$$\sigma_i^2 = \frac{2EW\delta l_i}{Al}$$

$$= \frac{2W}{A} \cdot \frac{E\delta l_i}{l}$$

$$= \frac{2W}{A} \cdot \sigma_i \qquad \ldots.(A.2)$$

$$\text{i.e. } \sigma_i = \frac{2W}{A}$$

Note stress in eqn. (A.2) is σ_i, not σ_s

Numerical results

static stress = 103·9 N/mm²
static displacement = 5·197 mm

(1) suddenly applied load — — $h = 0$ using eqn. (6)

$$\sigma_i = \frac{W}{A}\left(l + \left(l + \frac{2AEh}{Wl}\right)^{0\cdot 5}\right)$$

$$= \frac{1000}{9\cdot 621}\,(l + (l + 0)^{0\cdot 5})$$

i.e. σ_i = 207·8 N/mm² (twice static stress)

$$\delta_i = \frac{\delta_i l}{E} = \frac{207\cdot 8 \times 10\,000}{200 \times 1000}$$

i.e. δ_i = 10·394mm (2 × static deflection)

(2) Allow load to drop 5·197 mm

$$\sigma_i = \frac{1000}{9\cdot 621}\left(l + \left(l + \frac{2 \times 9\cdot 621 \times 200\,000 \times 5\cdot 197}{1000 \times 10\,000}\right)^{0\cdot 5}\right)$$

= 283·86 N/mm² (approx. 2·73 × static stress)

$$\delta_i = \frac{283\cdot 86 \times 10\,000}{200\,000}$$

= 14·2 mm (approx. 2·73 × static displacement)

We hope that Mr Sutton will find that this letter answers his question. We are grateful to Mr Hallett, also, for pointing out the limitations of the solution provided and reminding us of the assumptions made.

Other letters were received from Mr G. Eades of Stourbridge, West Midlands; Mr E. A. F. Robinson of Surbiton, Surrey; Mr Andrew More of Edinburgh; Mr D. Turner of Royston, Hertfordshire; Mr Peter Robins of the University of Technology, Loughborough; Mr P. J. Dixon of Manchester; Mr M. Young of Sheffield; Mr Edward Tufton of London; Mr E. W. Bentley of Ilkley, West Yorkshire; Mr H. W. Baker of Glasgow; Mr J. H. Sakula of Lusaka, Zambia; Dr G. R. Sharp of Colchester, Essex; Mr P. Chan of Hong Kong; Mr Eric Kan Shiu Kay, also of Hong Kong; Mr Sunil Sharma of Purdue University, Indiana, USA; Dr Ravi Jategaonkar of Halifax, Canada; Mr Ian Hunt of Selangor, Malaysia; Mr P. J. Bulman of Preston, Lancashire; Mr S. S. W. Wong of Ilford, Essex; Mr R. Catto of Ontario, Canada; Mr Sean Walsh of Bath, Avon; Mr S. S. Kulkarni and Mr C. L. Johnson of Birmingham, whose letter we refer to later. We are very grateful to all those who wrote providing the advice sought by Mr Sutton. Some of the points raised in these letters are now referred to.

One of our correspondents questioned the authenticity of the original inquiry by asking whether it was a 'set up', presumably to encourage some response from readers. While we receive many fewer letters from readers than we would wish, we did not resort to this device. If we have sometimes had doubts whether letters are read, these are now resolved by our readers' speedy and generous response.

Several letters questioned the wisdom of publishing a letter which was clearly based on a false premise, since the solution appears in many of the standard textbooks on strength of materials and applied mechanics. A few further suggested that it should not have appeared in The Structural Engineer. *Our own experience shows, however, that, if one reader has a query, there are others who are similarly puzzled and may therefore find the solution of value. Letters for publication are not subjected to rigorous appraisal by assessors and reporters as are papers. If they were, a number would be rejected and the intentional informality of the column would be lost. The* Verulam *column was introduced some years ago at the request of members to provide a means of communication on matters of general interest to structural engineers which were not apppropriate for presentation in papers or Institution notices. We are always pleased to hear from readers on what they want to see included in the column and, for that matter, what they would like to see left out.*

The masonry code—CP 111 and BS 5628

Mr A. N. Beal, writing from Leeds, draws our attention to differences between BS 5628 and CP 111 and asks why what was previously satisfactory is not now acceptable:

Amid all the correspondence on simplification of CP 110, I'm surprised that no one has suggested the biggest simplification of all which can be made, reducing time taken and also errors: if all the tabulated stresses are reduced by 1/1·5, all the load factoring and its attendant complications can be avoided and design based working loads — making things much easier.

However, my main point concerns not concrete but masonry subjected to lateral loads, comparing CP 111 with BS 5628. While there is relatively little difference between them in results where the brickwork has little vertical preload, the two Codes differ widely on the required safety factor where there is a substantial vertical load.

As an example, I calculated the safety factor against failure provided by various methods adopted under CP 111 and BS 5628 for a wall of 35 N/mm² bricks, laid in 1:1:6 mortar, carrying a dead load vertical stress of 0·5 N/mm² and subjected to lateral wind load. Assuming BS 5628's quoted 'characteristic' compression and flexural strengths are correct, the maximum load eccentricity the wall could sustain before failure is 0·47 × its thickness (based on overturning) or 0·30 × its thickness (based on flexural tension). Clearly, the first governs from a safety point of view.

Under CP 111, design was based on either (a) a limiting tensile stress of 0·07 N/mm² at working loads or (b) a check on overturning stability, with a safety factor of 1·4 and a load eccentricity of 0·45 t being commonly adopted. For the wall in question, the first of these gives a safety factor of 2·48, and the second 1·47. Both of these seem ample.

Under BS 5628, four different design criteria are possible: (c) a limiting flexural tensile stress (cl. 36.4.3); (d) an overturning stability calculation for a free-standing wall (cl. 36.5.3); (e) a vertical arching calculation, based again on overturning stability (cl. 36.8); and (f) as an alternative, cl. 36.8 allows the overturning stability to be checked in accordance with a different formula given in Appendix B, if preferred. All of these are based on factored loads. If γ_f is taken as 0·9 on dead load and 1·4 on wind load and γ_m is taken as 'normal' 3·5, the overall safety factor provided against lateral load can be calculated for comparison with CP 111. Calculated safety factors are as follows—(c) 2·86; (d) 1·80; (e) 5·12; (f) 1·76.

Several questions come to mind, bearing in mind that, in all cases, stability is provided mainly by the dead load and the section geometry, both of which can be regarded as pretty reliable.

(i) Why has the safety factor for design based on flexural strength been increased, when the safety factor of 2·48 provided by CP 111 seems generous by most standards?

(ii) Why have three different formulas been given in BS 5628 for essentially the same overturning calculation?

(iii) The safety factor of 1·4 or so adopted in the past for overturning stability in masonry subjected to wind loads is pretty well in line with practice in other materials. Why was it felt necessary to increase it to 1·76 or more?

(iv) Which of the two safety factors provided by the two alternative methods in cl. 36.8 is correct and, if it is the larger, why is a much lower safety factor permitted for a free-standing wall?

Apart from some wide variations in safety factor between clauses, BS 5628 appears to require substantially higher safety factors than designs that were previously considered to be satisfactory and that have given no trouble in service. Can someone explain why?

We too would like to know, as we are sure our readers would also. We are grateful to Mr Beal for his letter.

Micros and wider horizons

We were provided by Mr John Perks in September with information on procedures and costs for wider communication between engineers using their computers and in particular with some details of Micronet 800. We have now heard from Mr Peter Probert, Publicity Manager for Micronet 800, correcting some of the prices given for his services. Readers may be interested to have the further details now provided by Mr Probert:

The membership charges for Micronet 800 are £10/quarter and £6·50/quarter for Prestel; thus the combined cost for a domestic user is £16.50/quarter. Prestel's charge for business users goes up to £18.00 and consequently the total charge/quarter is £28.00 (nothing like the £44.00 quoted by Mr Perks).

While writing, I would also like to tell you a little bit more about Micronet 800.

Micronet 800 is a viewdata system, available through your micro, that provides a series of services unlike those of any other similar system. They fall into four main categories: communication, information, telesoftware, and the huge Prestel facts database. The pages are stored on six mainframe computers around Britain and anyone with a micro, modem, and telephone, can travel through Micronet's 40000-page database at his leisure. Micronet is unique among networks and bulletin boards as phonecalls are at special low-cost local rates whenever you go into the system. That's only around 40p for a whole hour. Connect time is free except during peak hours.

The network has access to around 60 000 users across the nation and with the free national electronic mail system, you can send and receive hundreds of personal messages and memos with this instant service. Your private mail awaits you whenever you 'log on'. Micronet's 'chatline' service also enables you to hold live public conversations and meetings with other members. The 'gallery' service allows members to produce and publish their own pages on Micronet. The cheapest 'home' magazine or news-sheet would cost a great deal more than the 25p/page fee on 'gallery', to produce, publish, and distribute. There is a lot more available but space prevents me from listing all of the 'communication' services.

The information services available on Micronet 800 fall into two categories: news, which is available 24 hours a day and updated daily, and education. The 'agony aunt' area was set up to give equipment and technical advice and is considered to be an essential service by all micro users, however capable they are, and Micronet also runs programing and language courses helping users to make the most of their micros.

And, of course, there is the telesoftware aspect of Micronet. Telesoftware is the term for software downloaded via the telephone line direct to your micro. Apart from the one hundred or so free software programs available every year (home accounting, games, etc.) members also have access to some exciting multiuser games. Rather than just downloading programs into your micro, users can enjoy strategic interaction with hundreds of other users, pitting their wits against each other in on-screen games controlled by powerful master computers.

As well as these services, members have access to the Prestel network which displays all manner of information and even enables users to do their shopping via their micro. All this for less than the cost of a daily newspaper! Micronet 800 really does bring your micro to life, and is leading the way in the revolution in home communication and interactive entertainment.

Further details are available from: Micronet 800, Durrant House, 8 Herbal Hill, London EC1R 5EJ (tel: 01-278 3143).

We are pleased to pass on this information.

Corrigenda

Mr C. L. Johnson, whose most recent letter has already been mentioned, also pointed out a number of errors in an earlier letter of his, as published in October. In this letter, he referred to a contribution from Mr M. I. Parker in May on the subject of shortcuts in the use of CP 110. In commenting on his letter, we confused his name with that of Mr Parker, and we apologise to him and to Mr Parker. We also compounded that error by printing errors in his program listing for the Casio fx-180P, which made it unworkable. The correct listing is now given:

Program listing Casio fx-180P

Reinforcement areas for beams/slabs to Cp 110

Preload

$M_v = 0{\cdot}15\ bd^2 f_{cu} \rightarrow K1$ & $bd\ f_{cu}/f_y \rightarrow K2$

Kout 2 ÷ 1 · 4 5 ÷ (Kout 1 ×
4 ÷ Ent M − 1) =

Key in applied moment (in same units as M_u). Press RUN to calculate reinforcement area in same units as $bd\ f_{cu}/f_y$.

This is an approximate value, within 2·238 % of exact value but on the 'safe' side. Resistance moment of a given area of reinforcement to CP 110 Preload

$M_u \rightarrow K1$ and $bd\ f_{cu}/f_y \rightarrow K2$, as above

Ent A_s ÷ Kout 2 = Inv Min × 5 · 8 −
6 · 3 8 × MR

InvX^2 = × Kout 1 =

Key in A_s. Press RUN to calculate moment of resistance of that area. This is an 'exact' value.

We are grateful to Mr Johnson for drawing our attention to the errors and again apologise to him. He tells us that he is preparing a new listing for BS 8110, which we hope to see in due course.

Seasonal greetings

At the end of another year, we take the opportunity of sending all our readers our best wishes for Christmas and the New Year and expressing our appreciation to all those who have written to us in the past year.

We once again venture to hope that the forthcoming holiday provides opportunities for the sort of reflection and rumination that will result in a full postbag for us in the New Year.

Continued from page 388

College questionnaire

30 replies were received—a significant number.

(a) Of those students from employment roughly 30 % were from consultancy, 35 % government and local authority, 25 % commercial, and 10 % other.

(b) Of those on courses 50 % were on HNC, 40 % on HND, and 10 % other types of course.

Employers' questionnaire

(a) The mix of mainly design to mainly detail is not too inconsistent.

(b) Of the technician engineers employed, AMIStructE consist of some 13 %. 19 % are in other bodies and 70 % are unaffiliated.

(c) There is therefore a large potential of members in that category who have no membership of professional societies or institutions. The importance of encouraging staff to qualify as AMIStructE cannot be overemphasised to employers, so that employees know that successful staff have passed an examination set by professional engineers and marked by them, which reflects a higher status employee who should deserve higher rewards and remuneration.

parallel to the bedjoints' is misleading. What is meant is the plane of failure.
$\frac{W L^2}{8}$ is of course the upper bound for both theories for vertical spanning only.
The load is multiplied by a factor γ_f in the load factor method in BS5628.
It is of interest to compare the results obtained by using the allowable tensile stress in CP111 and the CP114 moment with the BS5628 method.
The masonry having the nearest to unity μ value is $2 \cdot 8$ N/mm^2 concrete blocks in Table 3 of BS5628, using 1:1:6 mortar

$$\left(\mu = \frac{0 \cdot 25}{0 \cdot 4} = 0 \cdot 63\right)$$

$$\alpha W_k \gamma_f L^2 = \frac{f_{kx} Z}{\gamma_m}$$

γ_m = $3 \cdot 5$ for normal manufacturing and construction control
f_{kx} = $0 \cdot 25$ Table 3
γ_f = $1 \cdot 4$

$$Z = \frac{0 \cdot 008 \times 1 \cdot 4 \times 3 \cdot 5}{0 \cdot 25} L^2 W_k = 0 \cdot 16 L^2 W_k \text{ (BS5628)}$$

$$0 \cdot 0112 W_k L^2 = 0 \cdot 07 Z$$

i.e. $Z = 0 \cdot 16 W_k L^2$ (CP114 and CP111)

For lower values of μ BS5628 gives much more favourable results.

An interesting anomaly in the BS5628 approach is that, if a higher category of manufacturing control of block is taken in γ_m, there is a corresponding increase in flexural strength.

Dr. L. A. Clark of Birmingham has also written similarly to explain the basis of bending coefficients in CP114 as compared with BS5628, and the reason for the equality of horizontal and vertical moments in BS5628 arising when μ = 1.

Mr D. H. Camilleri, in Malta, provides his explanation for the inclusion in BS5628 of panels with strength ratios μ up to unity:

The reason why BS5628: *Part I* includes orthogonal ratios up to the value of 1 for the assessment of bending moments in panels of brickwork is not that such a particular type of brickwork may exist, as it certainly does not, but to cater for any vertical load present on the panel.

The effect of this vertical load would be to enhance the flexural strength in the parallel direction. This increase of strength may be catered for by modifying the value of μ by adding to the appropriate value of f_{kx} the stress due to the design vertical load multiplied by the value of γ_m.
i.e. the modified orthogonal ratio

$$= \frac{f_{kx}(\text{vert.}) + \gamma_m g d}{f_{kx}(\text{horiz.})}$$

The wall panel being subjected to both a bending moment in both directions and axial loading produces 'apparently' anomalous results with bending moments higher than $wh^2/8$, thus catering for the enhanced flexural strength.

These tables have been used to design built-up masonry rectangular walls by adopting an equivalent udl for the triangular water pressure and have been functioning satisfactorily.

Free-standing masonry walls

Following the initial query in July from Mr Tim Dishman as to where responsibility for designing masonry boundary walls should lie, and subsequent correspondence published last month, Mr R. G. Biggs, District Surveyor of Hackney, explains his authority's practice in the matter:

I thought that you would be interested to know that, in Inner London, we have dealt with boundary walls under Part IV of the London Building Acts (Amendment) Act 1939 for many years.

We do have a rule-of-thumb, which is that we do not deal with walls which are less than $1 \cdot 8$ m (6 ft) in height. This is because walls below this height represent a lesser hazard.

Part IV is also used to consent to high fences and other free-standing structures. Application to erect such structures is made to the District Surveyor in a manner similar to that required under the Building Regulations.

Permanent steel shuttering for concrete slabs

Mr J. M. Morton, in our June issue, raised queries relating to the balance of possibly higher repair costs of steel deck slabs used in composite construction following a fire, compared with lower repair costs, but higher initial costs, of traditionally reinforced slabs. Dr. R. M. Lawson of the Steel Construction Institute responds as follows:

There is now a considerable body of information on the behaviour of composite floors in fire tests. These floors consist of steel decking and concrete acting compositely, and supported on steel beams with welded shear-connectors. In principle, these tests have shown that, for the normal range of slab spans and loads, mesh reinforcement would be required in the slab for fire resistances up to 90 min. Additional reinforcement would be required for other cases, and therefore the decking would effectively be only permanent formwork, requiring no composite action.

In a fire test, there is normally good interlock between the deck and the concrete and any debonding tends to be localised. However, after the test, considerable debonding is often apparent. This is because of the irreversible extension of the deck relative to the concrete during heating which forces the deck away from the concrete on cooling. In a fire test, one is interested in demonstrating the loadcarrying capacity of the system and not its repairability. It would be unlikely that the composite slab could be retained after such an intense fire. Thin concrete slabs would be equally affected because the reinforcement would have weakened and the concrete cover would be shattered.

The question remains about 'small fires'. In a standard fire test, temperatures of over 700°C are reached within 10 min, and this could hardly be considered a small fire. In many natural fires in buildings of low fire load, or where there are other fire-protection measures, temperatures are relatively low. There would be three levels of potential fire damage to composite slabs which might be considered:
(a) there is no apparent debonding of the deck, ensuring that composite action under normal loads is maintained;
(b) there is debonding of the deck but little permanent deflection of the slab;
(c) there is gross permanent deflection of the slab.
The second category is the most interesting as regards repairability. Assuming that the deck was not just permanent formwork and therefore would not be required for in-service conditions, it would be necessary to remove the deck and replace it by additional reinforcement. This would be best achieved by locating bars between the ribs (fixed by shot-fired pins) and guniting in-place. Replacing the deck is not considered appropriate because of the lack of bond to the original concrete.

The argument over repairability is not just applied to composite floors. Lightweight and limestone aggregate concretes are less affected by fire than other aggregates, and this is used as a selling point. Whether specifiers consider any increased initial cost to be offset by ease of repair, in the unlikely event of a fire, is an interesting point. The relatively rare occurrence of serious fires in commercial buildings, where composite floors are most commonly used, and the fact that this method of construction appears to offer considerable savings in initial costs and construction time, suggests that specifiers and developers have established this balance.

Mr Morton also invited proposals for a simple site test to check the effectiveness of through-deck steel welding, advocated for attaching shear connectors to the top flanges of steel beams under steel decking. Mr N. W. Sutton has written from New Zealand offering detailed suggestions:

Stud welding has been around for years and years. In the 1960s the company I worked for used stud welding extensively to attach ¼ in. dia. stainless steel studs

Verulam

Queries, comments correspondence and curiosities

Settlement and liability

Responsibility in dealing with the settlement of terrace houses was first raised by Mr J. A. Tanner last July. Contributions to the discussion were received from Messrs E. J. Skilton, N. E. Hindley, and J. Pryke, in October, from Mr B. W. Totterdill in November, and from Mr R. C. Hairsine, writing on a number of subjects, in December. Another contribution has now been received from Mr G. Brandt, writing from Liverpool.

We have recently completed stabilisation of a pair of small Victorian terraced houses which were suffering from settlement of the party wall in particular, being founded on approximately 3 m of an esturine silt where it was diagnosed that voids were migrating upwards through the material and causing instability in a structure which had previously remained stable for approximately 100 years.

The stabilisation of the property was the subject of an insurance claim and the proposition was put to insurers that a number of solutions were available with corresponding advantages/disadvantages. The desire to avoid a hard spot was considered paramount and a 'soft solution' was favoured by use of a raft slab cast on to the existing ground with tenons projecting into perimeter brickwork.

As an addition to the raft solution, two further measures were proposed, i.e. reduction of the weight of the party wall by removal of chimney breasts to front and rear on both sides, together with the improvement of the existing ground.

Chimney breasts were duly removed and replaced by precast concrete flue blocks built against the party walls forming projections 450 mm × 100 mm and serving gas fires in the ground-floor rooms only.

Improvement of the silt was achieved by a 'vibrocompaction' method using a 'down the hole hammer', stone piles were inserted at 600 mm intervals of a nominal diameter of 150 mm. Each stone pile was driven and then redriven twice in order to compact successive stone fillings which consisted of a nominal ¾ in down stone aggregate locally available.

Removal of the disturbed surface was not feasible as formation level equated to underside of foundations. Therefore, the formation was compacted by use of a vibrating plate and a 450mm-thick blanket of stone was laid over the whole internal floor area compacted in 150 mm layers.

A 300mm-thick reinforced concrete slab was laid over the hard core and let into pockets in the perimeter walls.

To date, the properties appear to have remained stable without adverse effects on their neighbours.

The solution can only be described as empirical. However, the principle of vibrocompaction has also been adopted on a 'new built' local authority housing site 100 m away from the treated houses. The vibrocompaction process caused considerable noise and disturbance to adjacent properties; however, damage to finishes through the vibrations induced from the down-the-hole-hammer were minimal.

One house is now reoccupied, while the second is for sale at a price of £9000, approximately £5000 lower than the total cost of remedial works which included considerable internal refurbishment.

They are situated in a deprived area of Liverpool, and the method would clearly become much more economically viable in better areas of the country where property values are higher.

Mr Brandt says, in his letter, that he expects to monitor the situation and hopes to be able to report on further developments. We hope to hear later from him and meanwhile thank him for his letter. Although each individual job is relatively small, it is clear that the subject is important in the current context of housing rehabilitation and one that is exercising a number of our members. We shall be very pleased to pass on the experience of other readers.

Creep of timber

One of the other subjects mentioned by Mr Hairsine in his letter in December was creep of timber and its effect on the strength and stiffness of timber structures. He asked about the way in which the drafting committee for the timber Code had dealt with these characteristics. Mr J. G. Sunley of the Timber Research & Development Association, who is Chairman of the committee, has written in reply:

Creep in timber is a very complicated business, depending on such factors as the magnitude of the load in relation to the ultimate strength, the duration of the various components of the load, and the moisture content of the timber. Also, creep in joints cannot be deduced from that in the timber itself and varies with different kinds of fastening.

A study of design correlated with performance in service has indicated that current methods detailed in the timber Code give acceptable performing structures.

Within the Code we have both mean and minimum values of modulus of elasticity, deflection limitations, and different combinations and duration of loads, from long to very short-term. Thus in the design of domestic floors we use a mean modulus of elasticity and assume that all the load is of a permanent nature and that the deflection under this permanent load should not exceed 0·003 of the span.

In other cases, which may be used for storage or mechanical plant or equipment, the use of the minimum modulus of elasticity is recommended.

We are grateful to Mr Sunley for this explanation of the background to the timber Code.

The Code for masonry structures

Mr D. H. Camilleri has responded to the comments made by Mr A. N. Beal in December last, following correspondence initiated by Mr Beal a year earlier, which was concerned with the basis for the safety factors adopted in limit state design in the masonry Code. He writes from Malta as follows:

The problem of overturning and partial safety factors in BS5628 may be understood better by considering the stability of a free-standing masonry parapet wall subjected to wind pressure.

Three different methods of analysis are considered below and conclusions drawn from the results obtained.

The partial safety factors are 1·2 for wind and 0·9 for dead loading. BS5628 states that, if the collapse of a wall panel does not affect the overall stability of the structure,

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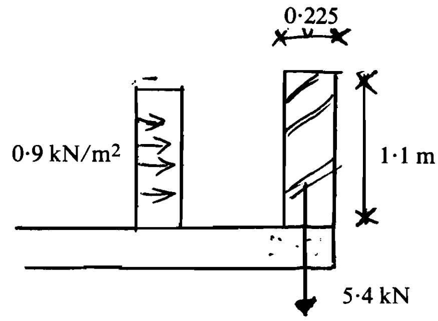


factors to be used are 0·9/1·2 instead of 0·9/1·4.

$$W = 5\cdot4 \times 0\cdot9 \qquad = 4\cdot86 \text{ kN/m}$$
$$BM = (0\cdot9 \times 1\cdot1^2/2)\,1\cdot2 = 0\cdot65 \text{ kN-m/m}$$

Method 1—middle third rule, no tension develops

$$e = \frac{BM}{W} = \frac{0\cdot65}{4\cdot86} = 0\cdot134 \text{ m}$$

$$e < \frac{d}{6} = \frac{0\cdot225}{6} = 0\cdot037 \text{ m}$$

as $e = 0\cdot134 > 0\cdot037$
the wall is far from stable under method (1)

Method 2—flexural tension allowed to develop

$$f_{kx} = 0\cdot2 \text{ N/mm}^2 \text{ (Table 3 BS5628)}$$

$$Z = \frac{bd^2}{6} = \frac{1\cdot0\cdot225^2}{6} = 0\cdot0084 \text{ m}^3\text{/m}$$

$$f = \frac{P}{A} \pm \frac{M}{Z}$$

$$= \frac{4\cdot86}{1 \times 0\cdot225} \pm \frac{0\cdot65}{0\cdot0084}$$

$$= 21\cdot6 \pm 77\cdot38$$

$= 0\cdot099$ N/mm² compression or $0\cdot056$ N/mm² tension

$$\text{fall} = \frac{f_{kx}}{f_m} \approx \frac{0\cdot2}{3} = 0\cdot067 \text{ N/mm}^2$$

as $0\cdot067 > 0\cdot056$ the wall is stable under method (2)

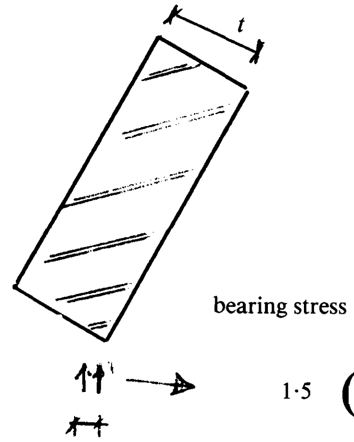
Method 3—stability moment method

$\left(\frac{1\cdot5 f_k}{\gamma_m}\right)$ for this particular masonry is taken as 3·6 N/mm²

$$X = P/1\cdot5\left(\frac{f_k}{y_m}\right)$$

$$= 4\cdot86 \text{ kN}/3600 \text{ kN/m}^2$$
$$= 0\cdot0013 \text{ m}$$

overturning moment $= 0\cdot65$ kN-m/m



$$1\cdot5\left(\frac{f_k}{y_m}\right)$$

stability moment $= W\left(\frac{t}{2} - \frac{x}{2}\right)$

$$= 4\cdot86\,(0\cdot225 - 0\cdot0013)/2$$

$$= 0\cdot54 \text{ kN}$$

as $0\cdot54 < 0\cdot65$ the wall is unstable under method (3)

From these three methods, two included the condition that no tension was to develop. The middle third rule result was completely out, while the stability moment result was not too far out. The middle third rule is based on elastic analysis and is not mentioned in BS5628. The stability moment method is based on the ultimate condition of the parapet wall, in conformity to the ultimate method of design outlined in BS5268. If no tension is to be allowed to develop because of an inadequate damp-proof sheet, the stability moment method is to be adopted, to give results more consistent with what actually happens.

If tension may develop, the stresses are obtained from an elastic analysis. The width of base necessary for stability is smaller than that obtained from the stability moment method.

We are pleased to pass on Mr Camilleri's comments.

Frost and the depth of foundations

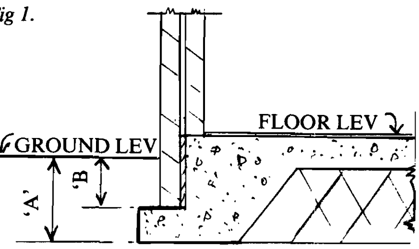
Mr Ronald Deen of Glasgow has asked for the opinion of fellow members on the appropriate cover to foundations for normal frost protection in the UK.

CP 2004:1972, section 3.2.7.2., states 'For most places in the British Isles a depth of 0·5 m (1 ft 6 in) below ground level will be sufficient for protection in this respect' and goes on to say mountainous areas, etc., could have greater frost penetration. As a 'mature engineer' who was always brought up on the adage 1 ft 6 in to the top of the foundation in former days, I should be grateful for the comments of fellow members with regard to Fig 1.

—Is the frost cover to dimension A or dimension B?

The same section further says 'Care should be taken not to leave, during or after construction, thin concrete rafts or floor slabs exposed to long periods of frosty weather where soils subject to frost heaving

Fig 1.



are within 0·5 m (1 ft 6 in) of the concrete surface'. Obviously, if the found thickness dimension A is greater than 0·5 m, it will protrude without cover. What minimum cover (dim. A) is therefore required with regard to normal frost conditions? The question does not refer to deep permafrost conditions or foundations where special precautions need to be taken against chemical attack.

However, the presence of a water table and soils subject to 'frost heave' near the underside of the foundation would have a marked influence in design and protection of overburden.

Mr Deen concludes that, although this may appear a simple question, the comments of younger colleagues suggest otherwise. Please let us have your views.

Other foundation problems

Some aspects of foundation design were drawn to our attention by Mr A. D. Moonasingha in December. He reported some ideas regarding the role of foundation beams linking pile caps and of earth pressure from backfilled soil, which he had come across when checking a foundation design, and he sought readers' views. We have now heard from Mr Daniel C. K. Wong of Singapore, who writes:

It is quite normal to connect spread footings with ground beams so that the bending moment acting on any footing could be transmitted to the adjacent footings via the ground beams. However, it is very rare to apply the same means to pile groups. A rigid pile cap with a number of piles is most efficient in transforming any bending moment from the columns into axial loads in the piles; that is the purpose of having a rigid pile cap. To reduce the number of piles by introducing a ground beam to connect two pile caps or more is most uneconomical. If the rotation of the pile cap is to be taken by the ground beam, the latter needs to be very stiff, hence very large. The saving in cost incurred by deleting some piles may not be offset by the additional cost incurred by introducing a very large ground beam. Furthermore, if the bending moment onto the pile cap is very large, it is always more effective to increase the spacing of the piles. It should be remembered that resisting bending moment by means of moment couple in the piles is most effective, economical, and safe. I therefore agree with Mr Moonasingha's point of view.

Regarding the second issue, I did come across some engineers who suggested that the backfill around the pad footing should

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done by junior members of the profession, unchecked. A lot, including some from consulting engineers, seem to regard the local authority as the checker!

Although the Building Regulations allow designs to CP111, BS449 and CP114 at present, these will all be phased out in favour BS5628, BS5950 and BS8110. Does this mean that local authorities will have to check structures designed to the former working stress Codes in accordance with the latter limit state Codes to satisfy structural adequacy and serviceability, or can local authorities just 'fail' them? If we 'fail' them, do applicants have any redress? It seems that undue pressure will be brought to bear on local authority checkers in either event.

This problem raises the issue of professional status. If a lot of these Building Regulation applicants use 'engineers' of somewhat dubious training and experience, should not the term 'engineer' be protected like the term 'architect'? If this happened, there would be a smaller need to update CP114, etc., as there could be requirements that all calculations to be submitted must be carried out (i) by a qualified 'engineer' of 'approved' status (i.e. chartered) and (ii) to BS8110, etc. This would no doubt raise standards and perceived status, as well as pleasing consulting engineers, but the 'small businesses' previously referred to would no doubt protest about restrictive practices acting to their and the country's detriment.

Another way to satisfy current Codes would be to promote guidance for 'simple' structures like simply supported beams (of steel or concrete) and vertically loaded masonry walls or piers that would comply with BS5950, etc., but would be presented much like the timber tables in the Building Regulations. This guidance could be restricted to 'domestic', if necessary.

There have been many discussions, some reported in The Structural Engineer, *on the role of Codes and their relationship to standards of professional competence and responsibility. It is interesting to have comments from one of the frequently much criticised engineers engaged in the approval procedure. We would be pleased to hear from more of such, and also responses from others who apparently provide them with problems!*

The Code for masonry structures

We have now had a lengthy correspondence on the Code for masonry structures instigated by Mr A. N. Beal (December 1985), who has been critical of the widely differing results obtained as between CP111 and BS5628 for laterally loaded masonry wall panels. He now comments on Mr D. H. Camilleri's response, published in March:

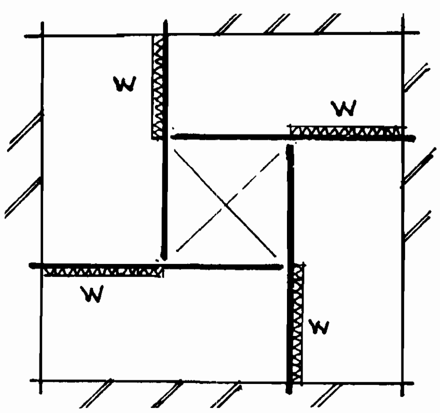
Mr Camilleri's comments are interesting but a bit confused. He shows that, for the wall he has chosen, it will carry the design loading only if reliance is placed on flexural tension and (assuming this can be developed) that stability moment and 'middle third' calculations are very conservative. This is true for this example but in no way contradicts my findings about the BS5628 rules.

BS5628 gives reasonable results (similar to CP111) where walls rely on flexural tension and have little vertical load but it gives wildly varying and highly conservative answers where vertical load is substantial, as outlined in my previous letters. Mr Camilleri's interest is welcome, but there has still been no proper response from the BS5628 committee, despite assurances that they have not abandoned their responsibilities in favour of Eurocode work. CP111 is scheduled to lose its 'approved document' status in the Building Regulations after April and then we'll be stuck with BS5628. In the circumstances I am most concerned at the lack of a satisfactory response from the BS5628 committee—if the Code is wrong (as it appears to be) should it not be amended? Is no one up there interested?

We think it could be a little unfair to call Mr Camilleri's comments confused—we thought them clear enough. Professor Hendry claimed, in his contribution in August 1986, that 'the safety factors from CP111 and BS5628 for equivalent cases obtained by Mr Beal are not grossly different' (our emphasis). Presumably, since the claim for limit state Codes is that they lead to a more consistent assessment of safety than the former working load Codes, one would not expect the nominal 'safety factors' obtained from the two Codes to be fully consistent. Are the orders of variation detailed in Mr Beal's letter of last December such as one would expect, or do they indicate the need for investigation? Should Mr Beal's prayer be answered?

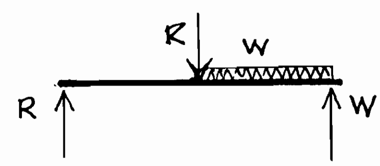
Framing a staircase

Finally this month we pass on a neat little result produced by Mr D. G. Evans of Sunbury-on-Thames, for which we thank him. The surprise or otherwise of readers when they first see Mr Evans' solution might, if it could be monitored, provide interesting commentary on the thought processes of structural engineers! How many readers would immediately say 'well, it's obvious', and how many would be puzzled?



It is required to frame in steelwork within a square stairway to provide a central lift shaft. The masonry around the stair provides only simple support, and the steel beams likewise provide purely vertical reaction.

Assuming the arrangement is symmetrical and all beams are loaded with a uniform load of W over half of their span (and of course by the support reaction from the beam which is carried), we have the interesting result that the reaction at one end must always equal the load at midspan



and no matter how the load W is altered in value or position, the right-hand reaction must always equal that load.

Book Reviews

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Bridge engineering

S. Ponnuswamy (New Delhi: Tata McGraw-Hill, 1986) 544pp. ISBN 0 07 451827 5.

As one might infer from the title, this book is intended to cover all aspects of bridge engineering: planning, investigation, design, construction, maintenance, and rebuilding. Hence, it is quite unique in that most books cover only a limited number of these aspects. However, UK readers should be warned that the book is intended for students and engineers of the Indian subcontinent. The author is well qualified to address such a readership, since he was formerly Additional General Manager (Technical) of Southern Railway, Madras. Consequently, there is a great concentration on river bridges in rural areas, and, in particular, with their foundations and construction techniques. Much of this material and much of the design section, which does not deal with the computer methods of analysis and design commonly employed in the UK, is not directly relevant to UK practice. However, the book is of immense value to UK engineers involved with projects in the subcontinent.

In a book of 544 pages, one accepts that there will be some typographical errors. However, it is annoying to find, as one does in this book, incorrect references to figures in the text and, in some cases, to figures which do not actually appear in the book. In spite of this, the reviewer would recommend it to UK engineers working in the subcontinent.

L. A. CLARK

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