FOUNDATION IMPROVEMENT TECHNIQUES FOR BRICK WALL STRUCTURES

by

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Abstract

When brick walls are constructed over weak soils, it is important to use strong foundations. The usual foundation strengthening methods such as inverted T-types or Vierendeel girder types suffer from the draw back of excessive cost. An alternative foundation strengthening method, which uses the brick wall acting together with tie beams provided at damp proof course level and window sill level, is presented. The design method for such a composite system is also presented.

1.0 Introduction

Brickwork is often used in single storey houses. It is also used as loadbearing walls in buildings with large number of partition walls such as multi-storey houses, hostels and hotels. Loadbearing brickwork structures are particularly suitable when the soil below foundation is weak since the loads are transferred as distributed loads. When weak soils are encountered, there are a number of techniques that are available to minimise the occurrence of cracks due to settlements associated with the foundations. These methods are based on improving the flexural resistance of the foundation.

A simple technique, which can be used to enhance the flexural resistance of the foundations using reinforced concrete tie beams acting in composite with brick walls, is presented. The main advantage of this technique is the use of tie beams that will serve other functions such as provision of earthquake resistance and thermal crack controlling for enhancing the flexural resistance of the foundations. Thus, the extra cost associated with foundation improvement technique can be maintained at a minimum level.

Provision of a sufficient strength for the foundation is very important since any weakness found subsequent to the construction can be very costly to rectify. If the cost due to the loss of utility during the repair is added to the actual cost of repair, elimination of foundation defects would give valuable financial benefits.

2.0 Types of cracks due to foundation movement

Any heaving or settlement of soil can lead to cracks in brick walls. Cracks can occur due to heaving of soil when foundations are laid on shrinkable clayey sub soils that are drier than normal [1]. This can be either due to abnormal climatic dry conditions or due to the ground having been cleared of large trees immediately prior to the start of the construction of the building. The subsequent wetting of clayey sub-soil is accompanied by an expansion and the ground exerts an upward pressure on the foundation thus causing vertical cracks in the brick walls.

The cracks due to heaving can be a possibility in many buildings constructed in the dry zone of Sri Lanka as soils with high clay content can shrink during the long dry spells and swell during the rainy season. It would be useful to enhance the flexural resistance of the foundations sufficiently so that this type of cracks can be minimised. It would also be useful to replace the clayey soil below the foundations with coarse sand so that the effects of moisture movement on clayey soils can be minimised. It is also advisable to clear the site well in advance so that a time lapse of about one year can be allowed before the construction commences.

Cracks due to settlement of soils is quite common when brickwork structures are constructed on weak soils without taking adequate precautions to improve the strength of foundations. When used as a loadbearing material, brickwork is characteristically

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stiff in the vertical direction and hence has only a limited tolerance for differential movement of the foundations.

It has been stated [2] that for reinforced concrete frame structures, the angular distortion (shown in **Figure 1** is more important whereas for loadbearing brickwork structures, the limiting deformation criterion defined in terms of deflection ratio as shown in **Figure 2** is more important.

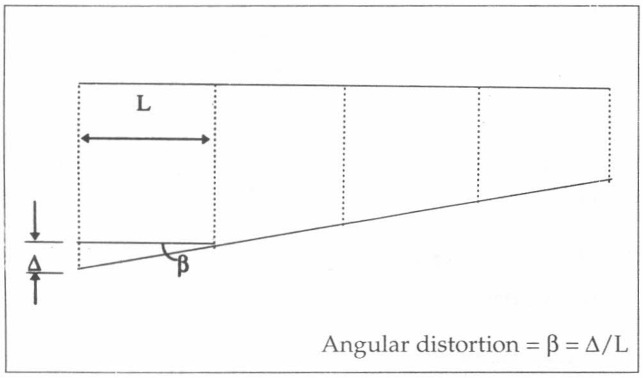

Figure 1: Definition of angular distortion for framed structures

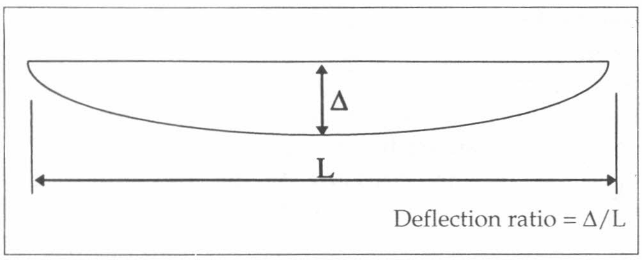

Figure 2: Definition of deflection ratio for loadbearing wall structures

A number of actual measurements made on existing structures constructed on weak soils in Sri Lanka have revealed that a deflection ratio of up to 1/2750 is acceptable for locally available building materials [3]. This value compares well with 1/2500 suggested above.

3.0 Foundation types for loadbearing brickwork structures

Some of the common types of foundations that have been used in Sri Lanka can be categorised as follows:

1. rubble foundations with damp proof course,
2. rubble foundations with plinth beams at DPC level,
3. inverted T - type foundations with reinforced concrete, and
4. Vierendeel girder type foundations with concrete beams and rubble infill.

Rubble foundations with damp-proof course and rubble foundations with plinth beams at DPC level suffer from the drawback of insufficient flexural strength and hence are of little use in weak clayey soils. The inverted T-type reinforced concrete foundations may have sufficient flexural strength, but suffer from the drawback of excessive cost. The alternative suggested to reduce the cost is the Vierendeel girder type of foundation [3].

The design method proposed in [3] for the Vierendeel girder type foundations is summarised here. The same design method is used to develop the rubble foundation and reinforced brick wall composite system.

A typical loading system for the foundation of a loadbearing wall will consist of two force systems of magnitude w_1 and w_2 as shown in **Figure 3**. The deflection associated will take the shape shown in **Figure 2**. Initially, the loading w_1 will be equal to loading w_2. With the settlement of soil that may occur

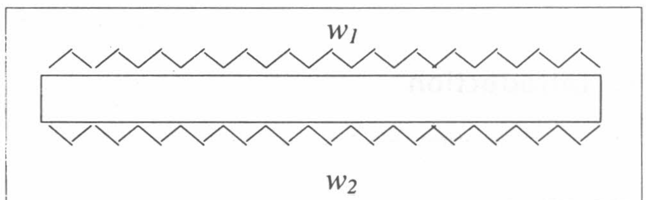

Figure 3: Loading system and deformation pattern for loadbearing wall

with time, w_1 can be different to w_2. Thus, the foundation has to be designed to resist these force systems.

The crux of the argument is that if a foundation is stiffened to resist the settlement, it should be able to resist a loading system that causes a similar deformation. The loading system considered is shown in **Figure 4.** The maximum deflection at the centre is given by $(5wl^4)/(384EI)$ for this system. This maximum deflection can be used to calculate the maximum deflection ratio which should be less than or equal to *1/2750* when constructed on soft clay [3]. This yields the following equations:

$$\Delta/L = 1/2750 = (5wL^3)/(384EI) \qquad (1)$$

$$w = (0.0279EI)/L^3 \qquad (2)$$

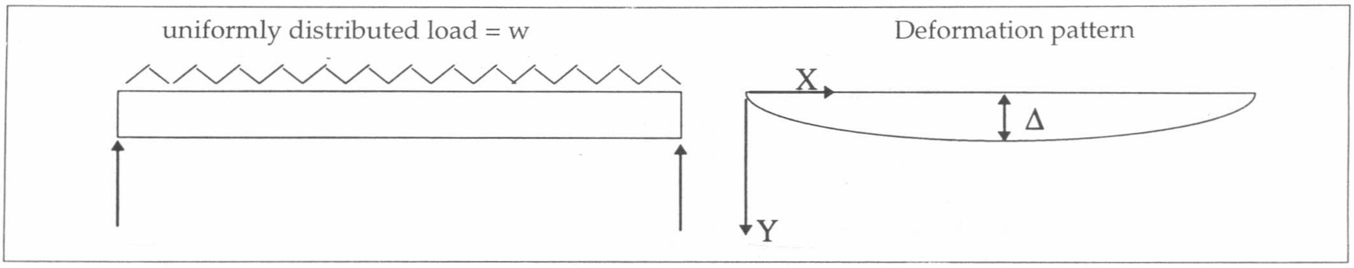

Figure 4: Loading system and deformation pattern for simply supported beam with uniformly distributed load.

This equivalent load, w, is used to calculate the maximum bending moment and the maximum shear force that should be resisted by the stiffened foundation to give rise to a deflection ratio within the allowable limits.

The Vierendeel foundation system shown in **Figure 5** can be used to provide the stiffness required at the foundation level, where the top and bottom reinforcement is calculated using the maximum bending moment given by $\gamma_f wl^2/8$, where the factor (f represents the appropriate load factor. The maximum shear force is given by $\gamma_f wl/2$. The shear stress should be sufficiently low to prevent any shear failures since no shear reinforcement is provided.

4.0 Vierendeel type foundations for brick walls

The Vierendeel type foundation as shown in **Figure 5,** consists of a reinforced concrete inverted T - beam where the webs are filled with rubble instead of concrete, thus reducing the cost. However, in order to ensure composite action of the top and bottom flanges, stub columns should be provided at appropriate intervals, generally taken to be equal to the lever arm of the foundation beam.

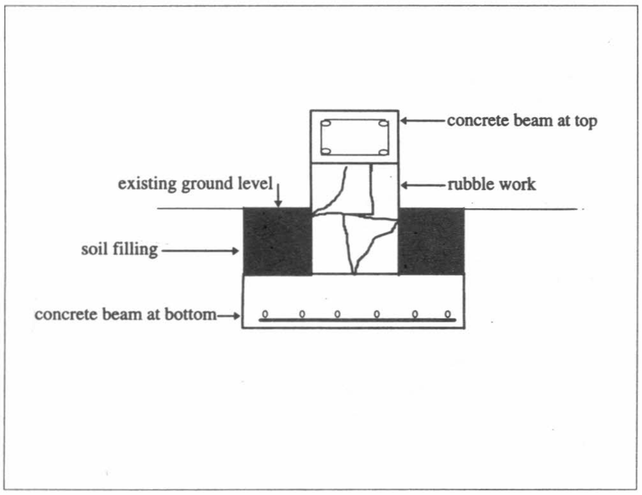

Figure 5: Vierendeel type foundation

5.0 Rubble foundation and reinforced brickwork system for weak clayey soils

The function of the Vierendeel type foundation explained in Section 4 is to stiffen the foundation so that it can resist the loads that would arise due to a limiting deflection ratio. As a result, the deflection ratio of the brick wall would be within the limits, and hence it would be possible to prevent cracking of the brickwork.

Since the cost of Vierendeel type foundations can be much more than the normal rubble foundations, an attempt has been made to introduce a composite reinforced brickwork and rubble foundation system where the brick wall has been given a flexural capacity instead of strengthening the foundation. This system makes use of the following tie beams as described below.

It is often found that when brick wall structures are constructed in weak soils, a tie beam is provided at the damp proof course level as a means of providing additional protection against settlement. This tie beam also can serve an important purpose in an earthquake prone region by tying the foundation together, thus preventing disintegration.

In brick wall buildings constructed with locally available bricks, a vertical crack generally develops close to the middle of window opening. This crack can be prevented by providing a continuous tie beam at the window sill level.

Thus, this foundation system makes use of two beams provided in properly constructed brick wall structures as shown in **Figure 6**, thus the additional cost incurred due to foundation improvements would be minimal.

When this type of composite system is selected to stiffen the structure, it is important to check the structural behaviour of the system to determine the modes of failure. The reinforced brick wall can resist flexural moments and shear forces arising out of settlement of foundations.

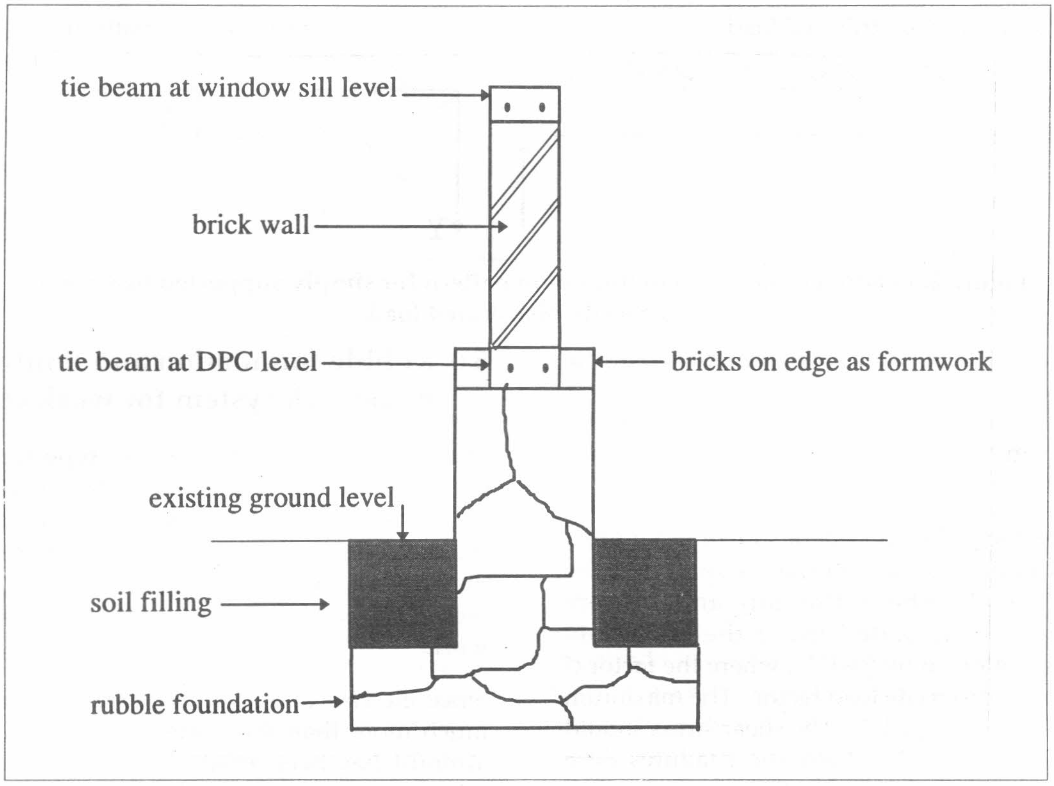

Figure 6: Rubble foundation with tie beams at DPC and window sill levels

The load w on the system can be calculated as explained in Section 3 using equation 2. This load w gives rise to a bending moment of $wl^2/8$ and a shear force of $wl/2$. The flexural moment can be resisted by providing adequate reinforcement in the tie beams. The shear force has to be resisted by the shear strength of concrete, brickwork and rubblework. Since brickwork is the weaker material, it is assumed that as soon as the shear strength of brickwork is reached, shear failure would occur.

The amount of reinforcement required can be calculated by using the bending moment calculated as above, modified by an appropriate partial safety factor. A value of *1.4* may be appropriate since one is dealing with permanent deformations caused primarily by the self weight of the structure. The lever arm used in the calculation can be based on the approximate assumption that the tie beam at the compression face is fully in compression. Once the amount of reinforcement is selected, it is possible to verify the validity of this assumption.

The shear stress on the composite system can be calculated by dividing the design shear force by the width and the total height of the composite system $[v = V/(t.H)]$.. The width is equal to the width of the brick wall and the height is equal to the height of the rubble foundation and the reinforced concrete brick wall system. Thus, the total height includes the height of two reinforced concrete tie beams, the brick wall in between the concrete beams and the height of the rubble foundation. This value should be less than the shear strength of masonry. The use of total height can be justified on the basis that the maximum shear stresses occur close to the ends of the foundation where the bending moments are almost zero, thus the rubble work is uncracked.

6.0 Shear strength of brickwork with local bricks

The shear strength of brickwork has been calculated on the basis of some research carried out on reinforced brickwork where locally available bricks have been used [4]. The shear force at failure of brick walls constructed on reinforced concrete lintels, which have failed in shear, is given in **Table 1.** The corresponding shear stresses are also given in the same table. The lintels are of size 216 mm width x 100 mm height and had 2 Nos 10 mm mild steel bars. The panels have been loaded by applying two equal point loads at one third the beam span from either support as shown in **Figure 7.** The mode of failure of these panels was the shear failure of brickwork.

It can be seen in **Table 1** that the shear stresses at serviceability limit state of cracking and ultimate failure are nearly the same. Thus, shear failure in brickwork occurs suddenly and should be prevented by keeping the shear stress sufficiently low. If a factor of safety of 1.5 is used against the shear failure, it would be possible to use a shear strength of *0.13 N/mm²* for brickwork.

Table 1
Shear forces and stresses at failure for reinforced masonry panels tested in two point loading [4]

Panel size with lintel (length x width x height) (mm)	Shear stress serviceability limit state (kN)	Shear force at serviceability limit state (N/mm^2)	Shear force at failure (kN)	Shear stress at failure (N/mm^2)
1657 x 216 x 559	23.234	0.190	24.706	0.204
1657 x 216 x 565	24.215	0.198	25.392	0.208
1655 x 214 x 560	23.850	0.199	25.071	0.209
1657 x 216 x 564	24.058	0.197	25.175	0.206

The use of *1.5* as the factor of safety against shear failure can be justified with the following argument. In this system, there are two modes of failures, flexural failure and shear failure. It would be prudent to control the area of reinforcement in the tie beams so that always the failure of the reinforced brick wall occurs in flexure with the yielding of reinforcement. Since reinforcement has a factor of safety of only *1.15*, the use of a factor of safety of *1.5* for the shear failure of masonry should be sufficient.

In order to verify the above results, another reinforced brick wall constructed using locally available bricks has been tested to failure using a similar loading arrangement [5]. The details of this test are as follows.

The reinforced brick wall panel consisted of two beams provided with *2T10* separated by a brick wall constructed with 1:6 cement-sand mortar. This panel had a lower tie beam of *100 mm* height x *200 mm* width, an upper beam of *75 mm* height *x 200 mm* width separated by a brick wall of 675mm in height and *200 mm* in thickness. The overall dimensions of the panel were *1550 mm* in length, *200 mm* in width and *850 mm* in height. This panel was tested at 28 days by applying two equal point loads at one-third the beam span from either support as shown in **Figure 7**.

In this experiment only one panel was tested since it had been reported in [4] that a number of previous researchers had used single panels for reinforced brickwork testing. The readings recorded include the load deformation behaviour, load at first crack exceeding 0.3mm and the load at failure.

The first crack of 0.3mm in width occurred at 6.6 Tonnes and failure occurred at 7.7 Tonnes. Since the load was applied at two points as shown in Figure 7, the shear force at the support was given by the total load divided by 2. The ultimate failure was due to two diagonal cracks initiated at the supports and propagated at an angle of about 45° to the horizontal as shown in Figure 7. These cracks can be attributed to a pure shear case where the shear stresses have given rise to principal tensile stresses inclined at 45° to the horizontal.

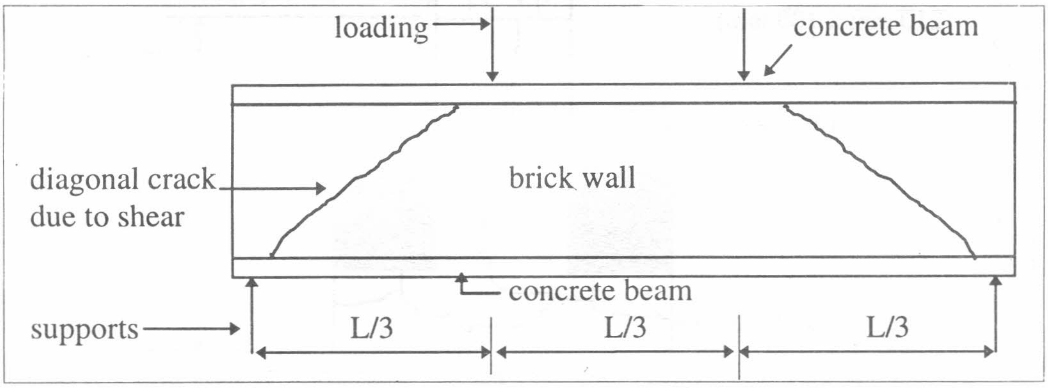

Figure 7: Testing of masonry beams in two point loading [5]

This panel gave a shear stress at serviceability failure of 0.19 N/mm^2 and an ultimate shear stress of 0.22 N/mm^2 [5]. These are approximately the same results as given in Table 1.

7.0 Application of limiting deformation ratio method for a weak clayey soil

By using the information presented above, a foundation design can be carried out as given in the following case study. The rubble foundation and the reinforced masonry wall system considered for the case study is shown in **Figure 8.**

The following assumptions have been made for the calculations:

1. The characteristic concrete strength is *20 N/mm^2* and the elastic modulus of concrete is *24 kN/mm^2*. The elastic modulus of brickwork constructed with locally available bricks is calculated using load deformation curves obtained from brick panel testing [6]. The values obtained for six panels are *0.877, 0.566, 1.129, 0.361, 0.880* and *0.881 kN/mm^2*. This gives an average value of *0.782 kN/mm^2*. Therefore, the contribution from the brickwork for the second moment of area, I, has been ignored in the calculations.
2. The second moment of area for the reinforced masonry wall system consisting of two concrete beams placed *650 mm* apart has been calculated as *4.87×10^{-3} m^4*, ignoring the contribution from the masonry wall.
3. The amount of reinforcement required is calculated by using a lever arm of z = *700 mm.*
4. The shear stress is calculated by considering that the rubble foundation acts in conjunction with the reinforced brick wall in resisting shear. Thus, the depth of the section for shear is 1725 mm and the width is 210 mm. This is a reasonable assumption since the shear failure initiates at the ends of the walls, where the bending moments are negligible.

Thus, the equivalent load acting on the composite system, w, can be calculated using equation 1, after introducing the partial factor of safety, γ_f:

$$w = \{[(\gamma_f \times 384)/(2750 \times 5)] \times E \times I\}/L^3 = 4569/L^3 \quad (3)$$

where γ_f is equal to 1.4 since a major portion of the load in a load bearing wall system consist of dead loads and self weight of brickwork.

It can be seen that the design load is a function of the length of the wall. The corresponding bending moment is given by $M = wL^2/8 = 571/L$. The shear force is given by $wL/2 = 2284/L^2$.

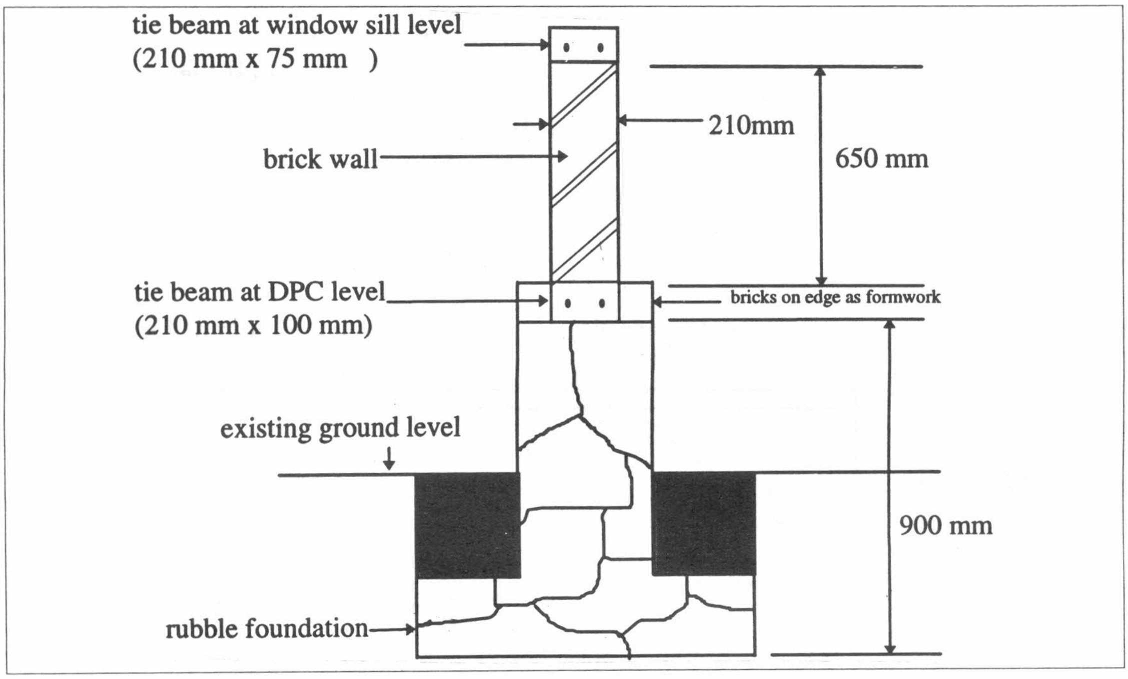

Figure 8: Rubble foundation with tie beams at DPC and window sill levels

The area of reinforcement required by the tie beam provided at the damp proof course level is given by

$$A_{s(required)} = (M \times 10^6) / (z \times 0.87 \times 460) = 3.56 \times M \quad (4)$$

The amount of reinforcement and the corresponding shear stresses are tabulated in **Table 2** for different lengths of the wall.

Table 2
The area of reinforcement and the corresponding shear stress in the rubble foundation and reinforced brick wall system for soft clay

Length of the wall (m)	Equivalent load on wall (w) (kN/m)	Design bending moment (kNm)	Design shear force (kN)	Reinforcement area required (mm^2)	Shear stress in the wall (N/mm^2)
20	0.571	28.55	5.71	101.63	0.015
15	1.353	38.05	10.1	135.45	0.028
13	2.079	43.91	13.51	156.32	0.037
12	2.644	47.59	15.86	169.42	0.044
11	3.432	51.90	18.87	184.76	0.052
10	4.569	57.11	22.84	203.31	0.063
9	6.367	64.46	28.65	229.47	0.079
8	8.923	71.38	35.69	254.11	0.098
7	13.32	81.58	46.62	290.42	0.128
6	21.15	95.17	63.45	338.80	0.175
4	71.39	142.78	142.78	508.29	0.394
3	169.2	190.35	253.80	677.64	0.700

The values given in **Table 2** can be analysed as follows. If two *10 mm* diameter high yield bars are used, which give a steel area of 157 mm^2, those can satisfy the reinforcement requirement in a wall more than 13.0m in length. The shear stress is 0.037 N/mm^2, and hence the chances for shear failure are very remote as the shear strength is more than $0.13\ N/mm^2$.

If three, *10 mm* diameter high yield bars are used, they give a steel area of $235.5\ mm^2$, which can satisfy the reinforcement requirement in a wall more than about *8.7 m* in length.

The reinforcement requirement increases rapidly for walls of shorter length when this criterion is used for the structural design. However, it should be noted that the limiting deformation criterion is applicable only for walls of length more than three times the height of the wall. Thus, the values given in the Table 2 for lengths below 8.1 m should be ignored since the minimum wall height will be 2.7 m to satisfy building regulations adopted in Sri Lanka. The walls of shorter length are less likely to deform as given above. They are more likely to behave as one unit and settle as one unit without undergoing much differential settlement.

It should be noted that there is no chance for shear failure to occur if the wall is more than *7.0m* long under the limiting deformation criterion since the shear stress is less that $0.13\ N/mm^2$.

The next question that arises is the amount of reinforcement that should be provided in the tie beams of the walls shorter than *8.1 m*. It may be based on the following criterion:

> If the wall ever displays a behaviour governed by the limiting deflection criterion, the wall should not develop any shear cracks, but should be allowed deform due to yielding of reinforcement in the tie beams.

The amount of reinforcement required in shorter walls can be calculated using this criterion in the following manner. The maximum shear force in this wall arrangement will be equal to $0.13 \times 1725 \times 210 \times 10^{-3}\ kN$. This gives a shear force value of *47 kN*. Thus the corresponding load on the reinforced brick wall can be calculated; it is given by $w = 47 \times 2/L$. This load can then be used to calculate the magnitude of the bending moment and the corresponding area of reinforcement using the equations presented above. This calculation is presented in tabular form in **Table 3.** Thus, the composite system will fail due to yielding of reinforcement rather than due to development of shear cracks. The amount of reinforcement provided should be less than the value given in Table 3. Brick walls of length less than 4.0 m are not considered since very short walls are quite unlikely to demonstrate the type of behaviour considered for the calculations.

Table 3
Reinforcement areas in tie beams to prevent shear failure in wall of length less than 8.1m.

Length (m)	Load on wall, *w*, (kN/m)	Bending moment (kNm)	Reinforcement area requried (mm^2)
7	13.4	82.1	292.3
6	15.66	70.47	250.8
5	18.8	58.75	209.1
4	23.5	47.00	167.3

From the results of Tables 2 and 3, it may be reasonable to suggest the following reinforcement amounts for the tie beam at the damp proof course for brick wall buildings constructed on soft clay; the tie beam provided at the sill level of the window can be provided with two 10 mm bars which would be sufficient to prevent thermal cracks in a brick wall:

1. For walls of length up to *5.0m*, use two *10 mm* diameter bars *(2T10)*; shear failure of brick wall governs.
2. For walls of length from *5.0 m* up to *13.0 m*, use three *10 mm* diameter bars *(3T10)*; shear failure or flexural failure of reinforced brick wall governs depending on the length.
3. For walls of length more than 13.0 m, use two 10 mm diameter bars (2T10); flexural failure of reinforced brick wall governs.

These calculations and recommendations are applicable only for this case study. For other cases with different dimensions, similar calculations should be performed. It is strongly recommended to improve the foundation with a compacted bed of sand, whenever a very soft clay is encountered.

It can be seen that this rubble foundation and reinforced masonry wall system makes use of two tie beams; one is provided at damp proof course to enhance the resistance to tensile forces induced by earthquakes or thermal effects and the other is provided at the window sill level to prevent thermal cracking. The resulting system will have enhanced flexural resistance to satisfy the strength requirements for limiting deformation criterion. Thus, this system allows strengthening the foundation at no or little extra cost. Therefore, it can be highly recommended to be adopted for brick wall structures.

It should be noted that the two tie beams are properly confined by the rubble foundation and the weight of brickwork from above. Therefore, stub columns would not be required to ensure proper composite action.

It may be possible to obtain satisfactory results using this rubble foundation and reinforced brick wall system for structures constructed on controlled fills, where granular soil is laid and compacted thoroughly in layers on weak soils, provided that the fill thickness is considerable. When the controlled fill is about 2.0 m or more, it would be possible to confine the pressure bulb to lie within the fill itself. The depth of the pressure bulb, which is usually about *1.5 x* the width of the foundation, can be restricted by controlling the width of the foundation since the bearing capacity of the granular soil immediately below the foundation is high. However, this application will need further field trials.

The performance of this foundation in uncontrolled fills cannot be assessed since the properties of the fill can vary within itself. Expert advice should be sought if this type of foundation is adopted in such a situation.

However, there are certain practical problems associated with this foundation improvement method. It is not possible to continue the tie beam provided at the window sill level when external door openings have to be provided. At such locations, there are two alternatives:

1. The door openings allow flexible areas in the wall through which the deformations can occur without causing distress in the wall. Therefore, the two walls on either side of a door opening can be considered as two independent walls.
2. Design the rubble foundation to provide the necessary flexural strength in the region where the door opening occurs. This can be achieved by providing a Vierendeel type rubble foundation as shown in Figure 5 close to door openings. The tie beam at the window sill level can be used to provide the composite action away from the door opening

8.0 Summary

It is practically possible to construct brick wall structures which would not show any signs of defects in the form of cracking, by taking adequate precautions. These precautions should be considered prior to starting the construction of the structure since some of them are applicable to site preparation and others to the construction of foundations. Therefore, the builder has to be aware of the cracks that are likely to occur due to weak foundations and should take appropriate action to prevent the occurrence of undesirable cracking which often impairs the serviceability of brick wall structures.

The precautions that have been explained in detail can be summarised as follows:

1. The construction of crack free structures should be started at the site clearing stage. The site should be cleared of all large trees about one year prior to the construction of the structure whenever possible so that the soil will be able to regain its natural moisture content during the rainy season.

2. A thorough soil investigation should be carried out at the site to identify the suitability of the soil. This can be done easily by using trial pits where the soil samples are inspected to identify the type of soil at every 0.3 m depth up to a depth of about 1.5 m - 2.0 m, depending on the type of soil. Special precautions such as improving the soil with sand should be taken, if undesirable soil types like peaty soil or clayey materials are encountered

3. The foundation should be adequately tied so that it will be able to resist earthquake loads without disintegrating. It is also possible to improve the behaviour of the foundation further by providing it with adequate flexural capacity either using strong foundations types such as inverted-T or Vierendeel girder type with concrete beams. On the other hand, it is possible to improve the behaviour by enhancing the flexural resistance of brick wall itself by using reinforced brick walls. The foundation system should be designed as described in the case study.

The design method that can be used for the design of rubble work and reinforced masonry conposite system has been highlighted. Design guidelines also have been given for a typical case. The same foundation system would be able to resist the forces due to heaving of soil as well since, in heaving of soils, a similar set of forces will act in the reverse direction.

9.0 References

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10.0 Appendix A - Design Example: A.1

In order to illustrate the applications of the limiting deflection ratio method, the following design example is presented. A Vierendeel type foundation has to be constructed for a loadbearing wall where the foundation rests on soft clayey soil. The foundation considered has a length of 20.0 m, and consist of concrete and rubble sections as shown in Figure A.1. Determine the amount of reinforcement required. Check the foundation for shear failure.

The following design data have been used:

1. The characteristic strength of longitudinal steel = fy = 460 N/mm^2

2.. The characteristic strength of concrete = f_{cu} = 20 N/mm^2

3. The elastic modulus of concrete = 24 kN/mm^2

4. The elastic modulus of rubble work = 5 kN/mm^2

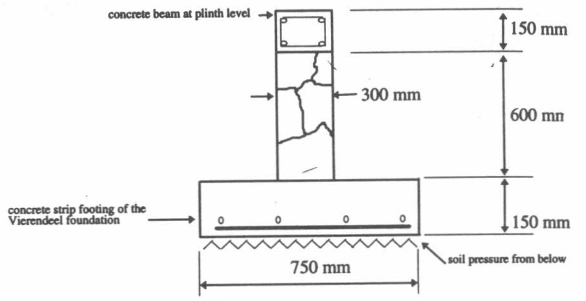

Figure A.1: The Vierendeel type foundation used for the design example

The calculations can be carried out in three main steps:

Step 1: Find EI

Concrete thickness equivalent to *300 mm* thick rubble work = *(5/24) (300) = 62.5 mm*

Distance to the centroid of the equivalent concrete section from bottom is calculated as *319 mm* by taking moments of area about the bottom.

The second moment of area is calculated as $2.0233 \times 10^{10}\ mm^4$ by using the parallel axes theorem.

Step 2: Equivalent load, the design bending moment and the shear force

The equivalent load is given by w = $0.0279EI/L^3$

$EI = 485.59 \times 10^3 \, kNm^2$

$w = 0.0279 \times EI/L^3 = 0.0279 \times 485.59 \times 10^3/20^3 = 1.693 \, kN/m$

Maximum bending moment = $wL^2/8 = 1.693 \times 20^2/8 = 84.65 \, kNm$

In order to determine the ultimate loads, the service loads are multiplied by 1.4 since the loads on a foundation of a loadbearing structure are predominantly dead and self weight.

For ultimate loads, bending moment = $84.65 \times 1.4 = 118.51 \, kNm$

Maximum shear force = $wL/2 = 1.693 \times 20/2 = 16.93 \, kN$

For ultimate loads, shear force = $16.93 \times 1.4 = 23.70 \, kN$

Step 3: Design the reinforcing steel and check for shear

It is assumed that the compression area is within the beam at top. Therefore, the lever arm between top and bottom steel = $75 + 600 + 75 = 750mm$

$A_s(0.87) f_y \times 750 = 118.51 \times 10^6$

$A_s = 394.83 \, mm^2$

Use 4T12 at top and bottom ($A_s = 452 \, mm^2$)

The maximum tensile force carried by this area of steel is $0.87 \times 460 \times 452 \times 10^{-3} = 180.9 \, kN$. The corresponding area of compression required is $(189.9 \times 10^3) / (0.4 \times f_{cu})$ = $22612 \, mm^2$ of concrete This gives a depth of $22612/300 = 75.4 \, mm$, thus the earlier assumption that the compression area is within the beam at the top is valid.

Shear stress in the rib of the foundation = $23.7 \times 10^3/(900 \times 300) = 0.087 \, N/mm^2$

This is a very small value and the rubble work should be able to resist it. In order to ensure proper Vierendeel girder action, stub columns have to be provided to connect the top and bottom reinforced concrete beams. They can be spaced at a nominal spacing of about 750 mm which is the lever arm.

The transverse reinforcement in the strip footing should be designed to resist the flexure due to soil pressure acting from below.