Deflections and failure modes in dry-stone retaining walls

by M.R. COOPER*, BSc, MSc, DIC, MICE, CEng

Introduction

THE MAINTENANCE, repair and replacement of ageing dry-stone retaining walls is a growing problem for many authorities. This type of wall provides support and protection to hundreds of miles of highway, often on major trunk roads. Jones (1979) has described the construction of these walls, and an account of current engineering problems associated with them was given in *New Civil Engineer,* 18th August, 1983. The typical construction with horizontally aligned flaggy backfill behind a carefully placed facing is well illustrated by the exposed cross-section of a typical wall shown in Fig. 1. A slight variation employed for down-slope walls where a road crosses side-long ground using lateral cut and fill is to use a double facing with flaggy backfill between. In either case the wall will gain much of its stability from being constructed with a back-tilt, or batter, as shown in Fig. 2, where the batter is highlighted by the angle between the vertical lamp post and the service pipes on the face of the wall. This batter can play an important part in influencing the eventual failure mode.

In preventive maintenance an essential first step is to observe or deduce any developing failure mechanism and so to identify the cause of distress. This is particularly true for dry-stone walls which can stand large movements before collapsing. The failure modes of rigid retaining walls are well known and form the basis of standard design procedures where the walls are checked for resistance to overturning, sliding and general slope failure. However, for retaining walls which do not act as rigid bodies additional failure modes are possible. In the particular case of dry-stone walls failure is often preceded by the development of a pronounced bulge, and eventually occurs by bursting at the bulge. An understanding of the mechanism involved in this type of failure should be useful in designing an efficient repair.

*Department of Civil Engineering, The University, Southampton.

Field studies

The study of failed walls can help in identifying possible mechanisms, but the wide range of applications to which dry-stone walling was put during the Victorian construction boom can give rise to misconceptions. For instance, what is in fact a masonry revetment to a roughly trimmed face of weathered rock with only a metre or so of infill may be taken for a high dry-stone retaining wall. A number of failures of such walls, and the closely related case of walls on steep side-long rockhead at shallow depth, have led to the belief that dry-stone wall failures are typified by a much steeper failure surface than would be predicted theoretically.

Field studies can also be complicated by the number of failures caused by recent alteration of, or interference with, short lengths of wall. The affected length then collapses leaving only the adjacent intact sections available for study. These sections though are unrepresentative of the conditions which led to failure and can give a misleading impression of the cause of failure.

Perhaps the greatest difficulty encountered in attempting to draw conclusions from

Fig. 1. Typical wall construction

Fig. 2. View showing typical batter of dry-stone walls

dry-stone wall failures is caused by the uncemented nature of the walls. The post-collapse appearance of most failed dry-stone walls is of a random pile of rubble yielding little useful information.

The most useful field studies for increasing the understanding of dry-stone wall behaviour would take advantage of the pliable behaviour of the walls. Careful and regular monitoring of actively deforming walls could yield much valuable information, but this type of extended survey is likely to be beyond the range of normal maintenance budgets.

Analysis

A simplified analysis of the stresses produced in a battered gravity wall by active earth pressure and self-weight provides the basis for a qualitative discussion of common failure mechanisms.

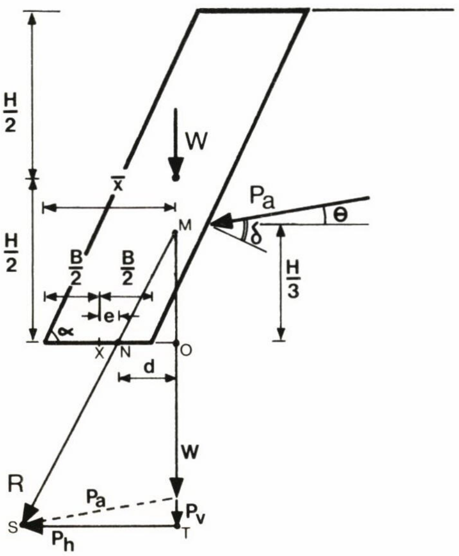


Fig. 3. Idealised section of upper H metres of wall

The dry-stone wall is represented as a parallel sided gravity retaining wall, as shown in Fig. 3. The first part of the analysis is to consider the resultant force, *R*, acting on a horizontal plane *H* metres below the top of the wall.

The weight of the wall, *W*, will act through the wall centroid, so that

$$\bar{x} - \frac{B}{2} = \frac{H}{2}\cot\alpha$$

The earth thrust, P_a, is assumed to act through the line of action of *W* at *M*, H/3 above the plane under consideration.

The resultant is then represented by *MS* in triangle *MST*, where *MT* represents $W + P_v$ and *TS* represents P_h; P_v and P_h being the vertical and horizontal components of P_a.

From similar triangles, *MNO* and *MST*;

$$\frac{d}{P_h} = \frac{H/3}{(W+P_v)}$$

giving

$$d = \frac{H}{3}\cdot\frac{P_h}{(W+P_v)}$$

The eccentricity of the resultant on the plane is then given by:

$$e = \bar{x} - \frac{B}{2} - d$$

$$= \frac{H}{2}\cdot\cot\alpha - \frac{H}{3}\frac{P_h}{(W+P_v)} \quad \ldots (1)$$

Where a positive value of *e* indicates a resultant acting to the right of X in Fig. 3.

P_h and P_v may be determined from Coulomb's analysis, expressed algebraically, giving, for $c' = 0$,

$$P_h = \tfrac{1}{2}K_A\gamma H^2\cos\Theta$$

and

$$P_v = \tfrac{1}{2}K_A\gamma H^2\sin\Theta$$

where

$$K_A = \left\{\frac{\sin(\alpha-\phi')/\sin\alpha}{\sqrt{\sin(\alpha+\delta')}+\sqrt{\dfrac{\sin(\phi'+\delta')\cdot\sin\phi'}{\sin\alpha}}}\right\}^2$$

for level ground

and

$$\Theta = (\alpha + \delta' - 90)^\circ$$

If, as in this sample analysis, the contact plane is perfectly flat and the materials of the wall are totally rigid, then relative rotation between blocks on either side of the contact can only occur if $e < -B/2$ or $> +B/2$. Furthermore, back rotation ($e > +B/2$) can only occur in conjunction with a forward displacement of the rotating blocks, since otherwise the back rotation would be prevented by an increase in earth pressure above the active value.

In practice the contacts between individual blocks in a dry-stone wall are not perfect or plane, and any horizontal plane will not be rigid since it may involve several uncemented blocks which are able to move slightly relative one to another. As weathering takes place the surface of the blocks will become compressible, and the effective width, *B′*, of the blocks will reduce as shown in Fig. 4. Thus the potential for relative rotations is much greater in a real wall and increases with the age of the wall.

Failure mechanisms

By considering the variation of eccentricity of the resultant over the full height of the wall it is possible to explain qualitatively some of the more common failure mechanisms, at least those involving only relative rotations within the wall. The line of thrust which would be produced by equation *(1)* for a typical battered wall is shown for two different degrees of batter as part of Fig. 5.

The positive eccentricity reaches a maximum in the upper part of the wall and then decreases towards the base, with the slope of the line of thrust becoming shallower. The eccentricity is therefore positive over most of the wall height, only a small part of a stable battered wall being subjected to forwards toppling moment.

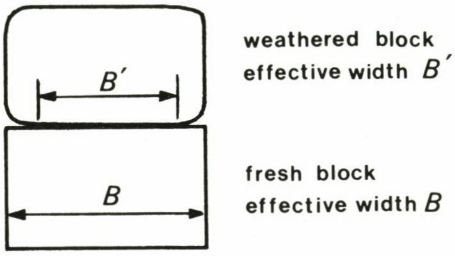


Fig. 4. Effect of weathering on block width

By considering lines of thrust, as in Fig. 5, we can associate various observed failure modes with the wall configurations that might give rise to them.

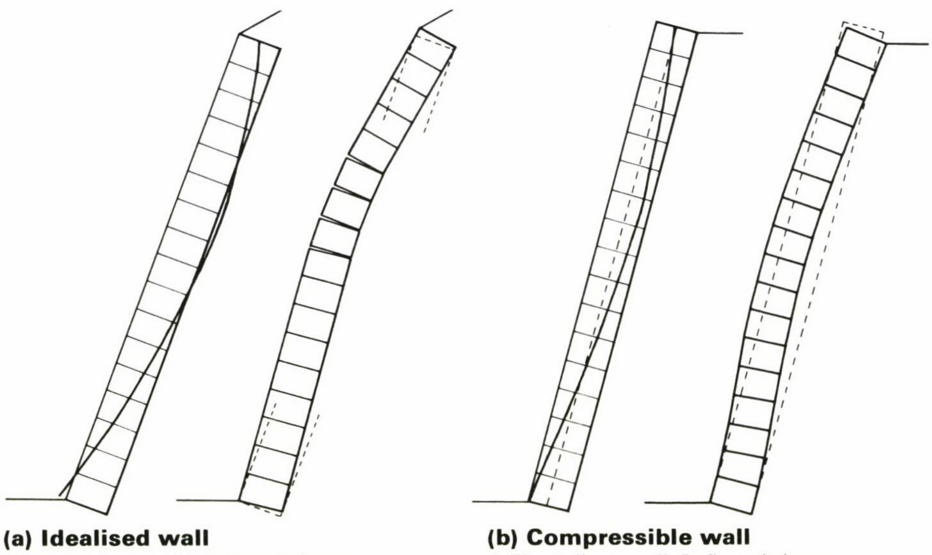
Toppling failure

The simplest failure mechanism is for a wall in good condition on a rigid base, and of a height such that the eccentricity of the base is negative and outside the base width. This wall will simply rotate about its toe. If the resultant is within the effective width in the rest of the wall no other deformation will occur and a straightforward toppling failure, as a rigid wall, will result. A toppling wall rapidly becomes less stable as rotation takes place, only the restraint of adjacent sections keeping it in place. It is therefore unusual to find walls with large pre-failure deformations due to toppling.

Bulging failure

The term bulging failure covers a group of closely related failure mechanisms in which part of the wall moves out to produce a convex vertical profile in a previously plane front face. For bulging failure to be caused by relative rotations of the wall elements, either the eccentricity in the upper part of the wall must be sufficient to give a tendency to back tilting under active earth pressures, or alternatively, the wall fabric must be sufficiently weathered to allow vertical compressive strains in the back of the wall, again under active earth pressures.

For a stable wall neither of these two effects will be evident as active earth pressure conditions will not apply. The tendency in each case for the wall to move against the backfill will give rise to higher earth pressures, increasing P_h and decreasing the eccentricity to produce



(a) Idealised wall
(i) Thrust line (ii) Deflected shape

(b) Compressible wall
(i) Thrust line (ii) Deflected shape

Fig. 5. Bulging failure modes on a rigid base

Fig. 6. Example of bulging on a rigid base — note decay of wall at base

equilibrium. However, for sufficiently high walls the eccentricity at the base will be negative, and may be large enough to cause forward rotation even on a rigid base.

The effect is shown in Fig. 5, where both perfect and weathered walls show a forward rotation at or near their bases, giving an overall forward movement, with back rotation of the upper parts of the wall producing a bulge. For the perfect wall the elements group into rigid masses within which the eccentricity is insufficient to produce relative movements. For the real

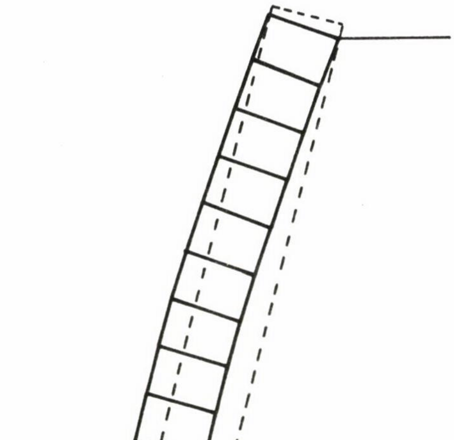


Fig. 7. Bulging failure on a compressible base

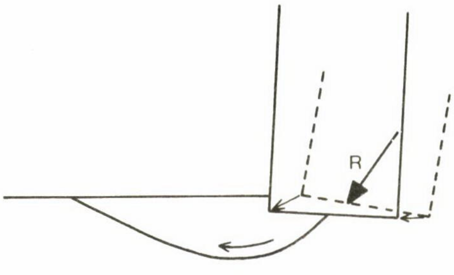


Fig. 8. Displacement of wall base due to bearing capacity failure

wall, where relative rotations are caused by compression of the weathered block boundaries, a more classic bulge is produced.

Bulging failures on truly rigid bases appear to be comparatively rare, except where accelerated weathering of the lower part of the face has increased the compressibility dramatically in the zone of negative eccentricity. Such a case is shown in Fig. 6, where the extent of the deterioration is evident in the foreground.

Bulging failure on compressible foundations

The development of bulging failure on a compressible foundation is very similar to that on a rigid base, but the nature of the foundation makes it more likely to occur. In this case it is only necessary for the eccentricity at the base to be negative for some forward rotation of the lower part of the wall to be produced by differential settlement of the foundation soils. It is possible for stable bulged profiles to exist when the differential settlement is complete, but these walls will be very prone to the effects of subsequent weathering.

Fig. 7 shows the bulged profile produced for a compressible wall on a compressible foundation from a consideration of the thrust line. This wall has the same batter as that in Fig. 5*(a)* but would fail at the lower height shown.

Bulging failure due to bearing capacity

Inadequate bearing capacity of the wall foundations is most likely to give rise to a bulged profile in the distressed wall. Ticof (1978) demonstrated that an eccentric vertical load on a footing will produce a bearing capacity failure emanating from the side favoured by the eccentricity. For a retaining wall this effect is magnified by the inclination of the resultant thrust on the base, and by the greater depth of the base with respect to the retained side. The resulting mechanism involves appreciable forward movement and rotation of the base, with a smaller downward movement concentrated at the front of the base as sketched in Fig. 8. This combination of base rotation and forward movement is typical of many retaining walls with pronounced bulges. Fig. 9 shows an example.

Shear failures

Failures involving relative sliding of the wall components appear to occur in one of two forms. In the most common case the active thrust is increasing more rapidly with depth than is the shearing resistance of the wall fabric and sliding is produced near the base. Complete sliding at the wall base is analogous to sliding of a rigid wall and differs only in post-failure appearance where the structure is usually badly disrupted. The results of a base sliding failure, instigated by slope instability below the wall are shown in Fig. 10.

Distributed relative sliding to give a concave front profile is a feature of the top sections of walls supporting sloping ground.

Fig. 9. Example of bulging due to inadequate bearing capacity

Here the softening and creeping of the near surface soils exerts an abnormally high thrust on the top parts of the wall. The result is a pushing forward of the top few elements as shown in Fig. 11.

Conclusions

The direct causes of dry-stone wall failures are not always located at the point of apparent distress in the wall. Moreover, the flexible nature of the structures and support from adjacent sections will allow large deflections to occur quite safely before collapse. A careful study of the pattern of deformation, based on concepts such as those outlined here, can give a useful insight into the real cause of distress. Remedial works may then be designed which deal specifically with the source of instability, and which are more likely to produce an effective long term repair.

As an example it can now be seen that maximum benefit will not be obtained from simply strengthening the wall or backfill in the vicinity of a bulge. If the bulge is due to decay near the base of a well founded wall then the base of the wall should be grouted; if inadequate bearing capacity is causing the bulge, buttressing could be provided or the area in front of the wall surcharged, in either case the improvement in the bearing capacity conditions will reduce the forward displacement as well as arrest the bulging.

At present there is little information on the compressibility parameters applicable to dry-stone walls and Figs. 5*(b)* and 7 have been produced using presumed angular compressibilities of 0.2°/kN.m for the boundaries between 0.6m x 0.4m wall elements. This angular compressibility is undoubtedly excessive for perfectly plane faces of fresh stone but is thought to represent fairly well the conditions in real walls of the form shown in Fig. 1.

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Fig. 10. Example of sliding on base, due to instability below wall

Fig. 11. Example of distributed sliding in wall retaining sloping ground

As more data becomes available the simple analysis presented here can be extended, not only to provide a more rational basis for dry-stone wall maintenance, but also to include consideration of the modern successors of dry-stone walling, such as crib walls and high gabion structures. These modern flexible walls with little or no tensile strength gain their stability in a manner very similar to dry-stone walls and it seems probable that their long-term behaviour will also prove to be similar, differing only in terms of wall compressibilities and construction geometries.

The aim of this initial review of dry-stone wall deformations however is to provide some insight into how a full and careful consideration of the actual causes and mechanisms involved can promote more efficient maintenance of deforming flexible walls.

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head beam (Fig. 4*b*). Both characteristic velocity-time histories are shown in Fig. 4.

The pile length was calculated assuming an average wave velocity of 4 200m/s. All the piles were cored. Fig. 5 gives a correlation plot between the length measured by the low strain testing method and by coring. Fairly accurate results for the determination of the pile length could be expected within a range of $\pm 10\%$. The concrete compressive strength was between $48 N/mm^2$ and $64 N/mm^2$, and mass density varied between 2.33 and $2.39 t/m^3$.

The attenuation of the wave as it propagates down a shaft is influenced by the construction method. It is believed that the slurry-cast pile had a very rough surface compared to the large diameter bored piles, for which casing was used. The length/diameter ratio was 7:1-10:1.

Conclusions

The stress wave propagation method is a simple, rapid, inexpensive and reliable method that provides for initial qualitative information about a pile. The basics for the application is the knowledge of the wave propagation behaviour in the shaft, which is influenced by several factors including concrete quality, length/diameter ratio, surrounding soil conditions, method of concrete placement and conditions and installation method of the piles.

This Paper has attempted to show a number of different velocity-time histories and their characteristics. It is, however, recommended that the method is only used in connection with additional investigations of pile concrete qualities and soil properties.

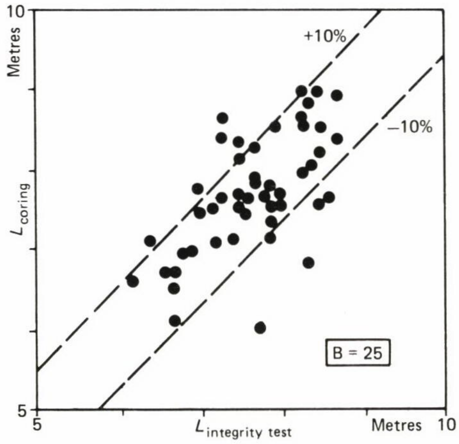

Fig. 5. Comparison of pile length

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