Articulated houses on stiffened slab foundations at Loraine gold mine, Allanridge

B E Tromp* and J T Pidgeon**

Synopsis
A relatively new and economic foundation system has been used for a recent housing contract at Loraine gold mine in the Orange Free State. Large differential ground movements caused by swelling or shrinking of expansive soils are reduced by using a stiffened raft foundation. The differential movements of the raft are sufficiently small to be accommodated by an articulated solid brick superstructure.

Design assumptions are being checked by monitoring several houses and a control area.

Samevatting
'n Betreklike nuwe en ekonomiese fondamentstelsel is gebruik vir 'n onlangse behuisingkontrakt by die Loraine goudmyn in die Oranje-Vrystaat. Deur voorstekte vlootondament te gebruik kan die omvangryke verskil in grotere gebiede vermindering word. Die ongelijke bewegings van die vlootondament is klein genoeg om opgeneem te word deur 'n geartikuleerde solide baksteenboebou. Die ontwerpaanname word gecontroleer deur die gedrag van verskeie huise en 'n kontrole gebied te monitor.

Introduction
The severe cracking of lightly loaded buildings, and in particular conventional single-storey brick houses, is a common occurrence in the OFS goldfields. The cracking and associated problems are primarily due to differential heaving of the underlying expansive soils which are common over large areas of this region. The heaving takes place as a result of an increase in volume of the expansive soil following an increase in moisture content. The main factors influencing the latter are the climatic conditions, the pre- and post-construction vegetation, the position of the water table and the nature of the soils making up the soil profile.

Because of the large investment in buildings in these areas, extensive research was carried out by the NBRi in the 1950's to find methods of construction which would prevent structures from being damaged by heave. As a result of this research various building procedures were recommended by Jennings and Evans (1962) for houses located on sites with expansive soil profiles.

These procedures were largely classified into five groups depending on the magnitude of heave likely to be encountered. In particular, the groups consisted of conventional construction, modified conventional, articulated (split) construction, articulated construction on piles to limited depth and normal construction on anchor piles. When the degree of heave warranted the use of a piled type foundation the costs of such a solution often became exorbitant, particularly when the expansive clay layer was deep, as is often the case in the OFS goldfields.

A relatively new economic foundation system has been used for a recent housing contract located on highly expansive clays at Loraine gold mine, Allanridge. Due to the problems and costs experienced by the mine over the years with existing solid brick buildings founded on conventional strip footings, it was decided to appoint consulting geotechnical engineers to evaluate and define the founding conditions for a new housing scheme involving 51 houses.

Thereafter, recommendations regarding foundation solutions were to be given which would be economical and which would give acceptable long-term performance with minimum maintenance. After extensive investigation and analysis it was decided that stiffened raft foundations would provide the best solution to the problem. In view of their research and experience in this field the NBRi were asked to assist in the design of the rafts.

Determination of site conditions
The geology of the area in which the housing scheme has been developed was established from the results of a detailed site investigation using augered large-diameter trial holes and test pits excavated by means of a backacting shovel. Most of the site is underlain by micaceous mudstones of the Dwyka Formation of the Karoo Sequence. The mudstones and shale rest uncomfortably on ansteats of the Ventersdorp Supergroup which outcrop in an elongated lobe approximately 500 m wide extending through the housing scheme in a north-south direction. In order to illustrate the relationship and extent of the underlying geology a typical idealized cross-section looking north through the township is shown in Fig 1.

Both the mudstones and the ansteats are overlain by a shallow surface layer of aeolian silt clay. A calcified horizon of moderately expansive clayey sand was encountered beneath the aeolian deposits in the areas underlain by mudstones. Although little decomposition has occurred in the ansteats the mudstones have weathered to form a deep residual soil profile comprising highly shattered and slickensided clayey silt or silty clays. A typical soil profile and information on the moisture conditions in the mudstone areas are presented in Figs 2 and 3.

During the excavation of the trial holes and test pits, both undisturbed and disturbed soil samples were taken from the various soil layers. Oedometer tests undertaken on the aeolian soils showed these to be potentially collapsible. Unfortunately, the shattered and fissured nature of the residual mudstones allowed only indicator tests to be carried out on samples from this layer. The results of the tests, however, confirmed the impressions gained on site that these soils have a high to very high potential expansiveness.

Initial prediction of heave
In the areas underlain by ansteatite it was concluded that foundation problems would be associated only with the shallow surface layer of aeolian silt sands and that only minor building precautions would be required for conventionally constructed houses founded in this area. In the areas underlain by the expansive residual mudstones, calculations based on unit heave curves for the OFS goldfields published by

*Partner, Schwatz Tromp and Associates, Johannesburg
**Senior Chief Research Officer, National Building Research Institute, CSIR, Pretoria

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Jennings and Kerrich (1962) predicted a total vertical heave of between 40 and 75 mm and, using Van der Merwe’s method (1964), a value of 40 mm was estimated. Although it was considered that collapse settlement could occur in the aeolian sands overlying the residual mudstones, it was decided that this should be treated as a separate problem and should not be relied upon to alleviate the potential heave.

Based on the results of the calculations and visual surveys of the effects of heave movement on existing buildings in the area, such as illustrated in Plate 1, it was estimated that the actual total heave was likely to be in excess of 60 mm. This estimate was used in the initial appraisal of the problem, but it was realized that a more sophisticated investigation would be required to define the magnitude of heave and the input parameters for the design of foundations more accurately.

Selection of foundation treatment

At this initial stage it was considered that, because of the large predicted heave, only two foundation options commonly used in South Africa could be guaranteed to provide satisfactory superstructure performance.

The choice was between: 1) anchor piles and suspended floors, and 2) piles of a limited length with suspended floors and articulated superstructure. In view of the depth of the expansive soil layer and the position of the water table, the first option was estimated to cost about R125/m². The second option did not provide a significant cost saving, because in order to ensure that differential movement transmitted to the structure did not exceed 25 mm, the piles would have had to be founded at a depth of 7 m. It was calculated that this type of foundation would cost about R100/m².

The cost involved in using either of the options was judged to be unacceptably high and a viable alternative had to be found. Attention was consequently focussed on the stiffened slab foundation, which had not been used extensively in South Africa to date, although it had been used successfully for many years in both the USA and Australia. It was estimated that, for the range of beam sizes and spacings likely to be used on this contract, the cost of the raft would be about R45/m².

The stiffened slab foundation

A typical stiffened slab foundation comprises a grid of beams with spacings ranging from 3 m to 4.5 m cast integrally with a floor slab, which normally has a thickness of between 100 and 150 mm. The plan configurations of stiffening beams adopted in the Type 21 houses and garages at Allanridge are shown in Figs 4a & 4b and a cross-section through the raft is shown in Fig 5.
The raft is not intended to be a completely rigid element, this would be inordinately expensive and thus it would lose its main advantage over other systems. The idea is to provide an adequate degree of flexural and torsional stiffness in order to limit the amount of differential ground movement transmitted to the structure to a level which it can accommodate. It is a general rule that the more flexible the structure the more economical the foundation, but the degree of flexibility that can be provided is normally restricted by architectural requirements for materials and aesthetic considerations.

In South Africa, the traditional dwelling has a solid brick type of construction, and to make this very brittle and rigid type of structure more flexible, it must be articulated. A plan of a Type 21 house at the Loraine mine showing the articulation and typical joint details is given in Fig 6.

It should be noted that the basic concept of articulating structures on stiffened rafts differs from the jointing of buildings in the 'split construction' method, which may be used in cases where the predicted differential heave does not exceed 15 mm. The stiffened raft is designed to deform reasonably uniformly and by an amount which would cause joints spaced at between 3 m and 4 m to move approximately 3 to 4 mm only. Consequently these joints can be more conveniently positioned and easily disguised internally, while being aesthetically acceptable externally.

In order to achieve an economical and reliable foundation design, a

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**Plate 1:** Severe external cracking in hostel block at Allanridge

**Fig 4b:** Actual layout of stiffening beams in garage of Type 21 house

**Fig 5:** Idealized cross-section through a stiffened raft foundation (Type 21 house)

**Fig 3:** Variation of soil suctions in trial holes A and B, Allanridge

**Fig 4a:** Actual layout of stiffening beams in Type 21 house
fairly sophisticated soil/structure interaction analysis is required, which takes into account an accurate assessment of the worst support condition that the raft is likely to experience.

Additional site investigations
After identifying the nature and magnitude of the founding problems and selecting the appropriate foundation and structural treatments, it was considered necessary to carry out additional site investigations to obtain input parameters for the computer program used in the design process. Two large diameter boreholes were augered to depths of 13.8 m (Trial Hole A) and 14.2 m (Trial Hole B). Samples were again taken for the determination of Atterberg limits and moisture contents, and Wescor thermocouple psychrometers were installed in the sides of the boreholes at regular intervals to a depth of 12 m in order to obtain information on the total soil suction profile.

The recorded measurements are shown in Figs 2 and 3. These indicated that the suctions significantly exceeded those corresponding to the negative hydrostatic pore pressure in both holes. The suction profile in trial hole B tied in well with the observed water table at 10.7 m, while in trial hole A, where no water table was encountered, the suction measurements predicted the water table to be at about 18 m. In both cases there was an upward flow of moisture caused by evapotranspiration and this was in a steady state flow condition, with velocities changing only at the boundaries of different materials in the profile (see Fig 3).

The tendency for moisture contents to remain fairly constant below 6 m, while the suction decreased, indicates the influence of increasing total stresses with depth. It was concluded that suction measurements on undisturbed but stress relieved samples could greatly overestimate the in situ suctions, and would consequently result in an overprediction of heave when using suction methods.

A series of lateral plate-jacking tests was carried out to obtain information on the elastic moduli of the various soil layers in the soil profile. Results were obtained for three layers, the surface aeolian deposits gave values ranging between 2 and 10 MPa, the underlying calcified clayey sand horizon gave a modulus value between 30 and 60 MPa, and the residual mudstones between 15 and 50 MPa.

Design of the stiffened slab
The design information required for a rational swell- or shrink-underload analysis following the approach outlined by Pidgeon (1980a) was

![Diagram of a slab with dimensions and annotations](image)

**Fig 6:** Articulation details for houses at Loraine gold mine, Allanridge

- **Fig 7:** Idealised initial centre dome mound shape

neither available nor easily obtainable. It was decided, therefore, to use a plate-on-mound method of analysis. It was felt that, in any event, the latter would be slightly more conservative than the former, provided that the initial mound shape could be predicted as accurately as the input parameters for the rational approach could be determined.

In the plate-on-mound method, the initial mound is taken to be the most severe that could be considered likely to form in the long term under a flexible, impermeable and unloaded area with plan dimensions equal to that of the proposed building. The assumed mound shape has the form shown in Fig 7. Although it was appreciated that because of the water table influence the top of the mound would probably not be perfectly flat, it was considered to be a sufficiently close approximation, and thus the shape could be defined by two parameters, the edge moisture variation distance (e) and the differential heave (Yn). It was also assumed that the curved portions of the mound were parabolic and that (e) remained a constant value around all the edges and in the vicinity of external and re-entrant corners.

The maximum differential heave was calculated by making the assumption that the equilibrium suction profile under the centre of the covered area would in the long term coincide with the negative extension of the hydrostatic pressure curve corresponding to a water table at approximately 10 m below ground level. The difference in suctions between the values existing at the time of the site investigation and the final equilibrium values were used in the equation proposed by Brackley (1980) to determine the total heave at the centre:

\[
S_{\text{per cent}} = \frac{P_I - 10}{10} \log \frac{s}{p}
\]

where \(P_I\) = the plasticity index
\(s\) = the soil suction in kPa
\(p\) = the overburden pressure in kPa

At the edge of the covered area it was assumed that the most likely condition was one where the soil suction at the ground surface equaled pH 4.2, this being the permanent wilting point of most shrubs and grasses normally found in gardens.

The assumption resulted in the prediction of a small amount of heave at the edge, even though it represented the driest possible condition following construction, this being caused by the veld grasses having created very high soil suctions, which were expected to diminish following their removal and the subsequent occurrence of rainfall. The difference between the heave at the centre of the area and the edge represents the differential heave and was calculated to be of the order of 60 mm.

With regard to the edge moisture variation distance (e), the following sources of information were considered:

1. Measurements taken on distressed buildings in Allanridge
2. Performance of flexible covered areas reported by de Brujin (1968-1974)
3. Recommendations made in the PTI design method (PTI, 1978)

A value for e of 2 m was chosen.

The interaction analysis between the raft and the mound was carried out using the finite element computer program, FOCALS. The raft was modelled using beam and plate elements and the soil by a layered elastic continuum. The mound was simulated by using an uplift facility in the program, but this has the limitation that only mounds with a rectangular cross-section can be modelled. It is therefore necessary first of all to
determine the ‘e’ value of the rectangular mound which is equivalent in terms of support conditions to the assumed initial mound shape.

The graphical technique which was employed for determining ‘e’ is illustrated in Fig 8 and discussed in more detail by Pidgeon (1980b, 1983). This method has several advantages over those proposed by Fraser & Wardle (1975) and Pitt (1982). Firstly, accuracy is not controlled by the chosen grid spacing. Secondly, the influence of varying the initial mound shape can be simply investigated without additional computer runs and thirdly, the computer run time is much less than that required for an iterative solution.

The soil mound and profile was represented by a three-layered model of finite depth and infinite lateral extent. The surface deposit was represented by a 2.3 m thick layer with an elastic modulus of 5 MPa, the calcified clay-sand horizon by a 2.9 m thick layer with a modulus of 30 MPa, and the residual mudstones by a 10 m layer with a modulus of 15 MPa. Poisson’s ratio of the surface and calcified layers was taken as 0.25 and of the mudstones as 0.3.

The accuracy with which the raft can be idealized for computer analysis was restricted by the chosen grid spacing and the actual location of the stiffening beams. The final beam positions chosen for the Type 21 house are shown in Fig 4; this can be compared with the idealized foundation used in the analysis, which is shown in Fig 9.

**Contract details**

A total of 35 of the 51 houses were located on the weathered mudstones and these were provided with the stiffened slab foundations. The remaining houses were of a slightly modified standard type of construction, with particular attention being paid to surface and subsurface drainage details to reduce the possibility of collapse settlement.

The plan area of the houses (not including the garages) ranged from 120 to 250 m², and the cost of the completed structures was approximately R440/m². The average contract price for the raft foundations was R55/m², slightly higher than the estimated cost, but still considerably cheaper than any of the alternatives. The first houses were completed in March 1983. The total value of the contract was R2.5 million. The raft foundations and the houses in various stages of construction are shown in Plates 2 to 6.

**Monitoring**

In order to establish the accuracy of the input parameters used in the computer analysis and confirm the predictions made regarding ground and foundation movements, three of the houses constructed on rafts together with a flexible impermeable and unloaded control area are being monitored on a regular basis. Precise level observations are being taken on a 1 m grid covering the entire control area, which has a shape corresponding closely to that of a Type 21 house, garage and patio area.

After a period of one year the area has undergone large differential movements which are shown in Fig 10. The maximum heave recorded under the cover is 50.3 mm, while up to 9 mm shrinkage has occurred along the edge of the area. The latter indicates the influence of the indigenous grass cover which in the drought conditions currently being experienced is causing desiccation of the expansive soil layers relative to the start-off conditions.

Moisture access tubes have been installed to facilitate the monitoring of moisture content changes taking place under and adjacent to the control area using a neutron moisture probe. The readings taken so far indicate that under open veld conditions heave of the ground surface takes place only as a result of fluctuations in the water table. Heave under the impermeable cover is being caused by lateral and vertical moisture migration of surface moisture through the permeable upper soil layer into the underlying medium expansive clayey sand and at the same time upward migration of moisture from the water table into the highly expansive residual mudstones.

Heave exceeding that already measured is not expected to be great. The maximum total predicted heave of 75 mm probably not being realized; however, the worst differential heave as measured across the entire control area has already reached 60 mm. Movements of the house foundations are being monitored by taking precise levels on a plug-in level peg which is inserted into machined brass tubes located at 1 m spacings in the sides of the rafts. Unfortunately only movements around the outside edges of the rafts are being measured, but these are still yielding valuable information.

The distorted shapes of house foundations H1 and H2 are shown in Figs 11 and 12. From these it can be seen that both H1 and H2 are tilting towards the north, but that H1 has heaved on the south side by a
Plate 2: Excavation for stiffened raft foundation with underfloor PVC sheeting in position

Plate 3: Concrete being placed in a garage foundation. The layout of steel reinforcement can be clearly seen

Plate 4: Completed raft being cured by ponding

Plate 5: A typical construction joint between internal and external brickwork

Plate 6: Internal doors with fanlights to ceiling level provide internal articulation

Fig 10: Contour levels for flexible control area at Allanridge: 15 February 1982 to 3 February 1983

Contour Int. = 5.00 mm
- Max. Curv. on edge = 5.55 km
- Max. Curv. on area = 13.00 km
Max. Heave = 50.3 mm
Min. Heave = -9.0 mm
maximum of 50.5 mm and on the north by 31 mm compared with 17 mm and 11 mm respectively on H2. In addition, H1 is displaying a hogging mode of distortion with a deflection ratio of 1:1800 in the long direction, while H2 is experiencing a sagging mode with a deflection ratio of 1:2200. This may be compared with the design value of 1:1000 hogging, which was considered the maximum capable of being accommodated by articulated solid brick construction.

The joints provided in the structures to allow for the expected raft movements have been instrumented to enable their performance relative to vertical foundation displacements to be gauged. These are indicating that the joints are functioning as they were intended to do and that movements of up to 3.0 mm have taken place without causing any aesthetic damage. Thermal movements exceeding 0.5 mm have been measured on joints located on the north-facing walls.

**Conclusion**

The design of an economical stiffened slab foundation depends for the most part on the selection of two factors. The first is the choice of the allowable deflection ratio, which is controlled by the type of superstructure adopted, and the second is an accurate assessment of the worst support condition which the raft is likely to experience. The former is essentially in the hands of the client and his architect, while research is currently under way to provide design guidance on the latter. It should be noted that the information required from the site investigation in the raft design process is much more detailed than that normally required for initial foundation selection.

It would appear that there is scope for reducing the cost of construction. As this type of foundation becomes more popular and experience is gained in its construction, designs will be modified on the basis of feedback from the site, and techniques will be developed for increasing productivity, thus hopefully resulting in a cheaper foundation. A similar process is likely to occur with regard to the articulation details of the superstructure. In this project, the houses had been designed before it was appreciated that there would be a need to articulate the walls; consequently the positioning of the joints and their design were dictated by this fact.

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