result with very impressive graphics, but it is very easy for that result to be wrong, and depth of understanding of the problem and of the analytical method is needed to ensure that results are good. It is also very time-consuming for routine work. Besides the material properties such as the unit weight of stone, the internal friction angle of the wall, the distinct element method also asks for the knowledge of joint stiffness (normal stiffness and shear stiffness), which is not simple at all to determine. In case there is no data on stiffness parameters (as in the tests by Burgoyne 1853), researches on stiffness properties for rock interfaces may be consulted. There remains inevitable uncertainty, and as the results obtained may be quite sensitive to this parameter (Walker *et al.* (2007)), the method cannot be very reliable as a predictive tool, even though it is useful as an investigative tool. Harkness *et al.* (2000) and Claxton *et al.* (2005) compared the results of drystone wall behaviour using the distinct element method and limit equilibrium and found out that the results given by the two methods could be in agreement with the experimentation data partly given by Burgoyne. That is, even though the geotechnical parameters were not given by Burgoyne, the differences in the reported behaviour between the four test walls in relation to their geometries could be predicted using distinct element method and using limit equilibrium assessments.

3. LIMIT EQUILIBRIUM ANALYSIS

The limit equilibrium analysis has been used for a long time in the design of gravity retaining walls. It is based on the equivalence between stabilizing and unstabilizing actions applying to the considered wall. But it should be noted that in the usual case of an unreinforced concrete retaining wall for example, only an external stability is considered. That is, the wall acts as a single monolithic body, and the failure surface is defined as the contact surface between the foundation and the structure. In contrast, in case of drystone retaining walls there is also the possibility of internal instability with the failure surface passing through the wall; this was for example referred to by Harkness *et al.* (2000) while comparing the results of analysis using the distinct elements method and the limit equilibrium. But Harkness was not the first to address the problem. More than a century ago, Constable (1874) introduced the same idea, carrying out reduced scale experiments using blocks of pine as bricks and oats as backfill. These experiments showed that scaled walls did not overturn in their entirety, but the failure surface made an angle of 45° with the base. The consequences of this observation will be developed in the description of the limit equilibrium model.

The use of limit equilibrium analysis for drystone retaining walls was studied by Villemus *et al.* (2007) and then Mundell *et al.* (2009). Villemus *et al.* (2007) developed calculations considering that the wall was monolithic, whereas Mundell *et al.* (2009) presented a computer program which treated the wall as a series of stacked layers to enable investigation of the position of the line of thrust.

3.1. Monolithic wall analysis

Figure 2 represents the calculation model used by Villemus *et al.* (2007) for the internal stability of drystone retaining walls. Two modes of failure were considered, sliding and overturning. In both cases, the structure is separated into two monoliths by a plane an angle ψ to the horizontal. The value of ψ is determined by consideration of the physical characteristics of the structure being analysed. For failure by sliding, the recommended values are 0 if the wall is built from cut stone and 0.2 radian (12^o) for rough stone. The authors did not give a specific value for ψ for overturning, but suggested that it should lie between 0 and 0.2 radian. The limit will in fact be given by the slenderness of block, the implication being that stones below this line will not be lifted up as the wall above it overturns. The lower the blocks in relation to their width, the lower will this angle be. Whilst the lower monolith does not contribute to the overturning resistance, neither will the pressure on the back of it contribute to the overturning force, and it is not immediately obvious which of these factors should be critical. The yield design analysis described below considers the possible values for ψ explicitly, but for the purpose of comparison an angle of 1 vertical to two horizontal (i.e.30^o) will be taken to be representative for the limit equilibrium calculations. The same considerations apply to both methods.

The analysis requires a value for the friction angle at the wall-backfill interface (δ), which was not considered by Villemus *et al.* in their analysis of hydraulically loaded experimental structures, which could not apply any friction on the internal face of the wall. However, in the general case of earth backfill, based on the work of Colas *et al.* (2008), δ may be set equal to φ_s - the backfill friction angle. This value was also used by Mundell *et al.* (2009), giving results which corresponded very well with full-scale tests pursued to overturning failure.

3.1.1. Wall stability against sliding

The wall is considered to be stable against sliding if the safety factor of wall stability against sliding is not less than 1. For $\psi = 0$:

$$F_{sli} = \frac{V \ \tan\varphi}{H} > 1 \tag{1}$$

in which:

φ represents the friction angle of the blocks and is determined by the shearbox tests.

V and H are respectively the vertical and horizontal component of the resultant of external forces applied to the failure part OIJO'.

Solving the inequality (1) will give us B^{sli} – the minimal base thickness required to assure the wall's internal stability against sliding.

For $\psi > 0$, V and H are replaced by the forces acting normal to and along the failure plane.

3.1.2. Wall stability against overturning

The wall stability against overturning is assured if the resisting moment (M_r) is greater than or at least equal to overturning moment (M_{ov}). In the other words, the overturning safety factor:

$$F_{ov} = \frac{M_r}{M_{ov}} > 1 \tag{2}$$

By equilibrating M_r and M_{ov}, we can find out B^{ov} – the minimal base thickness required to assure stability against overturning.

In the end, we have the ultimate base thickness required defined as:

$$B^{ult} = \max\{B^{sli}, B^{ov}\} \tag{3}$$

3.2. Multi-blocks wall analysis

Mundell *et al.* (2009) developed a program using the Delphi development environment to analyze the stability of walls. This program was created to investigate the position of the “line of thrust” within the wall (Cooper, 1986), and how it changes in response to changes in a wall’s geometry and loading. A wall is stable provided the line of thrust remains within the width of the wall; this is equivalent to the analysis used in arch bridges (Heyman, 1966, 1988). The aim of the program was to give some insights into wall behaviour very quickly, by comparison with the time-consuming complexity of distinct element modelling as described above. The wall is considered to be composed of a series of stacked blocks with horizontal upper and lower surfaces. Each block extends from the front to the back of the wall and represents a complete course of stone within the real structure. It is identified by the co-ordinates of its 4 vertices (figure 3), from which its area and centroid are determined. The geometry and position of these blocks thus determines the overall geometry of the wall. The program allows this geometry to be altered by a mouse-click on the cross-section shown on the computer monitor, or by entering new values into the table of data. The new positions of the resultant forces at each block interface are shown virtually instantaneously, together with the line of thrust. Provided that the line of thrust lies within the structure, then overturning will not occur.

The applied loads to each block are composed of:

- Block weight (W): Block weight is calculated by multiplying the area and the unit weight of the material. It takes the centroid of the block as its point of application.
- Backfill pressure (*P*): It is represented by a force acting at an angle of δ with the normal of the internal face, placed at a height determined by the difference in pressure between the top and bottom of the block.
- Surface loading (q): The influence of a load applied to a limited area of the ground behind the top of the wall is determined by assuming that the load spreads out over an area which increases with depth by a ratio of (1 horizontal : 2 vertical) in all directions, but limited by the position of the back of the wall. The surface load application is only taken into consideration when this load spread touches the wall.
- Load transmitted from the block above (zero for the topmost block).

The calculation begins from the block at the top and continues to the lowest one. To evaluate the wall stability, the program checks three possible failure modes: overturning, sliding and block rotation. The wall is no longer stable when the sliding or overturning forces overpass the resisting ones.

Although based on the same theory of limit equilibrium, Mundell's approach differs from that of Villemus, where it is considering different mechanisms. The reason comes from their different aims: Villemus wanted to build a model to design new drystone retaining walls whilst Mundell aimed at assessing the stability of existing walls.

- Villemus assessed the stability of the part of the wall above a single failure line while Mundell checked the stability at the level of each course.
- The failure line used by Villemus's case was in fact a zigzag line passing through different layers of stones, while Mundell's was a straight horizontal line which separates two courses.
- Besides the two familiar failure modes of sliding and overturning, Mundell's program also considered the rotation of an individual block at the front of the wall. If the resultant force comes close to the face, the horizontal thrust as well as vertical load can rest on a single stone. If this stone is high compared with how far it extends back into the wall, then it can rotate forwards, precipitating failure of the rest of the structure. This is a simple matter to check – the use of limit equilibrium analysis requires the engineer to consider which failure mechanisms might occur, but it requires knowledge of the geometry of the blocks of stone used in the construction.
- As noted above, whilst Villemus had no need to consider the influence of the value of δ (soil/stone interface angle), Mundell like Colas (see below) proposed to take $\delta = \varphi_s$ (internal friction angle of the soil), on the basis of the rough face presented by the drystone construction to the backfill.
- Mundell took into account for the first time a surcharge load. Though a strip load would be strictly compatible with the two dimensional analysis, the implemented approach was easily extended to a square or rectangular applied load, to give equivalent values which could be used. However, if the loading was localised, the wall experiencing this loading would be restrained by adjacent

unloaded sections of wall. This restraint is actually dependent upon the quality of the construction, a fact confirmed by the full-scale tests carried out at Bath.

4. YIELD DESIGN ANALYSIS

In a general case, the yield design is used to determine the ultimate load which a structure can sustain knowing the geometry of the structure, the applied loads and the resistance capacity of the material. Two approaches can be used: an interior (static) approach which is based on statically admissible stress fields and gives the lower bound of the solution domain; or an exterior (kinematic) approach which is based on kinematically admissible virtual velocity fields and gives the upper bound of the solution domain. Colas *et al.* (2008, 2010a) chose kinematic approach in combination with the homogenization theory developed for periodic masonry (de Buhan and de Felice 1997) to model drystone retaining walls.

Firstly, the wall was approximated as built from regular cut stone blocks so that it could be considered as periodic. It was then homogenized using the theory of homogenization for periodic masonry, which took the problem from microscale to macroscale, and the strength domain of the homogenized drystone wall could be traced out.

Secondly, the yield design was applied to calculate the possible ultimate backfill height. The geometry of the wall was defined in advance by the wall height h (m), the base thickness B (m) and the batter of the internal wall face f_1 (%). The applied loads consisted of the density of the wall γ (kN/m^3) and the density of the backfill soil γ_s (kN/m^3). To define the resistance capacity of the material, the frictional Mohr-Coulomb criterion was used for both stone and soil as well as at the contact surface between the wall and the backfill. Noting that the back of the wall is not smooth but quite rough, the friction angle at the wall-backfill interface δ was set equal to the backfill friction angle

φ_s. Knowing all necessary parameters, the possible ultimate backfill height was calculated considering the inequation between the work of external actions (W^e) and the maximum resisting work (W^{rm}):

$$W^e \leq W^{mr} \tag{4}$$

Two different modes of failure were considered: translation of the wall and of the soil; and wall rotation and soil shearing. The smaller result of the two cases was taken as the final result. These failure modes were verified by 2D scale-down tests using Schneebeli rods to simulate backfill soil in two dimensions (Colas *et al.* 2010a).

It should be noted that Colas *et al.* (2010a) considered a backfill height different from the height of the wall and evaluated the wall stability in relation to the soil height that the wall could support. However, in a design problem, the backfill will generally extend to the top of the wall, and the base width is the unknown parameter which needs to be determined.

$$B = B(\underbrace{h, f_1}_{\text{geometry}}; \underbrace{\gamma, \gamma_s}_{\text{load}}; \psi, \psi_s) \tag{5}$$

Although the yield design is more complicated than the limit equilibrium, it has been considered to give better results than the approach of Villemus which depended upon a measurement of the angle ψ between the horizontal and the failure line through the wall, whereas in the yield design this angle is calculated in the homogenisation process. However, this process depends on the geometry of the construction in the same way as in the limit equilibrium approach of Villemus, because that failure plane steps up through the courses of masonry in exactly the same way as it can in the homogenization. The difference is that homogenization implicitly allows steeper angles provided that they also step through the structure in a similar way, while Villemus implicitly checked for just $\psi=0$, and for the first stepping value. A thorough limit equilibrium check would as a matter of course consider these mechanisms also, but those considered by Villemus would normally be critical, so the differences are rather academic. In all cases, a conscious decision must be made to consider steeper values of ψ, and the actual possible values depend upon the geometry of the stone used in the construction. However, the range of uncertainty introduced by this is not very significant compared with the margins of safety required in normal constructions.

5. DESIGN CHARTS

Design charts are graphs that summarize results of calculations using either the limit equilibrium analysis or yield design theory as presented above. The distinct element method is not considered here as it is not used for design. Design charts can be used to provide initial indication of the expected geometry in an initial design of drystone retaining structures. An example is found in a Design guide published in 2008 by ENTPE (Ecole Nationale des Travaux Publics de l'Etat) in cooperation with specialized masons in drystone walling. The expertise of such masons is required to ensure compliance with good practice, to produce final constructions that allow the engineering assumption of monolithic behaviour of the walls in 2D. This was verified by experiments (Villemus *et al.* 2007, Colas *et al.* 2010b, Colas et *al. 2012*, Mundell *et al.* 2009).

These charts were established using the limit equilibrium theory for the monolithic wall which was presented in section 3.1. The friction on the back of the wall was assumed as $\delta = \varphi_s$. A security factor of 1.5 which is usually used on gravity retaining walls in France was applied.

5.1. Utilisation of design charts

The guide provides 18 charts in total, corresponding to 2 kinds of stone, 3 values of backfill slope (β) and 3 different values of external batter of wall face (f_1) as follows:

- Materials : limestone, schist
- Backfill slope: 0°, 10°, 20°
- External batter: 0%, 10%, 20%

Figure 5 gives an example of chart designs that are found in the guide, showing the case of walls in schist with external batter of 10% and a backfill slope of 10°. The X-axis represents the backfill friction angle (φ_s) measured in degrees while the Y-axis represents the base thickness of the wall (B) measured in meters. Ten curves are given corresponding to ten different wall heights varying from 1.5m to 6m. Therefore, once knowing φ_s, the engineer can preset the wall height (h) and then consult the chart to find the base width of the wall. This is the minimal value to ensure that the wall is stable. For example, for a 2.5m high schist wall retaining a backfill with friction angle 30°, the minimal base width required so that the wall remains stable is 1.4m, allowing for the safety factor. The thickness at the coping could also be calculated if necessary, based on the 3 parameters: h, f_1 and B, 1.15m in this case.

It should be noted that only the walls with vertical internal faces ($f_2=0$) and the horizontal courses are considered. For other cases of β and f_1 which can't be found in the list above, there are two ways to solve them: we could either use the method of linear interpolation or simply take the closest value of the reference parameters, whilst erring on the safe side.

5.2. Graphical comparison between the results of limit equilibrium and yield design

The results of limit equilibrium analysis and yield design modelling are compared for a drystone schist wall of 2.5 m height (figure 6). A safety factor of unity is chosen in both approaches for the purpose of the comparison. The yield design method gives results close to the Limit Equilibrium method in this particular case of the failure mechanism of figure 4 and in the case of cohesionless joints. From the graph it may be seen that the two approaches are very close, with a maximum difference in base width of only 5 cm at an angle of friction of 50°. This difference is small by comparison with the required factor of safety, and indicates that either method can be used with confidence in design.

6. CONCLUSION

Studies on drystone structures are still ongoing to increase the depth of understanding, especially with respect to a wider range of construction styles. In France and in UK, where the transport infrastructures depend upon a large number of drystone retaining walls, this research has economic and environmental importance. Replacement of efficiently designed drystone structures with concrete alternatives has serious environmental and aesthetic impact, and may be significantly more expensive. Comparisons between the different methods described in this paper, and with full scale tests carried out in France and in the UK, indicate that the behaviour of drystone retaining walls is understood in detail, and can be predicted using methods suitable for routine engineering design. Design charts have been presented for a range of cases. Therefore when drystone structures need to be replaced, engineers can be confident in replacing them with newly designed drystone structures which will meet current engineering standards. These structures will be efficient and sustainable, and will sit well in their environment

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Figure 1. Distinct element model by Claxton et al. 2005.

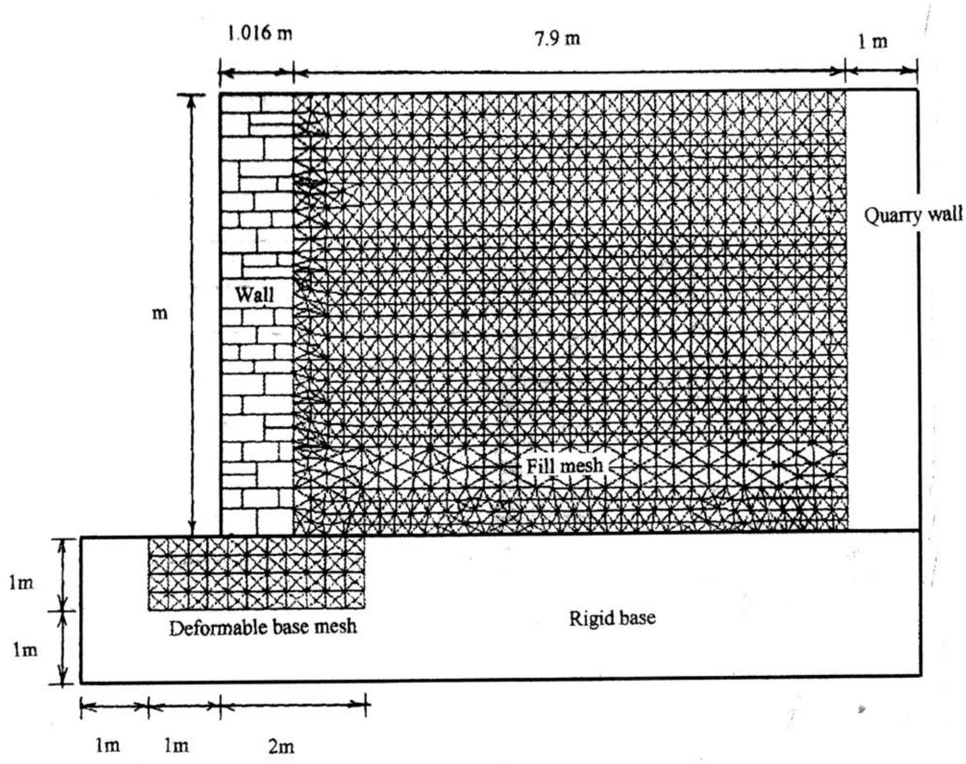
Figure 2. Model of limit equilibrium – monolithic wall.

Figure 3. Model of limit equilibrium – multi-block analysis.

Figure 4. Model of yield design.

Figure 5. Design chart for drystone retaining walls with β=10° and f_1=10% .

Figure 6. Comparison between results of yield design and limit equilibrium (h=2.5m; β=10°; f_1=10%).



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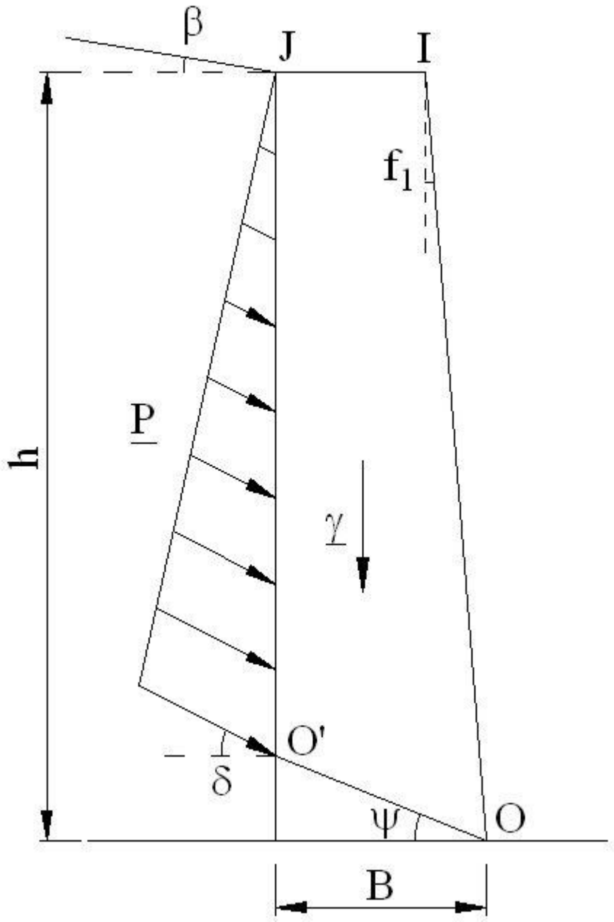

Figure 2. Model of limit equilibrium – monolithic wall

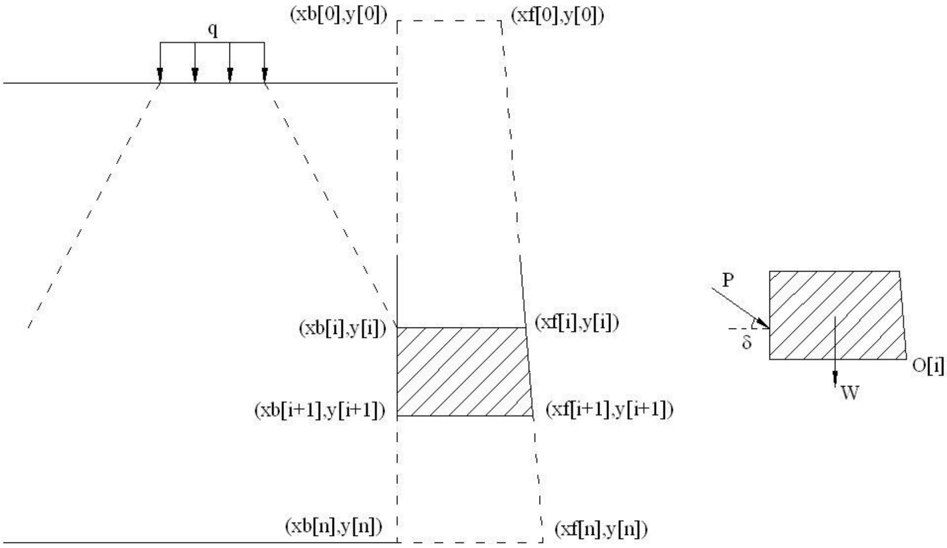

Figure 3. Model of limit equilibrium – multi-block analysis

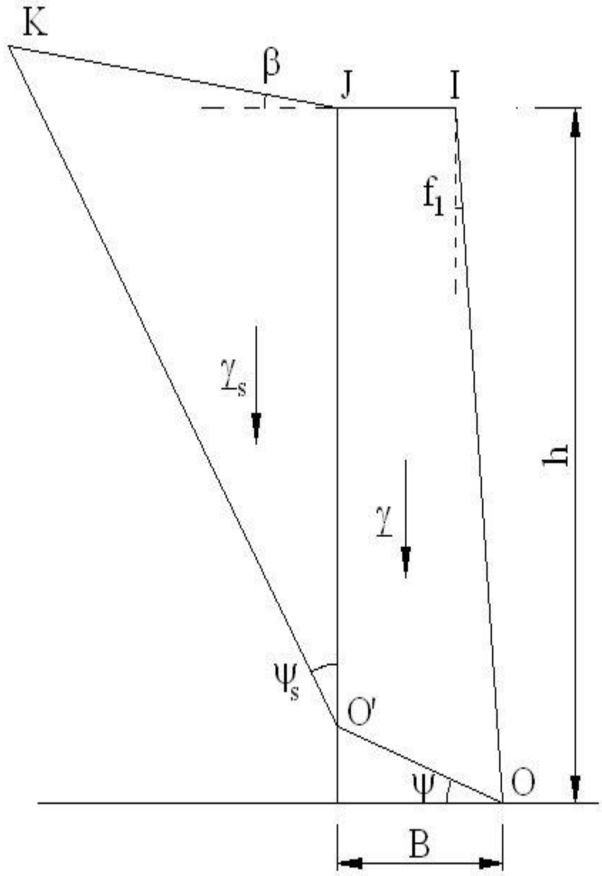

Figure 4. Model of yield design

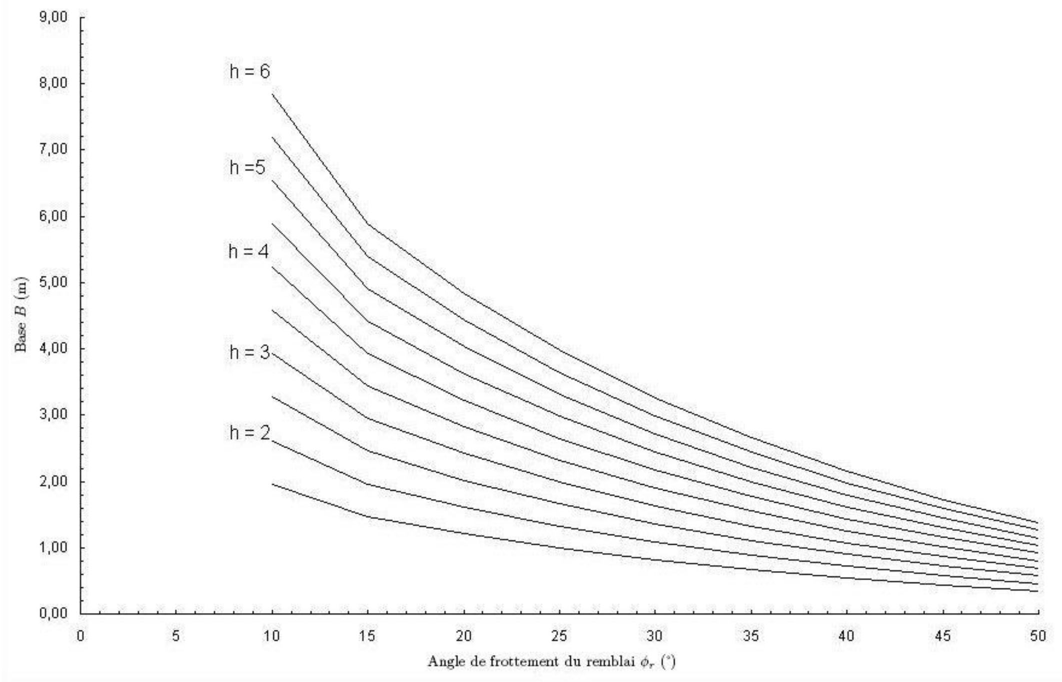

Figure 5. Design chart for drystone retaining walls with β=10° and f_1=10%

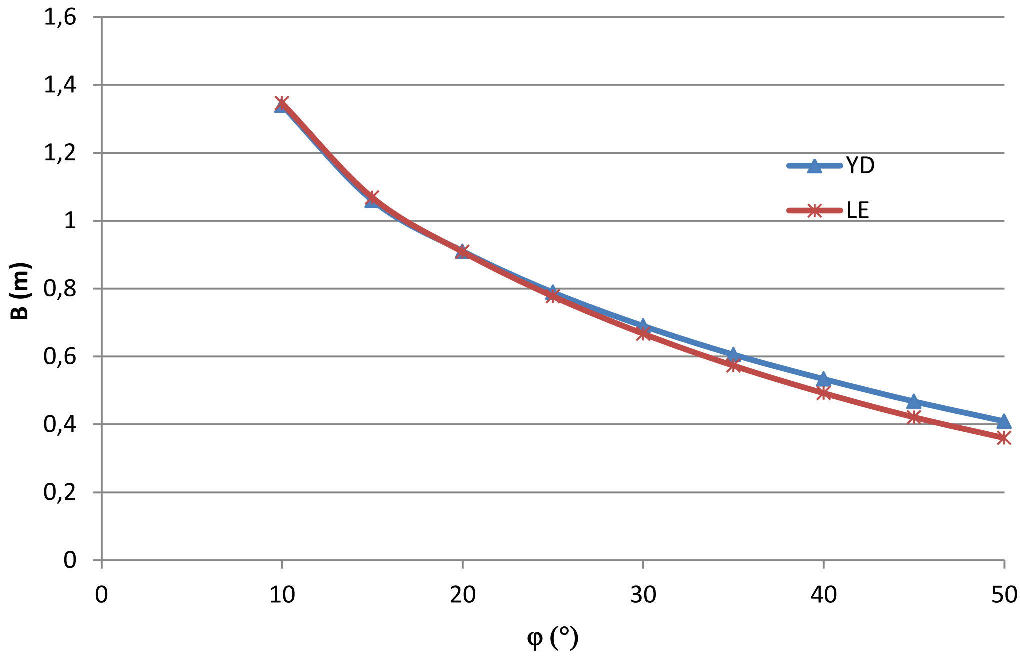

Figure 6. Comparison between results of yield design (YD) and limit equilibrium (LE) (h=2.5m; β=10°; f_1=10%)