TEST DATA

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Technical Consultancy

Structural Performance Division

Onsite Structural Testing and Assessment of the CINTEC Anchor Reinforcement System at Basildon, Essex.

By R J Currie
February 1993

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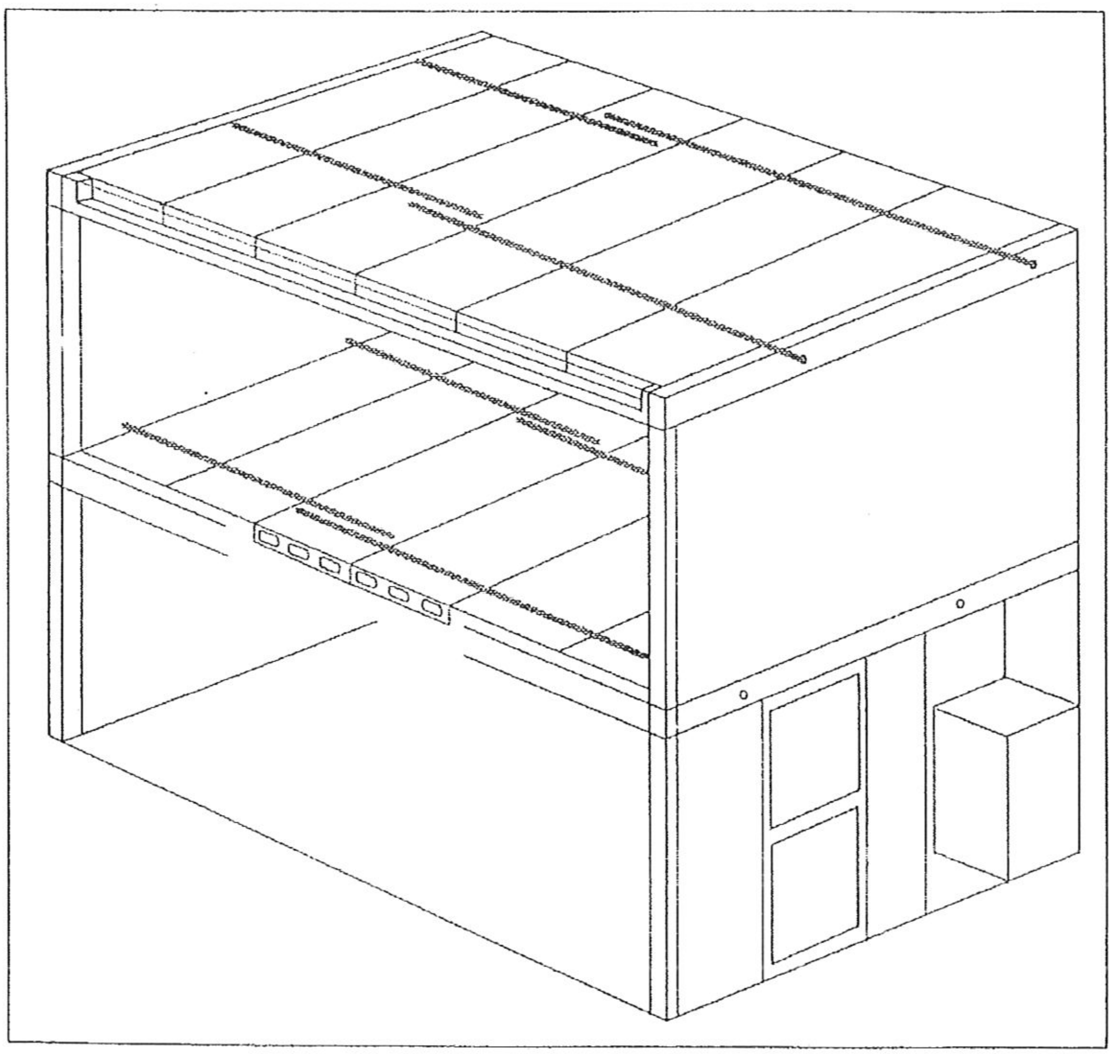

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Executive Summary

The strengthening system was developed by Scott, Wilson, Kirkpatrick & Partners using the Cintec anchor system supplied by Cintec International Ltd and was tested by BRE using dynamic and static techniques. The anchor system had been installed in the floors and roofs of an estate of large panel system dwellings at Basildon in Essex to improve the robustness of the construction.

A terrace of three dwellings was made available by the Commission for the New Towns for development and testing and the performance of the floors and roofs in two of the dwellings were assessed with and without the strengthening anchors.

Finally, collapse of the two end dwellings was initiated by removing the external ground floor load bearing walls and the influence of the anchors on the mechanism of collapse was recorded and analysed.

This work was undertaken by BRE as a collaborative project for Scott, Wilson, Kirkpatrick & Partners, the Commission for the New Towns, Building Regulations Division of DOE, the Housing Directorate of DOE and Cintec International Ltd, manufacturers and suppliers of Cintec Anchors.

Onsite Structural Testing and Assessment of a Cintec Anchor Installation at Basildon, Essex.

Background

During the course of an investigation into the condition of High Speed System Build (HSSB) dwellings in Basildon, Scott Wilson Kirkpatrick & Partners (SWK) were asked to assess the vulnerability of these concrete panel buildings to accidental loadings. It was concluded, as a result, that strengthening was required to increase the robustness to be equivalent to that of conventional domestic properties (BS8103: Part 1: 1986 Cl 4.1). An important part of the strengthening system would be to make the floors and roofs, which are constructed from precast concrete planks, act as a monolithic slab or plate. If this could be achieved, forces applied to the ends of the planks from the party walls of the dwellings could be transmitted to the shear walls which lie parallel to the direction of span of the planks and stability could be assured.

To achieve this SWK in collaboration with Cintec International Ltd developed an anchor arrangement which involved forming holes by dry diamond drilling horizontally through the floors and roofs at right angles to the direction of span of the planks and grouting in a Cintec anchor to lock the planks together and provide lateral continuity.

As the system was new and untried a block of three, two storey dwellings was selected for installation trials to establish the practicability of the system.

Testing Conducted by BRE on behalf of SWK

BRE was initially commissioned by SWK to establish if the trial installation of the anchors induced effective lateral connectivity between the planks.

To establish this BRE conducted dynamic tests on two sections of floors in areas A and B (Fig. 1) since this is the most economic and quickest form of full scale testing available.

Before the anchors were installed dynamic impact tests were conducted in rooms in areas A and B to identify the most suitable sections to test. Then a controlled forced vibration test was conducted on floors in areas A and B before the anchors were installed and the same tests repeated after installation of the anchors.

These tests established the following:-

1. The floors in end B were considerably stiffer than in end A, both before and after the anchors were installed. This may have been due to initial curvature induced in the floors in area B as a result of clay heave acting on the walls of the structure.

2. The stiffness of the floors in both areas A and B was increased by the addition of the anchors.

3. Before the installation of the anchors the deflected shape of the floors in both ends A and B was consistent with a system in which little lateral connectivity between planks was present (Fig. 2).

4. With the anchor system installed the deflected shape of the floors in areas A and B was consistent with a system with lateral continuity such as a plate (Fig. 3).

The dynamic tests therefore demonstrated that the anchors induced plate action in the plank floors which supported the presence of the lateral connectivity assumed by SWK in justifying the stability of the dwellings with the anchors installed.

As a result of these tests, and trials conducted by SWK to assess the reliability of the installation method, the whole estate of HSSB housing was strengthened using the anchor system without the need to decant the occupants.

Further Collaborative Test Programme Conducted by BRE

Background

When the strengthening work was completed on the estate an opportunity was available to conduct further testing and research on the anchor system which would enable its effectiveness to be measured right up to failure of the floors and roofs and its influence on the structures' post failure behaviour to be evaluated qualitatively.

The dynamic testing had not used all the resources set aside for testing the effectiveness of the strengthening system and as a result the Commission for the New Towns, who have responsibility for the dwellings, gave permission and support for SWK to conduct some additional testing. This was to be undertaken as part of a demolition scheme for the three dwellings which were no longer required.

SWK approached BRE with proposals with a view to commissioning a test programme.

On consideration of the SWK proposals it was clear that although BRE could devise and execute a test programme to provide the assessment an opportunity existed to demonstrate a much wider capability of the anchor system. If this proved successful it would provide building owners with an effective additional method of strengthening against loading conditions such as gas explosions, progressive collapse etc., in high rise construction.

In view of this potential the DOE (Building Regulation Division and Housing Directorates) agreed to fund some additional testing to investigate the possible wider applications. With the additional DOE funding, the SWK commission and a contribution from Cintec International Ltd. BRE put forward a proposal for a range of tests to evaluate the influence of the anchors on the structural behavior of the dwellings from the elastic range through to collapse.

Summary of Test Programme and Findings

Series 1 – Static Load Testing

There were two motives for conducting these tests. The first was to establish if the plate action induced by the anchors, indicated earlier by the dynamic testing was maintained beyond full service loads and the second was to provide additional information to complement a large programme of full scale floor testing previously conducted by BRE.

A number of load tests were conducted on floors A and B with the anchors in place which investigated; lateral load transfer under different concentrations of load, the deflected forms of the individual planks and the transverse deflected profile of the floors.

The anchors in the floors in area B were then cut and the load tests repeated on the same areas. Although the absolute deflections were still very small under these serviceability load levels the tests showed that the deflections of the planks increased when the anchors were cut and the floors reverted to a predominantly one way spanning system.

Therefore these static tests demonstrated that (a) two way spanning action was maintained up to full service loads and (b) confirmed that by cutting the anchors the floors reverted to a predominantly one way spanning system in accordance with conclusions reached from the dynamic testing.

Series 2 – Lateral Plate Action

This test was designed to check the assumption that the anchors produce plate action by connecting individual precast planks so that they may transfer "in plane" forces to the parallel shear walls.

This load testing was conducted on both ends A (anchors intact) and B (anchors cut) on the roof components with the waterproofing screed having first been cut along the lines of the plank junctions to avoid the complications of composite action with the roof finishes.

The test method employed was to apply a direct tensile force to the ends of one of the planks (Fig. 4) near the centre of the roof area and to monitor the deflected profile along the edge of the roof.

If the planks were effectively connected the end profile would be in the form of a smooth curve with evenly distributed deflections from a maximum at the point of load application to a minimum adjacent to the side walls. If unconnected, one would expect most of the movement to be concentrated at the plank subject to load application with a much poorer distribution of movement across this width of the roof.

Test 1 – End A (anchors intact).

Using the edge beams as reaction points for a deep beam a tensile load was applied incrementally to one of the central planks and the movement of all the floor planks recorded. When the applied load reached approximately 16 tonnes (T) the lateral movement of the loaded plank was 4.4mm and the profile across the end of the roof was as indicated in Fig. 5 position 1.

At this point the whole roof rapidly moved bodily on its supports a few millimeters and the load was therefore released.

Inspection revealed that the bonding between the roof and the walls underneath had been broken which allowed the central portion of the roof to move laterally relative to edge beams due to the 16T.

To see if the integrity of the ties had been maintained under this shock loading and to examine if it was mainly frictional resistance between the roof and its supports which had been overcome when the roof moved it was decided to re-test the roof.

The position in which the roof planks came to rest is given in Fig. 5 position 2 which was the new datum position of the roof for the start of the second load cycle. This shows that approximately half the deflections recorded previously were recovered when the load was removed with a permanent “set” of about 3mm between the centre and edge planks.

When the load was applied again it was clear that the roof was behaving in a similar way to the first load cycle with deformations being shared across the roof width.

The load was increased up to the previously recorded maximum of 16 T and the deformation profile was recorded Fig. 5 position 3. Position 4 (Fig. 5) shows the measurements in position 3 adjusted to allow for the initial datum position at the outset of the second load test.

As the load was increased to 16.4T, one of the threads of the rods used to apply the tensile load failed releasing the load instantaneously and the test on end A was therefore brought to an end.

Test 2 – End B (anchors cut)

Exactly the same procedure of loading and deformation recording was carried out on the roof at End B. When load was applied it was immediately apparent that greater movements than those recorded at end A were occurring.

When the maximum deflection at End B was similar to the maximum deflection recorded under the 16 T load in the previous test on End A the load was held constant. At this point the load applied to End B was 5 tons and the deformed profile at this load is given in Fig. 6 position 1.

When the loading was increased to 6 T the deformation increased dramatically as indicated in Fig. 6 position 2. At this point the load was released and the structure inspected. No recovery of the deformations occurred on removal of the load, the deformed profile being maintained under zero load.

There was no visual evidence of the floor moving on its supporting walls and to confirm the deformations being recorded by the instrumentation, a line was draw on the underside of the roof to record the relative movements between the planks.

CRITICAL STAGES IN THE COLLAPSE OF DWELLINGS AT END A WITH ANCHORS

A4
Shows the stage where virtually all the load bearing support has been removed from the end of the structure. At this point there is no significant disruption to the structure and the first floor planks have just started to deflect under the load.

A5
The last remnants of the supports are removed and the first floor starts to fail slowly under the weight of the panels above.

A8
The first floor continues to fail as a unit leaving behind the roof which is unsupported but still intact. The side walls start to buckle out at first floor level under rotational forces induced by the first floor wall panels.

A10
The first floor collapse is complete, the side walls are now buckling badly but the unsupported roof remains in place and intact.

A12
There is complete loss of stability of the side walls due to rotation at first floor level forcing down the roof structure as a unit.

A14
Collapse of the complete end of the structure. Note the upper storey side walls being pulled inwards and roof now in free fall but still continuous.

A35
Final condition of collapsed structure. Note most of the debris lies within the original dimensions of the building.

CRITICAL STAGES IN THE COLLAPSE OF DWELLINGS AT END B WITHOUT ANCHORS

B5
This shows a stage where the support wall is being removed. It can be seen that the first floor has slipped down at the side nearest the camera and the end first floor wall panels and the side walls have started to rotate.

B6
The first floor is failing rapidly as the excavator bucket attempts to remove the remaining supports and the side walls have commenced buckling at first floor level.

B7
As the failure progresses rapidly the roof breaks up and fails following the first floor down. Note the first floor centre wall panel is sandwiched between the roof and floor and the end is falling as a whole.

B8
The roof continues to follow the first floor in free fall.

B9-11

Complete collapse of the end of the building.

B36

Fragmented side walls had been contained by the safety scaffolding.

PHOTOGRAPH 1

Reduced support prior to removal of ground floor wall.

The Building Research Establishment is the main organisation in the United Kingdom carrying out research into building and construction and the prevention and control of fire. With over 60 years of experience, it has built a worldwide reputation for independent, authoritative research and advice. Staffed by experts who are leaders in their fields, BRE has many specialised technical facilities, some indeed unique, and provides services to Government, the construction industry and its clients, and product suppliers. With its international links and its breadth of experience, BRE is where construction professionals themselves turn for advice.

The 6 T load was reapplied and the loaded plank continued to pull out of the floor at this constant load. The test was terminated when the maximum lateral deflection reached was 18.4mm and the final deformed profile under the 6 T load is given in Fig. 6 position 3.

Summary of Findings from Series 2 Tests

End A - (anchors in place)

1. The deformed profile of the ends of the planks at End A (anchors in place) showed that throughout the loading cycle, movement was fairly evenly transferred from the loaded plank to the adjacent planks. This behaviour was maintained up to a load level of 16 T the limiting capacity being due in the first test to the friction between the roof and its supports and in the second test the capacity of the test rig.

2. At the maximum load of 16 T approximately half of the deformation present was recoverable and deformation under the repeated 16 T load was similar to those recorded during the first load cycle.

End B - (anchors cut)

1. The deformation at End B corresponding to the maximum deformation recorded at End A occurred under approximately one third of the load applied to End A.

2. From the beginning of the load cycle the loaded plank moved out from the adjacent planks and the limited movement of the other planks was heavily skewed to one side of the roof.

3. The movement of the loaded plank due to the 6 T load was non-elastic and continued to 18.4mm when the test was terminated.

4. The limitation on the capacity to resist and transmit inplane loads was the failure of the loaded plank to act in conjunction with the adjacent planks.

Series 3 - Collapse Modes

Although the anchor installation was not intended to provide any flexural capacity in the transverse direction to the planks and had only been designed to transmit in plane shear forces to the side walls it was thought it would be interesting to see if the mode of collapse of the structure was influenced by the anchor installation.

To do this some controlled demolition was undertaken and recorded using time lapse photography at five frames a second and video recorders.

The same procedures were used on both Ends A and B and were as follows:-

1. - The end ground floor concrete wall panels which support the ends of the first floor planks, the first floor end wall panels and the ends of the roof planks were cut away to leave a minimal support at the centre and the edges of the end wall (Photo 1).

2. The remaining central support section was then carefully removed using an excavator bucket.

3. The remaining side supports were then progressively taken away leaving the end of the first floor unsupported.

4 The ensuing collapse was filmed and recorded

Findings

The only way to fully appreciate the mechanisms of collapse which were involved is to study the video recordings and the time lapse photographs. For the purpose of this report a number of the time lapse photographs are referred to which illustrate critical points in the collapse mechanisms.

End A - (anchors in place)

Photograph A4 -	Shows the stage were virtually all the load bearing support has been removed from the end of the structure. At this point there is no significant disruption to the structure and the first floor planks have just started to deflect under the load.
Photograph A5 -	The last remnants of the supports are removed and the first floor starts to fail slowly under the weight of the panels above.
Photograph A8 -	The first floor continues to fail as a unit leaving behind the roof which is unsupported but still intact. The side walls start to buckle out at first floor level under rotational forces induced by the first floor wall panels.
Photograph A10 -	The first floor collapse is complete, the side walls are now buckling badly but the unsupported roof remains in place and intact.
Photograph A12 -	There is complete loss of stability of the side walls due to rotation at first floor level forcing down the roof structure as a unit.
Photograph A14 -	Collapse of the complete end of the structure. Note the upper storey side walls being pulled inwards and roof now in free fall but still continuous.
Photograph A35 -	Final condition of collapsed structure. Note most of the debris lies within the original dimensions of the building.

End B - (anchors cut)

Photograph B5 -	This shows a stage where the supporting wall is being removed. It can be seen that the first floor has slipped down at the side nearest the camera and the end first floor wall panels and the side walls have started to rotate.
Photograph B6 -	The first floor is failing rapidly as the excavator bucket attempts to remove the remaining supports and the side walls have commenced buckling at first floor level.
Photograph B7 -	As the failure progresses rapidly the roof breaks up and fails following the first floor down. Note the first floor centre wall panel is sandwiched between the roof and floor and the end is falling as a whole.
Photograph B8 -	The roof continues to follow the first floor in free fall.
Photographs B9/11 -	Complete collapse of the end of the building.

Photograph B36 - Fragmented side walls had been contained by the safety scaffolding.

Summary of Findings

Apart from the rate of collapse being far greater at End B, there are a number of key differences in the sequence of events between the failure of Ends A and B these are:-

1. The roof structure at End A remained intact spanning laterally the width of the building throughout the collapse until the supporting side walls became unstable due mainly to impact from debris loading. Even at this point the roof collapsed as a continuous unit.

2. The roof structure at End B fragmented at an early stage of the test and followed immediately the failure of the first floor. Both roof and floor came down in free fall together.

3. The side walls at End A only buckled significantly when the first floor failure was complete and debris loading from the wall exerted its maximum lateral thrust. (Photo A12).

   At End B a similar degree of buckling to the side walls was present by the time the first floor had moved down approximately 300mm (Photo B6).

4. The lateral spread of debris was greater at End B than at End A and the construction immediately behind the collapsed portion of End B was vertically cracked indicating that lateral load had been transmitted back through the structure.

Discussion

In a progressive collapse energy is continually fed into the event and failure is transmitted through the structure via the connections. An important option in preventing local collapse progressing disproportionately is to ensure that locally damaged areas can be bridged over by the remaining structure and the resulting loads can be transmitted safely through alternative load paths.

Although the anchor system was not designed to improve post failure behaviour nor had their locations been chosen with this in mind, the test demonstrated two significant improvements in the post failure performance of the end strengthened with the anchors.

1. The roof structure was able to bridge the width of the building when its support had been removed and therefore its weight did not contribute to the energy input to the failure at first floor level.

2. The lateral tying delayed the buckling failure of the side walls until the impact of the debris loading from first floor forced the lower panels outwards.

   At End B the buckling failure of the side walls occurred almost at the same time as the first floor failure started.

Conclusion

1. The anchors clearly have the potential to modify the mode of failure of a structure and therefore offer a valuable tool to structural engineers in designing against the collapse mechanisms in existing structures.

2. Correctly designed and installed this form of anchor can effectively induce a high degree of plate action in floors and roofs comprising of individual hollow precast concrete components, enabling them to span laterally.

3. The introduction of plate action can stiffen existing precast floors and roofs.

4. The ability of the strengthened floors and roofs to transmit vertical and inplane forces through plate action was maintained, even in the presence of extreme deformations.

5. Repeat loading did not indicate any degredation in the anchors performance.

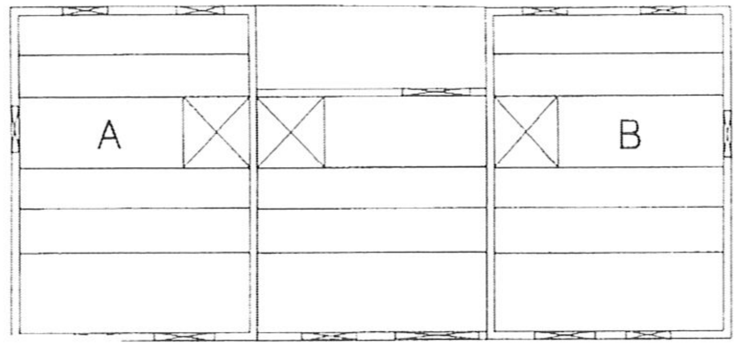
6. The ability of the roof system to transmit horizontal loads to the front and rear edge beams was increased at least $2^3/_4$ times by the addition of the anchors; the limiting factor during the test being the capacity of the test equipment.

7. Ultimately the total collapse of the end dwellings was initiated by lateral instability of the side walls. In the dwelling with the anchors cut this instability occurred almost immediately the first floor lost support, but only occurred when the first floor had completely collapsed in the dwelling where the anchors were intact.

8. The strengthening system does not introduce additional stresses into the structure and requires small deformations before it becomes active.

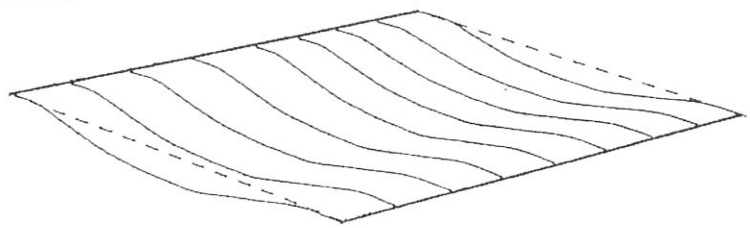
9. The structural model indicated by the dynamic testing was confirmed by the subsequent static load tests.

Fig. 1



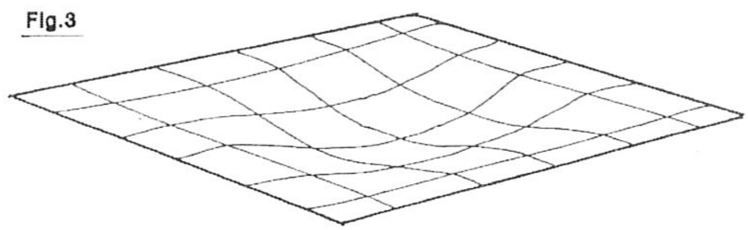
Plan at first floor level of terrace of 3 dwellings.

Fig.2



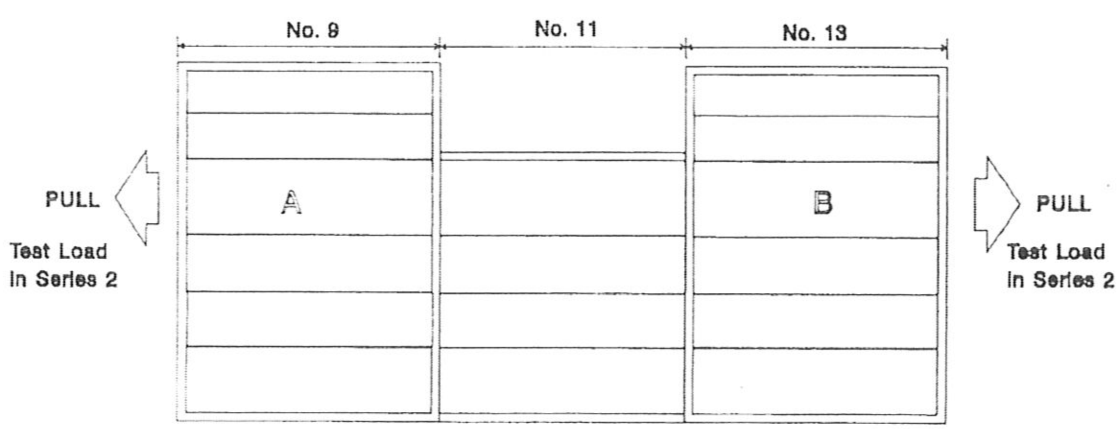
Floor spanning in one direction.

Fig.3



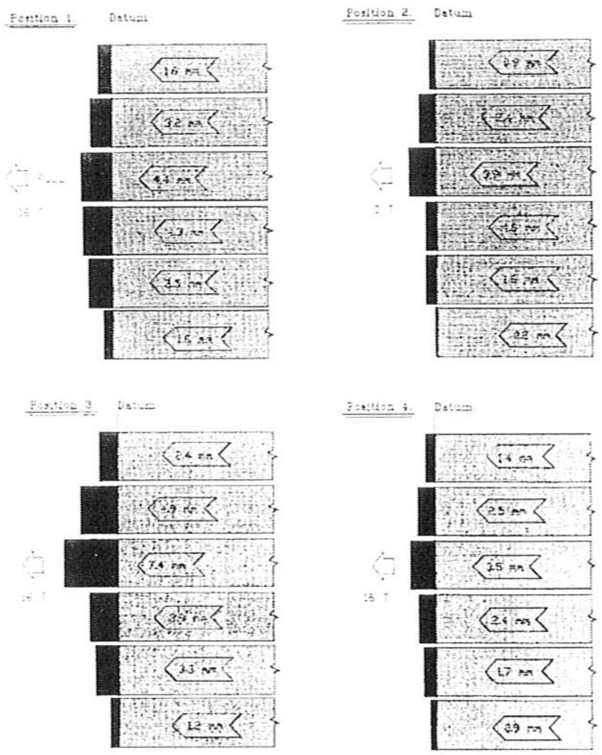
Floor spanning in two directions.

Fig. 4



Plan at roof level of terrace of 3 dwellings

Fig. 5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Position 4
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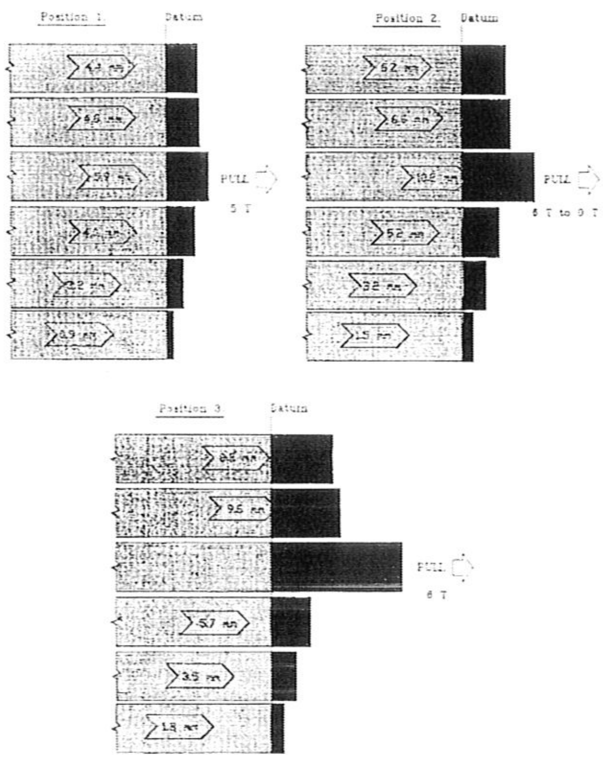
Fig. 6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