DIAGNOSIS AND ASSESSMENT OF STRUCTURAL FAILURE OF THE R & D BUILDING AT RIIC, KANYE, BOTSWANA

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An investigation was carried out by University of Botswana Structural Investigation Team, to establish the cause of structural defects to the R & D Building at RIIC Campus in Kanye. The structural defects consisted of cracks in all exterior as well as most of the interior walls of the building. Cracks also appeared in some floors and the ring beam. The investigation consisted of measurements of crack widths, crack mapping, determination of wall displacements using survey instruments, and geotechnical investigation of the surrounding soil. The investigation revealed that the site was underlain by expansive soils, and the damage was caused by differential movement of the soil below foundation level due to moisture variation. This paper reports on the examinations carried out and remedial reassures were proffered.

1.0 BACKGROUND

Rural Industries Innovations Centre (RIIC) site is situated in Kanye. The site belongs to Rural Industries Promotions Company (RIPCO), which has its Head Office in Gaborone. The R & D Building is located within the RIIC site, and it is used as office block to house most of the technical staff of RIPCO. The building was designed in house and construction was completed in 1995. It is of modern construction consisting of external face brick and internal plastered stock brick. Inspection of the construction drawings showed that the building was founded on a conventional strip foundation at a depth of 900mm below foundation level. The foundation size and depth was not informed by the soil condition as no soil exploration was carried out prior to design and construction. There was no special treatment to the soil layer immediately below foundation level.

2.0 INITIAL INSPECTION

An initial inspection of the building was carried out to establish the extent of the structural damage. The subsequent sections present the visual observations.

2.1 Cracks and Their Patterns

Refer to **Fig. 2.1**

- The cracks are concentrated at the North-Western side of the building. The largest crack starts at the DPC level in the Western Wall and runs in the vertical direction, Fig 3.1(a) and (b). The crack suddenly changes direction to become horizontal and runs towards the North-Western corner of the building. This type of crack clearly shows the presence of tensile stresses in the region. The crack extends into the Northern Wall, still running horizontally, Fig. 3.1(c) and (d), and again implying tensile stresses in this region of the wall.
- Other cracks were spotted in the interior walls, most of which run in the vertical direction. Walls spanning in the North-South direction, i.e. the walls parallel to the Western Wall, contain vertical cracks, while walls in the perpendicular direction are cracked horizontally. The vertical cracks in the interior walls are narrow close to the base of the walls and widen as they go up the walls, Fig. 3.1(e).
- Some cracks were spotted in the Ring Beam, in the region around the main entrance to the building. These appear to be shear cracks, Fig. 3.1(f).
- Some minor cracks were also spotted at various other places in the structure.
- The Northern Wall appears to have moved inwards at the DPC level.

2.2 Site Topography

- Vegetation: There are trees all over the site with some very close to the building being investigated. There are patches of green vegetation indicating the existence of a shallow water table.
- Drainage: Water ponding on the site was reported to be a common problem thereby suggesting the existence of low permeability subsoil leading to poor drainage.
- Steep slopes: Generally the site slopes to the west. However the slope is not steep.
- There are no indications of ground movement in terms of ground cracks, trees leaning over the slopes, etc.
- Soil type and condition: The top soil and exposed sub soil were generally grey to light

brown in colour and of stiff consistency. This suggests the presence of clay minerals.

- State of existing buildings: Almost all buildings within the corridor of the building under investigation are showing signs of foundation distress in the form of cracks on the walls. This indicates that the site is underlain by problematic soils.

3.0 INVESTIGATIONS UNDERTAKEN

To test the hypothesis that the damage is caused by the differential movement of the soil as a result of moisture changes, it was necessary to carry out various investigations and tests. Accordingly, the following specialist investigations and tests were carried out.

- Geological maps and reports for the area were bought and studied to establish the geology of the area.
- A level survey was undertaken around the damp proof course level to establish the extent and direction of any foundation movement.
- Trial pits were excavated and logged to establish the sub-soil type and condition.
- Samples were collected from the trial pits for laboratory tests to establish the salient properties of the soil.
- Field tests in the form of the dynamic cone penetration were performed to establish the consistency of the soil below bottom of the trial pits.
- A Finite Element modeling of the structure was carried out to establish stress distribution patterns and type of stresses that could result from the hypothesized deformation. A software program called LUSAS [1] was used to carry out the analysis.

4.0 ANALYSIS OF SPECIALIST INVESTIGATIONS

The results of the various specialist investigations listed above are presented in the following sub-sections.

4.1 Level Survey

The detailed Surveying Results are presented in appendix A. The main results are presented below:

- According to the leveling results the reduced level of the Damp Proof Course is not constant, it varies.
- The lowest height of the DPC was found to be 99.453 m Course (DPC) while the highest height was found to be 99.493 m.
- Generally the height of DPC along wall 1 is lower than the height of DPC along wall 4 with an average height difference of 0.020m.
- Since we do not have the initial DPC heights at the time of construction one cannot tell whether wall 1 (Southern Wall) is sinking or wall 4 (Northern Wall) is heaving. But there is noticeable differential movement between the two walls. This difference suggests either a sink or a heave in one of the two walls.
- Wall 3 (Western Wall) reveals that the two readings at the middle are higher than those at the ends of the wall. These suggest that the ground might have heaved at those two points. There is a noticeable vertical crack around the same points which suggest a movement of the wall around those two points.
- The two corners on the southern side were found to be vertical whereas the other two corners on the northern side were found to have deviated from the vertical by 2.73' and 3.40' respectively.
- There is also a noticeable horizontal crack on wall 4. The entire wall has shifted by 2.6' after the crack moving inwards.
- From the results obtained above, it is quite evident that there is a significant movement on wall 4 and relatively little or no movement on wall 1.
- In summary one can say that the ground has heaved on the northern side of the building causing movements on wall 4 while there was relatively little or no activity at wall 1.

4.2 Ground Investigation Report

The detailed ground investigation report is presented in appendix B. The main results are as follows:

- The building is founded on silty-clay residual arkose. Therefore the sub-soil is expansive rather than collapsible.
- Further characterisation of the fines using the Atterburg limits indicate that the soil exhibits low to medium expansive potential and therefore will heave with increase in moisture content.
- There is no water table at shallow depths and therefore the heaving of the clay is triggered by some other source.
- The material in TP 4 (Fig. B1) was very wet despite the fact that the location of this trial pit has always been covered with paving blocks

to reduce the ingress of rain water to the subsoil. This high moisture content suggests an existence of a water spot nearby.

- It is considered that the material within the vicinity of the water spot heaved due to the presence of water leading to differential movement.
- The occurrence of an upward movement at TP 4 location is further demonstrated by the development of a foundation crack at the location (Figure B4).
- It is concluded that heaving of the soil surrounding the free water spot has set up stresses all over the building and hence the development of the structural cracks.

4.3 Structural Report

The detailed structural investigation report is presented in appendix C. The main results are summarized here:

- The major structural failure is in the form of cracks, measuring between 7 and 15 mm wide, in the Western and Northern Walls of the building.
- The portion of the Northern Wall below the horizontal crack is slightly leaning outwards, while at the same time sliding at the DPC level (about 15 mm inwards). This indicates that this wall is tipping over as a result of the differential movement of the foundation.
- The leaning of the Northern Wall is further evidenced by the vertical cracks in the interior walls spanning in the north-south direction. The vertical cracks in these interior walls (particularly the wall dividing the Technology Transfer Unit and the Water & Energy Room) are narrow at the base and widen as they go up the walls.
- The eastern side of the Northern Wall is more stable and shows less deformation. This is possibly being assisted by the stiffness of the Ring Beam.
- As a result of the cracks, the structure has undergone some load-redistribution, which has resulted in overstressing at other parts of the structure far from the troubled spot.
- Failure of the Northern and Western Walls has also placed more loading on the Ring Beam, resulting in development of vertical shear cracks above the main entrance.
- It is concluded that all signs of failure are a result of the foundation movement.
- Finite Element Analysis has revealed that excessive compressive and tensile stresses have been set-up in different parts the structure, as a result of the prescribed displacement at the troubled spot.

5.0 CONCLUSIONS

The findings of the three independent specialist investigations (i.e. Levelling survey, Ground investigation, and Structural analysis and modeling) agree that the root cause of the damage is heaving of the ground on the western wall towards the north west corner of the building. This is clearly demonstrated by among other things:

- High reduced levels of the DPC on the concerned area.
- Development of a foundation crack in the same area.
- Trial pit at the crack location indicated the highest moisture content. This high moisture content triggered the heaving of the surrounding soil.

6.0 REMEDIAL MEASURES

A two phase remedial approach is recommended. Phase I comprises of underpinning the existing foundation to prevent any further foundation movement while Phase II comprises of repairing the existing cracks.

6.1 Underpinning

Differential foundation movement is the root cause of the damage to the building in question. Accordingly, any remedial measure should first prevent further foundation movement. Further foundation movement can only be prevented through an additional structural support to the foundation of an existing structure. This additional structural support is generally achieved through the process commonly known as underpinning. There are a number of different techniques which can be used for foundation underpinning. The techniques range from pressure grouting to installation of different types of piles.

It should be noted that foundation underpinning is a highly specialised type of work only carried out by a few foundation repair contractors. Generally the specific method varies from one contract to the other. For this reason, no specific method is prescribed in this report.

The authors are not aware of any specialised foundation repair contractors in Botswana who can undertake underpinning. However, a few companies capable of carrying out underpinning are available in South Africa. Generally, these are design and build contractors who will design the scheme, construct and supervise the works. Given this scenario, it is recommend that:

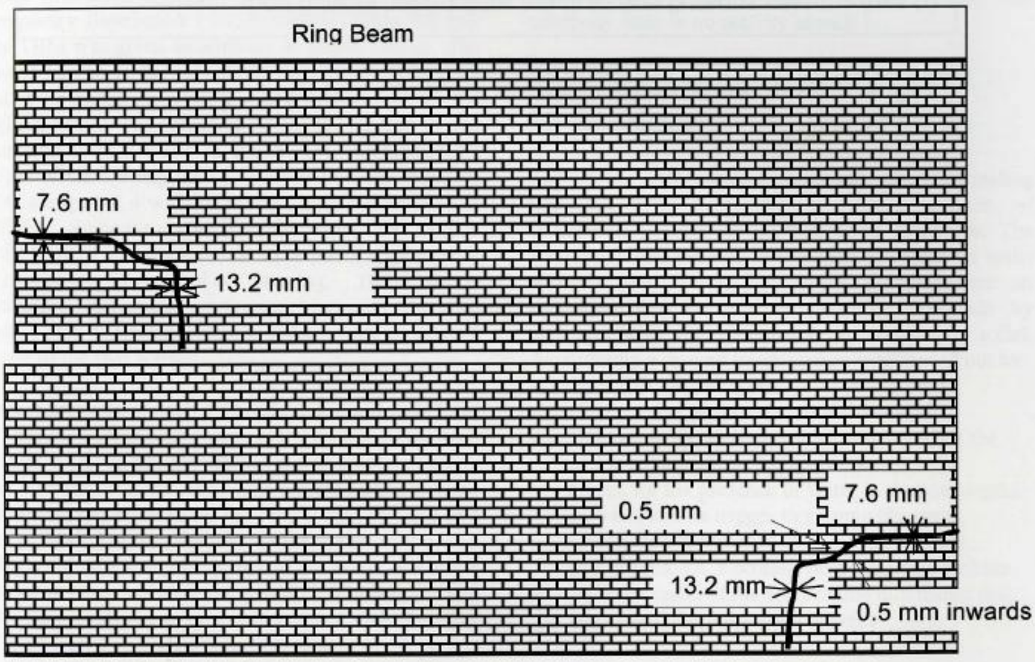
- A specialist foundation repair company should be engaged to undertake the underpinning work.
- Depending on the RIPCO regulations, the company can be selected on the basis of a tender process or a quotation scheme.
- Whatever competitive method is used to select the best contractor, the evaluation should be carried out by a qualified engineer.

6.2 Repair of the Existing Cracks

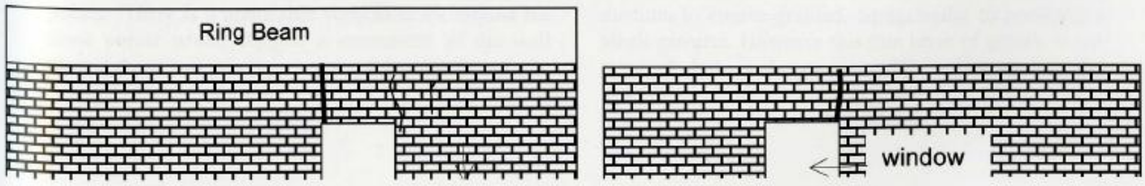
Once further foundation movement has been completely stopped, repair of the cracks can be carried out by any reputable small to medium size local contractor. Alternatively, the repair of the cracks can also be awarded to the foundation repair contractor. The foundation repair contractor may then subcontract the crack filling work to a local contractor.

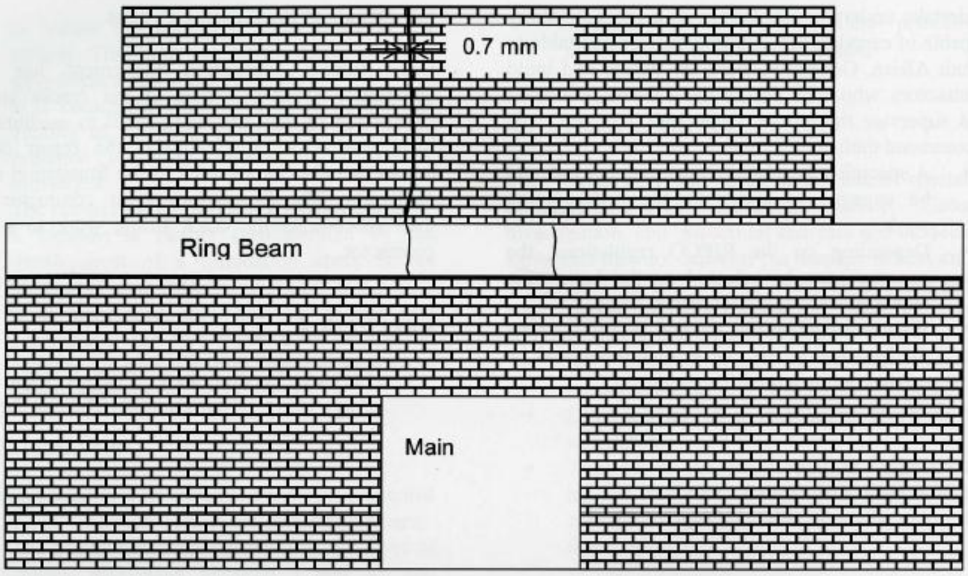
REFERENCE

1. LUSAS Version 14.3, LUSAS FEA Ltd., Forge House, Kingston upon Thames, 2010.



(b) Western Wall, looking from inside the building





(f) Eastern Wall showing cracks in the Ring Beam, looking from outside

Fig. 2.1: Sketches of major cracks in the Walls and Ring

APPENDICES

APPENDIX A: Survey Report

Methodology: Levelling

A concrete marker found on site was adopted as temporary benchmark for this particular exercise since there were no permanent benchmarks nearby. The temporary benchmark was given an arbitrary height of 100m. Levelling commenced on this point through some selected points along the DPC and closed back on same (Figure A1). The leveling results were computed as shown on Table A1 and misclosure of 3mm was obtained. The misclosure was distributed among the results and adjusted reduced levels were obtained (Table A1). The results obtained in Table A1 were later used for further analysis.

Analysis of leveling Results

According to the leveling results the reduced level of the Damp Proof Course (DPC) is not constant; the level varies with different walls. The results are based on a Temporary Benchmark (TBM) located within the site. The TBM was given an arbitrary height of 100 m. The lowest height of the DPC was found to be 99.453 m and it is found on Wall 1 (Figure A2). While the highest height was found to be 99.493 m and it was found on Wall 2 (refer to figure A3). Generally the height of DPC along wall 1 is lower that the height of DPC along wall 4 with an average height difference of 0.020m. Since we do not have the initial DPC heights at the time of construction one cannot tell whether wall 1 is sinking or wall 4 is heaving. But there is noticeable differential movement between the two walls. This difference suggests either a sink or a heave in one of the two walls.

Further analysis on wall 1 reveals that the lowest level of DPC along this wall occurs at the middle of the wall (Figure A2). A similar pattern is being noticed along wall 4 (refer to figure A4). This pattern suggests some activity taking place along the central axis of these two walls. Wall 2 shows that the northern side of the wall is higher than the southern side of the wall. A similar pattern is being noticed along wall 3 (Figures A3 and A4). This confirms an observation made earlier on that DPC of wall 4 is higher than that of wall 1. Wall 4 is on the northern side while wall 1 is the southern side. Wall 3 reveals that the two readings at the middle are higher than those at the ends of the wall. These suggest that the wall might be heaving at those two points. There is a noticeable vertical crack around the same points which suggest a movement of the wall around those two points.

Establishment of Wall Verticality

A theodolite was used to determine the verticality of the walls. This was done by determining the angle by which the walls deviated from vertical cross hair. The verticality of wall 1 was found to conform with the vertical cross hair from the top to the bottom of wall. There was no deviation from the vertical cross hair. The two corners on the southern side were found to be vertical. Whereas the other two corners on the northern side were found to deviate from the vertical by 2.73' and 3.40' respectively (refer to figures A6 and A7). There is also a noticeable horizontal crack on wall 4. The wall entire seem to have shifted by 2.6' after the crack moving inwards. From the result obtained above, it is quite evident that there is a significant movement on wall 4 and relatively no movement on wall 1. In summary, one can say that the ground has heaved on the northern side of the building causing movements on wall 4 while there was relatively little or no activity at wall 1.

APPENDIX B: Ground Investigation Report

1.0 Introduction

The initial inspection of site and the surrounding indicated that there was a general problem of foundation distress on most existing structures. The foundation distress was in the form of cracks on walls and the ring beam. Foundation distresses are an indication that the site is generally underlain by problem soils (i.e. collapsible or expansive soils). Accordingly a ground investigation was carried out to:

- Establish the soil type
- Characterise the properties of the subsoil
- Determine on the basis of the tests results if the soil is collapsible or expansive.
- Check for the presence of water at shallow depths as water acts as trigger to volume changes.
- Check for the presence of bedrock or stiffer stratum (i.e. DCP refusal) below the foundations
- Infer the cause of the soil volume movement that has caused the foundation distress

2.0 Geology of the Site

The site is situated over a fault and a thin band of medium to course grained, equigranular to porphyritic alkali granites. However this thin layer of granite is not exposed but it is overlain by sediments of the weathered cross-bedded red quartz sandstones, arkoses and conglomerates of the manyelaneng hill formation of the Waterburg Group. Outcrops of this formation are visible on site and hill where the Kanye Seventh Day Adventist Hospital is situated. Furthermore, there is prominent in-situ weathering of the arkoses sandstone and conglomerates and to lesser extend transported talus slope sediments or Hillwash.

From the above geological appraisal, it is apparent topmost rock is arkose, which is a sedimentary rock composed of sand-size fragments that contain a high proportion of feldspar in addition to quartz. It is known that the decomposition of feldspar lead to the formation of clay minerals. Therefore the resulting residual soil on the site will contain a significant amount of clay leading to foundation problems.

3.0 Investigation Procedure

Five trial pits were excavated in the existing in-situ material at locations shown on the site plan (Fig. B1). These have been designated TP1 to TP 5. The trial pits were excavated to a depth of 1.2m. Profiling of the trial pits was undertaken in accordance with Jennings et al (1973). Disturbed and undisturbed samples were recovered from the trial pits for laboratory testing.

Dynamic Cone Penetrometer (DCP) tests were carried out at the bottom of the trial pits. This was for the sole purpose of checking the existence of a harder stratum (DCP refusal) within and below the foundation stress influence zone.

4.0 RESULTS OF THE INVESTIGATIONS

4.1 Subsurface Soil Profile

The site is underlain by a thin layer of loose grave, and weathered arkoses and conglomerates of the manyelaneng hill formation overlying granite of the Gaborone group. The typical profile is summarised in Table B1. The detailed profiles of the trial pits are presented on pages 24 - 28.

4.2 Ground Water

No ground water was encountered in all the trial pits. However, the soil in TP 4 was very wet especially close to the wall. It should be noted that prior to excavation the location for TP 4 was covered by paving blocks to reduce the ingress of rain to the subsoil. Therefore the expectation was that the soil will exhibit a low moisture content compared to other trial pits. However, contrary to the expectation, the moisture content was higher than in any other trial pit. Although it was not possible to establish the exact source of the moisture, it was apparent that the source is somewhere inside the building.

The assumption of the existence of a free water spot inside the building towards the northern corner seems to agree with the information obtained from the workers at RIIC. The workers indicated that a leaking water pipe used to pass in the area where the building is located. The pipe was said to have leaked for a long time and it has now been diverted. It looks like the leakage took place at a spot near TP 4. Even when the pipe was diverted water could not easily flow out of the spot due to low permeability of the soil.

The theory of the existence of a water spot is further supported by the development of big vertical crack running through the wall to the concrete foundation (Figure B4). This is due to heaving of the wet soil to the north of TP 4 while there was no movement to the south. It is this movement that has set up stress in the wall as well as the ring beam resulting in cracks.

4.3 Foundation indicator tests

Foundation indicator tests were performed on disturbed samples taken from the bottom of the five trial pits. The overall results of the indicator tests are summarised in table 2 while the grading curves are presented in Appendix B. In exception of grading curve for TP1, the other four show similar characteristics in terms of shape. TP 1 was a little bit far from the building and that explains why the material is slightly different than that of the other four trial pits which were surrounding the building. Further analysis of the grading of the materials on which the building is founded (i.e. TP2-5) shows that 60 – 80% of the material pass the 0.425mm sieve size. Accordingly the soil can be broadly classified as fine graded. Generally fine graded soils are also cohesive as they contain appreciable clay content. Due to the presence of clay minerals the soil is expected to exhibit volume changes with the variation with moisture content thereby leading to foundation distress.

The characteristics of the fines (i.e. materials passing 0.425 sieve) were further determined through Atterburg limits tests. The tests included liquid limit (LL), plastic limit (PL), and linear shrinkage (SL). From the liquid limit and the plastic limit, the plasticity index (PI) was computed. The PI is a measure of the clay content in the soil. The higher the PI the high the clay content in a soil sample. The SL is also an indicator of the content of the clay minerals in a soil sample. A high value is obtained for samples with a significant amount of clay.

The Atterburg limits obtained are presented in Table B2. The average PI and SL values are 17% and 11 % respectively. This magnitude of PI and SL indicate that there is appreciable clay content in the soil supporting the building. Therefore differential heave resulting in wall cracks is inevitable if foundations are not design for the condition.

From indicator tests results discussed above, it can be concluded that the soil is expansive and not collapsible. Therefore the foundation problem is caused by heave induced by increase in moisture content of the soil.

4.4 Dynamic Cone Penetrometer (DCP) Tests

As already stated, DCP tests were carried out at the bottom of the trial pits to establish the existence of a hard stratum below the foundation. The tests were taken to a depth of about 1m below the foundation level. The results of the DCP tests are presented in Fig. B3. Further analysis of the results shows that the sub-soil is generally of firm to stiff consistency. However the material for TP5 is very stiff in consistency. Although no hard material (i.e. DCP refusal) was encountered in all the trial pits it is evident from DCP soundings curves that the consistency increases with depth. It is estimated that a hard horizon lies at a depth of 2 - 2.5m from ground level.

5.0 Conclusions

- The geological appraisal, profile descriptions, and indicator tests results show the presence of appreciable clay content in the sub-soil.
- Further characterisation of the fines using the Atterburg limits indicate that the soil exhibit low to medium expansive potential and therefore will heave with increase in moisture content.
- No water table was encountered at the shallow excavation depths.
- No bedrock or hard stratum within the depth of 2m from ground level.
- The material in TP 4 was very wet despite the fact that the location of this trial pit has always been covered with paving blocks to reduce the ingress of rain water to the subsoil. This high moisture content suggests an existence a water spot nearby.
- It is considered that the material within the vicinity of the water sport heaved due to the presence of water leading to differential movement.
- The occurrence of an upward movement at TP 4 location is further demonstrated by the development of a foundation crack at the location.
- It is concluded that heaving of the soil surrounding the free water sport has set up stresses all over the building and hence the development of the structural cracks.

APPENDIX C: Structural Report

C1: Structural Failure

The major structural failure is revealed by the crack that emanates in the Western Wall of the building (Fig. 3.1 in the Main Report). For the most part, the crack runs horizontally, effectively separating the structure into two potions - a portion below the crack, and another above the crack. With the structure having been separated into two pieces, each portion started to behave almost independently. The portion of the Northern Wall below the horizontal crack has slightly leaned outwards, while at same time sliding at the DPC level (about 15 mm inwards). The leaning of this wall is further evidenced by the vertical cracks in the interior walls spanning in the north-south direction. The vertical cracks in these interior walls (particularly the wall dividing the Technology Transfer Unit and the Water & Energy Room) are narrow at the base and widen as they go up the walls, see Plate C1. The portion above the crack, however, remains more stable and shows less deformation. This is possibly due to the stiffness of the Ring Beam.

Cracks in masonry buildings develop as part of the stress relief of a building. As a result of the cracks, it can be expected that the structure would undergo some load-redistribution and as such, some parts of the building would carry more load than they initially did. The horizontal crack that runs across the Northern Wall means that the wall can no longer transmit gravity loads from the roof down to the foundation. Such loads end up being transferred to other parts of the structure such as the rest of the walls and the Ring Beam. Notably, the portion of the Northern Wall above the horizontal crack is currently suspended on the Ring Beam. This extra load on the Ring Beam has resulted in the beam developing some cracks at the region above the main entrance. The loads carried by the Ring Beam also have to be transmitted down to the foundation through the undamaged walls. Additional load on the undamaged walls may cause overstressing at some areas, which can be evidenced by a cluster of fine vertical cracks or localized crushing in some regions.

C2: Finite Element Analysis

In order to gain insight into the nature of stresses resulting from the deformation of the foundation, the R&D Building was modeled in a Finite Element Analysis (FEA) computer program. A commercial finite element analysis package called LUSAS was employed to carry out the analysis. The important results required here were the direction of displacement vectors, as well as the type of stresses and their distribution patterns, but not displacement and stress magnitudes. As such, material properties were of secondary importance, hence it was not deemed necessary to obtain exact material properties from laboratory tests. However, standard values of material properties were used as input data. The geometric properties were taken from the architectural drawings of the structure. The structure was modeled with quadrilateral elements throughout, and assigned different material properties to approximate those of the real structure. Thin-shell type of elements was used. To account for the support conditions, the foundation was fixed in the two lateral directions (x and y), and assigned a spring stiffness support condition in the vertical direction to model the bearing stiffness of the underlying soil. Then prescribed displacements were defined at a portion of the foundation to model the differential vertical movement of the foundation. The only loading condition defined was the self-weight of the structural components.

Fig.C2 shows the finite element mesh of the building. The results obtained from the FEA are presented in Figs. C3 and C4. Examination of these figures reveals that the differential displacement of the foundation induces both compressive and tensile stresses in different parts the structure. Furthermore, the relative displacements follow the same pattern as those observed in the real structure.

To compute displacement and stress magnitudes using the FEA model would require more exact modeling of the structure, with all parameters properly accounted for. However, judging by the distribution pattern of the stresses and the crack sizes, it can be reasonably concluded that considerable tensile stresses had been induced in the cracked regions of the walls as a result of the foundation displacement. Hence, it cannot be ruled out that other areas which have not developed cracks are still subjected to some tensile stresses. Due to the brittleness of the material, tensile stresses must be avoided by all means in brick buildings, especially where the walls are all load-bearing. Excessive tensile stresses in other regions of the walls may lead to sudden failure (collapse) of some parts of, or the entire, building.

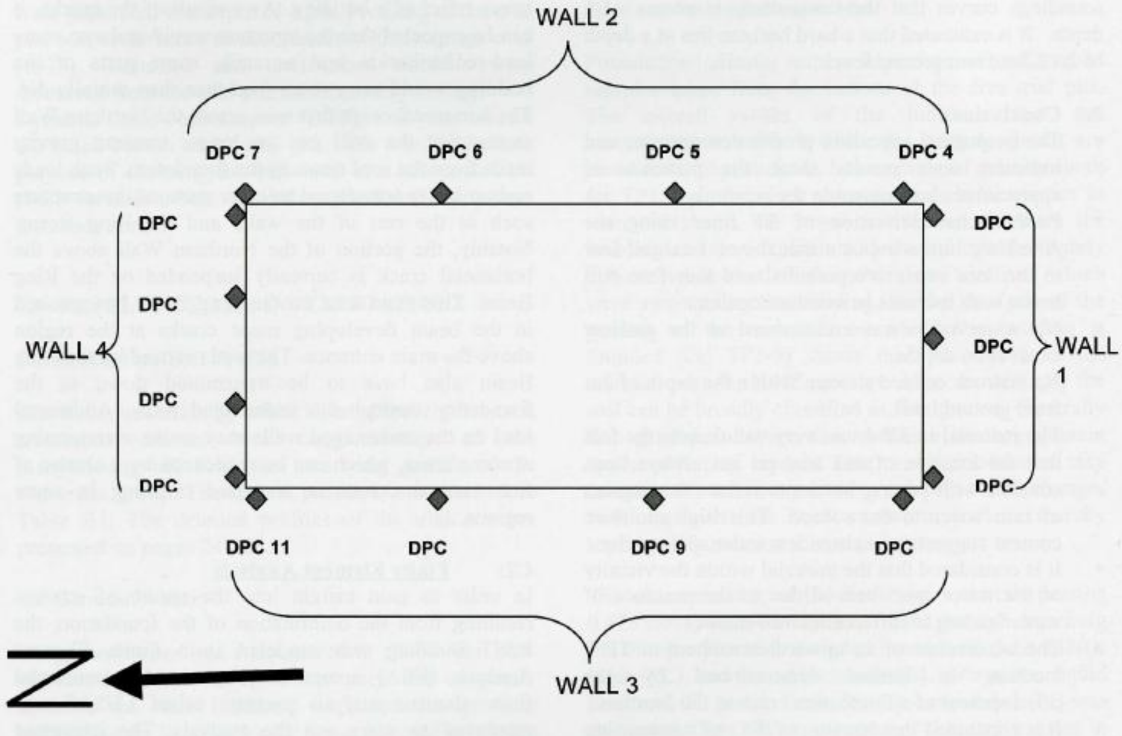


Figure A1: Sketch showing DPC Leveling Points (Not to scale)

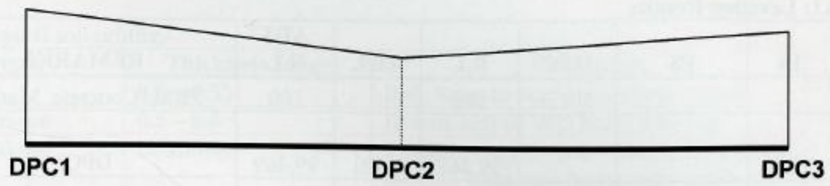


Figure A2: DPC level along wall 1 (not to scale)

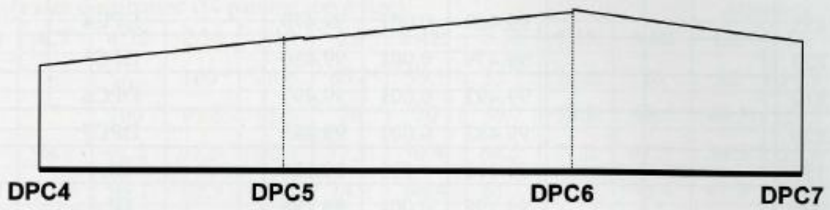


Figure A3: DPC levels along Wall 2 (not to scale)

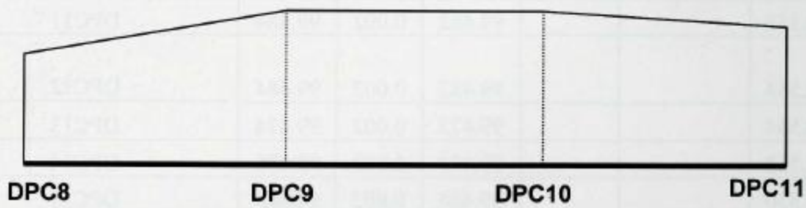


Figure A4: DPC levels along Wall 3 (not to scale)

Table A1: Leveling Results

BS	IS	FS	HPC	R.L	ADJ.	ADJ R.L	REMARKS	
1.302			101.3	100		100	TBM (Concrete Marker)	
	1.834			99.468	0.001	99.469	DPC1	
	1.850			99.452	0.001	99.453	DPC2	WALL 1
	1.845			99.457	0.001	99.458	DPC3	
	2.169			99.133	0.001	99.134	GL3	
	2.069			99.233	0.001	99.234	GL2	
	1.970			99.332	0.001	99.333	GL1	
	1.833			99.469	0.001	99.470	DPC4	
	1.823			99.479	0.001	99.480	DPC5	WALL2
	1.810			99.492	0.001	99.493	DPC6	
	1.820			99.482	0.001	99.483	DPC7	
	2.189			99.113	0.001	99.114	GL4	
	2.004			99.298	0.001	99.299	GL5	
	1.937			99.365	0.001	99.366	GL6	
0.954		2.220	100.04	99.082	0.001	99.083	CP1	
	0.579			99.457	0.002	99.459	DPC8	
	0.550			99.486	0.002	99.488	DPC9	WALL 3
	0.550			99.486	0.002	99.488	DPC10	
	0.554			99.482	0.002	99.484	DPC11	
	0.554			99.482	0.002	99.484	DPC12	
	0.564			99.472	0.002	99.474	DPC13	WALL 4
	0.563			99.473	0.002	99.475	DPC14	
	0.550			99.486	0.002	99.488	DPC15	
1.788		0.715	101.11	99.321	0.002	99.323	CP2	
		1.112		99.997	0.003	100	TBM (Concrete Marker)	

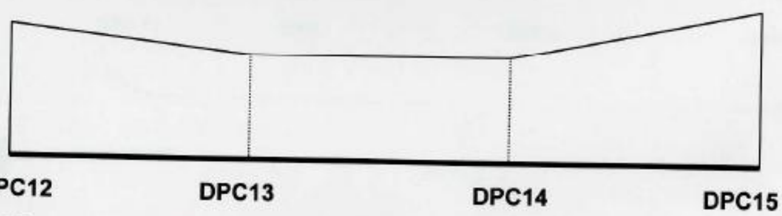


Figure A5: DPC levels along Wall 4 (not to scale)

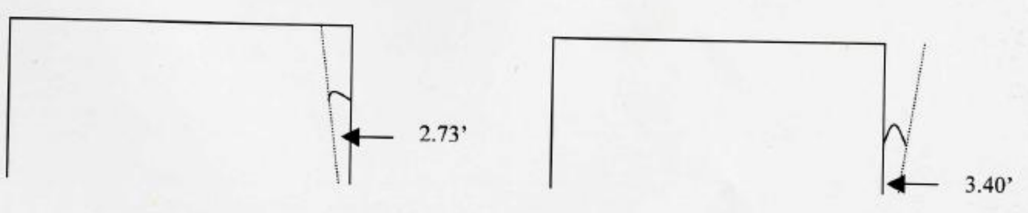


Table B1: Typical soil profile

Horizon	Thickness (mm)	Description
Colluvium	0.2 – 0.35	Red, loose to very loose sandy gravel
Residual Arkose	0.5 – 0.6	Brown, soft to stiff intact silty clay
Residual Arkose	Unknown	Reddish, dark and brown becoming mottled black and brown with depth, firm to very stiff silty clay with inclusion of calcrete.

Table B2: Summary of foundation indicator tests

	Particle size distribution (% passing sieve size)											Atterburg limits			
	13.2	9.5	6.7	4.75	2.36	1.18	0.6	0.425	0.30	0.15	0.08	LL	PL	PI	SL
TP1					100	76.6	66.7	39.7	35.7	30.6	25	32	17.9	14.1	10.6
TP2				100	97.3	91.3	76	70	59.7	53.8	46.4	44.5	27.8	16.7	12.6
TP3		99.2	98.7	98.2	97.5	92.1	77.5	70.4	64.5	57.2	47.5	36.5	15.6	20.9	10.1
TP4		97.8	97.3	96.7	95.5	91.5	74.9	66.4	61.1	53.9	43.4	41.7	20.0	21.7	11.8
TP5					99.5	95.2	65.1	57.0	53.0	45.6	35.8	38.5	25.1	13.4	9.7
											AV	38.6	21.3	17.4	11.0
											STD	4.8	5.1	3.8	1.2

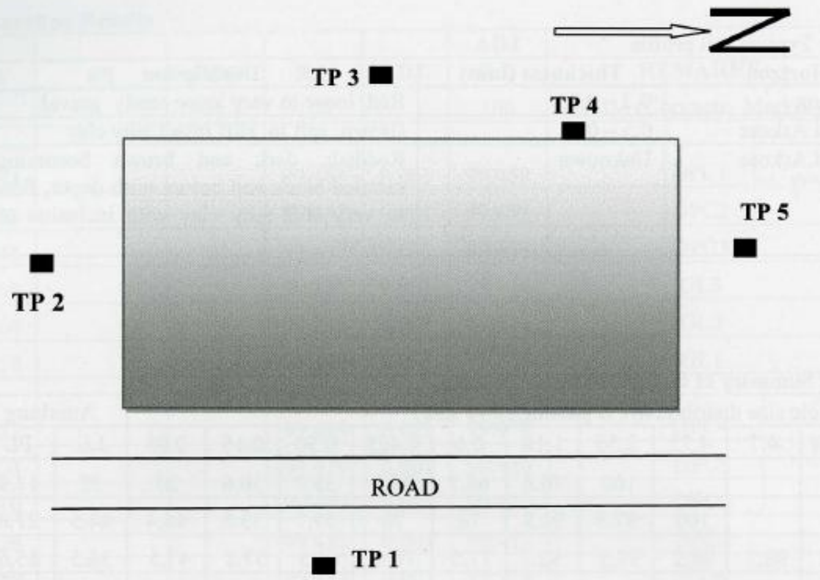


Figure B1: Trial pits locations

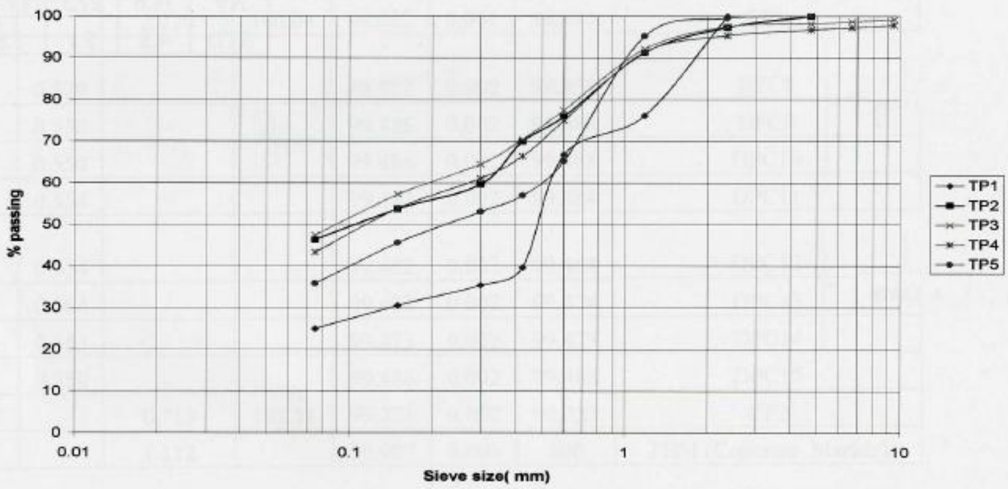


Figure B2: Grading curves

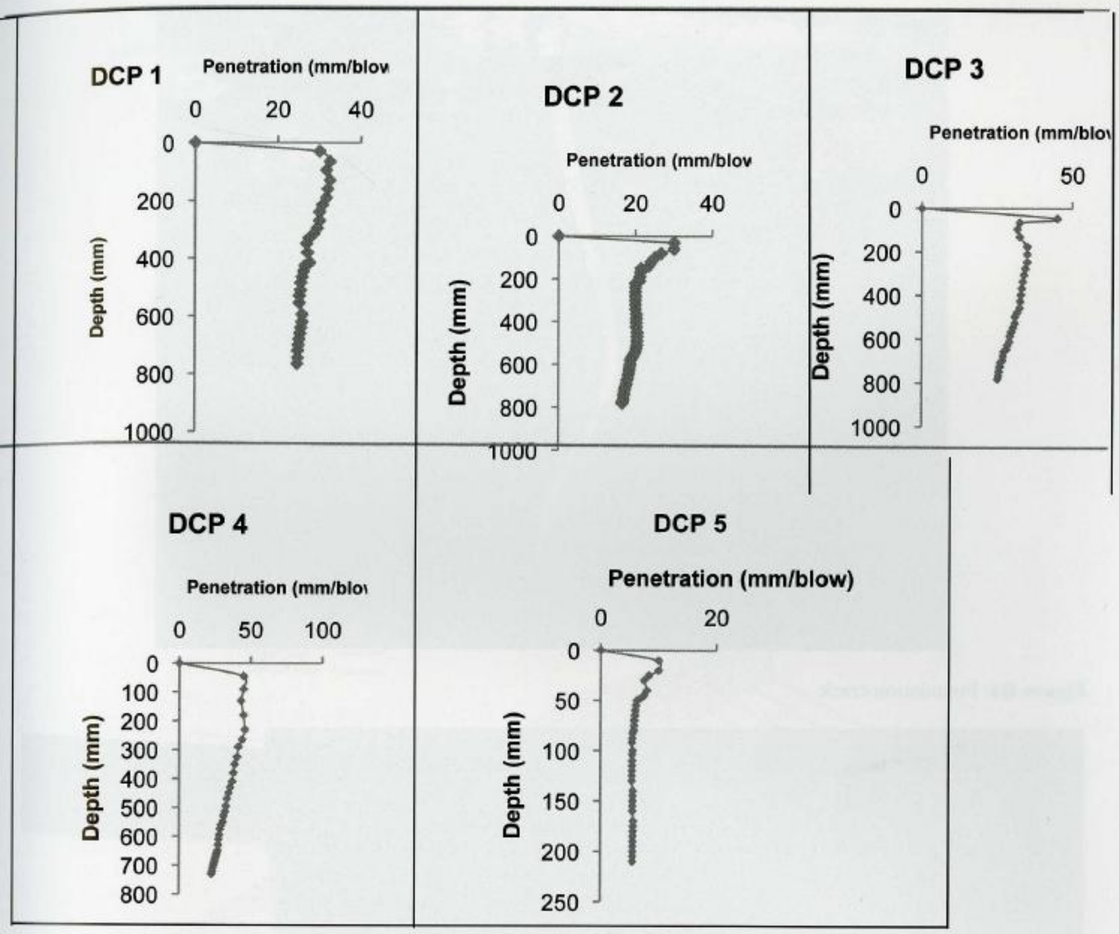


Fig. B3: DCP Tests Results

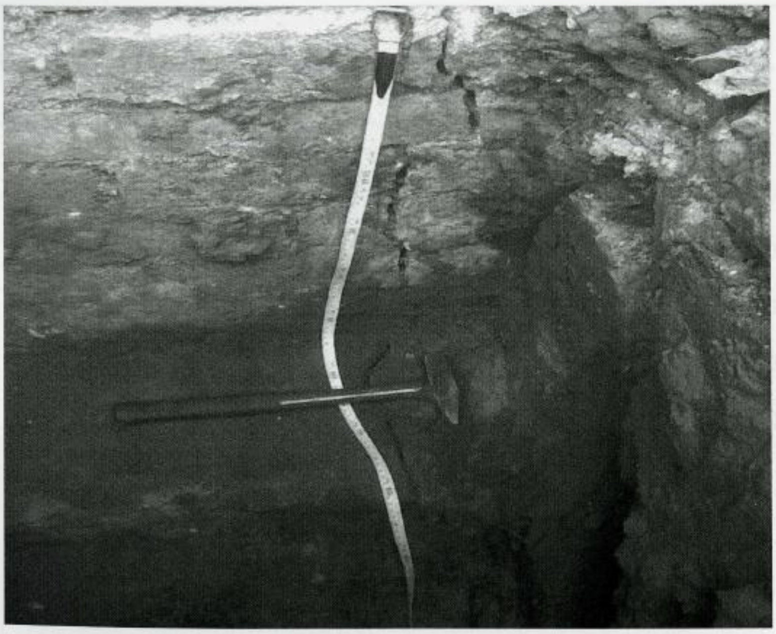


Figure B4: Foundation crack

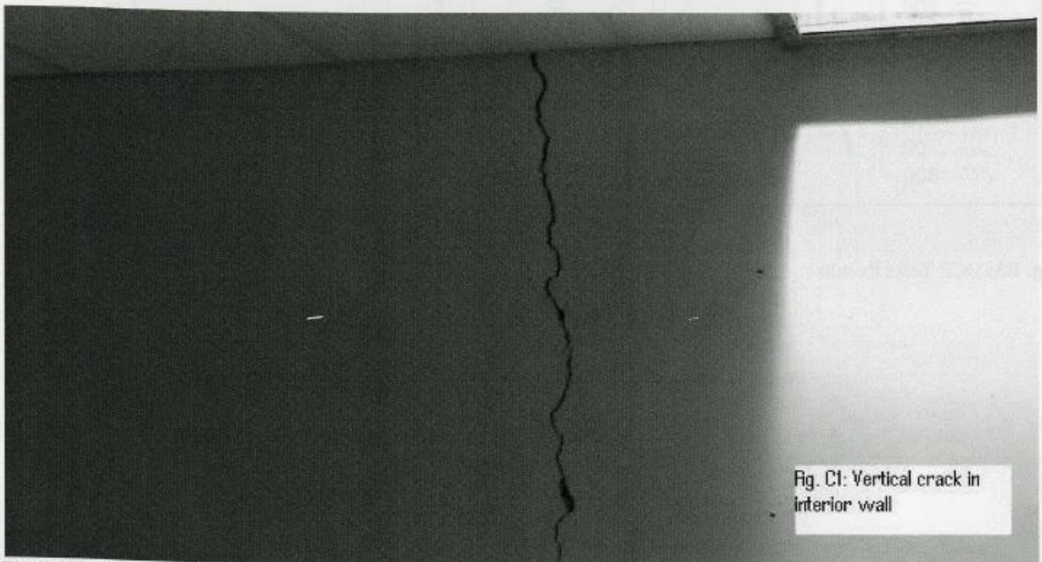


Figure C1: Interior Wall crack

Fig. C2: Finite Element Mesh

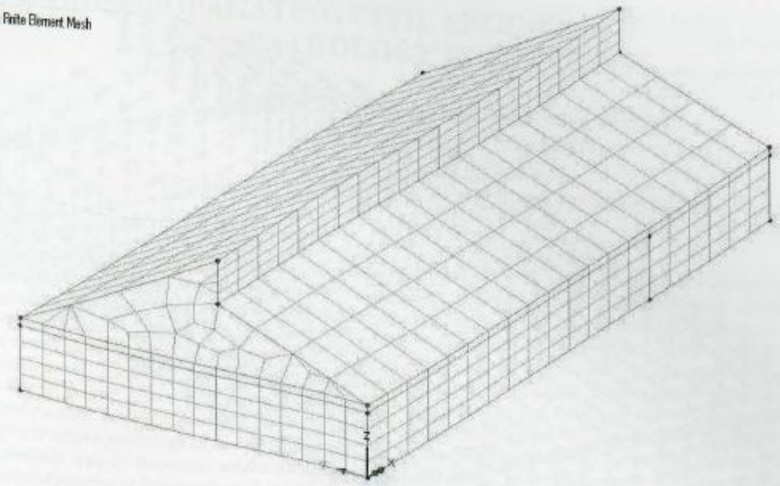
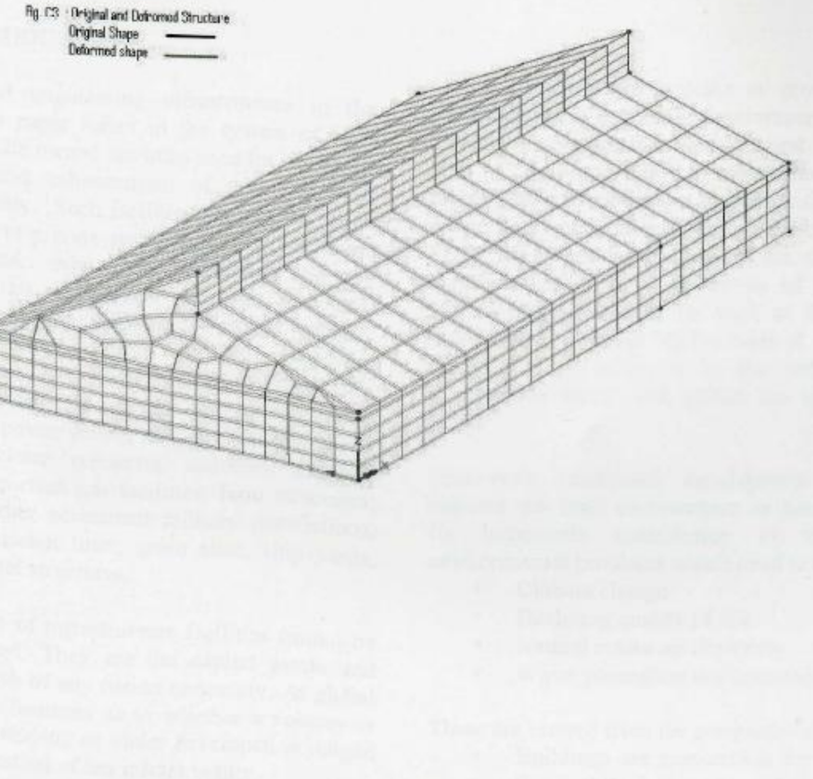


Fig. C3 : Original and Dofromed Structure
Original Shape
Deformed shape



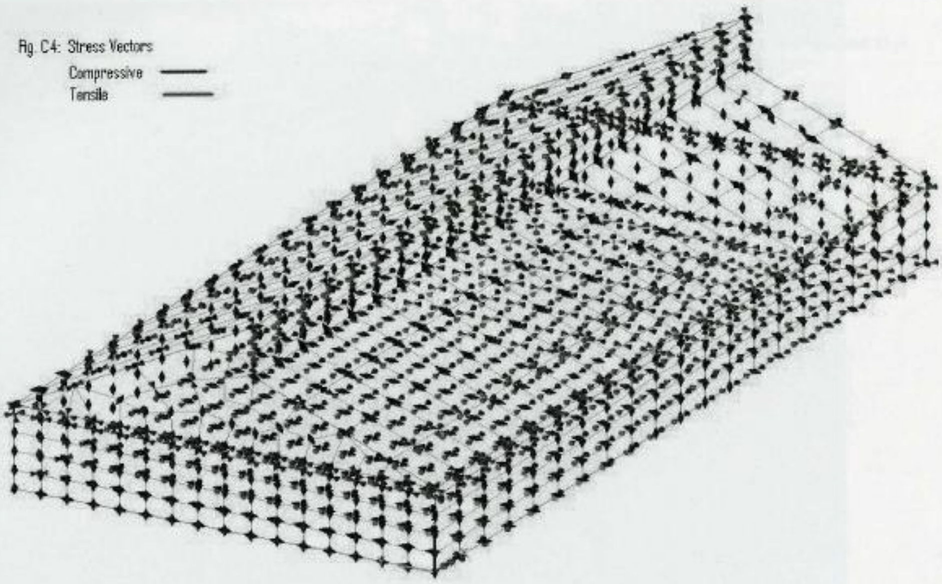


Fig. C4: Stress Vectors
Compressive
Tensile