A Soil-Structure Interaction Analysis of a Damaged Building

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SYNOPSIS: A twenty years old reinforced concrete building suffered from fracture of one of its ground floor corner columns. The extent of damage resulting from fracture of the column is examined. Plane frame structural analysis is used in the interpretation of the signs of damage of the building. Soil compressibility is incorporated in the analysis by assuming the footings to rest on Winkler type subgrade. The theoretical analysis is assessed by comparing the results with the actual behaviour of the structure.

INTRODUCTION

Engineers are often confronted with the difficult task of assessing the damage suffered by a structure and suggesting the remedial measures to restore its safe serviceability. In some cases, the repair work may be difficult and costly that demolishing and rebuilding the structure may provide the best and sometimes the only solution. Before taking a decision, the engineer has to assess the building condition on the basis of his observations of the signs of damage. In doing so, he usually relies on previous experience and engineering judgement.

Numerical methods of structural analysis can be of great value to the engineer in his assessment of the building damage as a complete solution for axial and shear forces, moments as well as deformations can be obtained from such methods.

This paper presents a case study of a damaged building. Plane frame analyses are performed using the stiffness method of structural analysis with provision being made for soil compressibility according to the Winkler concept. Predicted results are compared with the pattern of cracks and deformations of the building in an attempt to assess the theoretical analysis.

DAMAGED BUILDING

Building Description

The damaged structure is a twenty years old four-story reinforced concrete building of the skeleton type. It is located in Helwan, one of the southern suburbs of Cairo and covers an area of about 420 square meters. A plan showing the columns and beams locations together with the areas covered by the different floors and roof is given in Fig. 1. The columns rest on isolated footings and are connected above the footings by a system of ground beams which support the ground floor masonry walls.

Damage Observations

The building suffered from fracture of the ground floor corner column marked D in Fig. 1 after about 20 years in service. After the incident, the first floor was extensively supported by timber props to stop further deterioration of the building resulting from the column fracture.

Inspection of the fractured column indicated that it was weakened due to drilling of several relatively deep and large holes through the concrete for the purpose of fixing a heavy steel door. The whole building was affected by the column fracture as eivdenced by the wall cracks appearing evverywhere in the building. Fig. 2 shows the pattern of cracks in the exterior walls at the faces AD and CD of the building. The inclination of the cracks indicates that the walls were subjected to shear deformations. As with the exterior walls, most cracks in the interior walls were also inclined. The crack width exceeded 10 mm in various parts of the building.

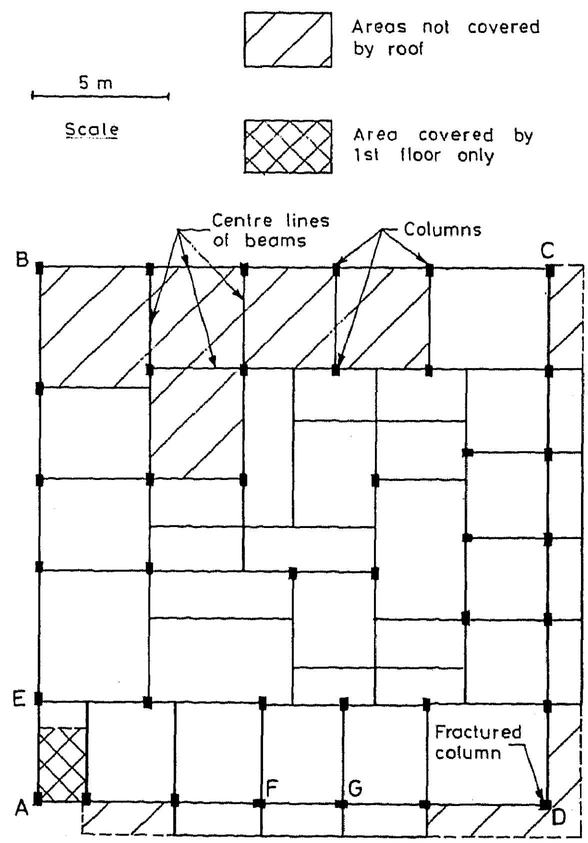


Fig.1 Locations of Columns and Beams

Visible cracks in the reinforced concrete elements were limited. Serious cracks appeared only in the column marked B in Fig. 1 which is located at the farthest point from the fractured column. The cracks could be seen in all the floors just below the intersection of the column with the beams.

Inspection of facade AD (Fig. 1 and 2a) indicated that the building was leaning towards the fractured column. The horizontal movement of the point marked E in Fig. 1 and 2a at the roof was nearly 100 mm measured with reference to the adjacent building. The movement decreased downwards to reach a zero value at ground floor.

Settlement of the ground inside the building area was another sign of damage. The settlement was particulary irregular near side AD, Fig. 1. In the zone between columns F and G, the ground settlement was greater than in the surrounding area by about 150 mm. This figure may be indicative of the order of magnitude of the settlement and differential settlement of the foundations.

METHOD OF ANALYSIS

The stiffness method for linear analysis of plane frameworks was adopted in studying the damaged building. Soil compressibility is simulated in the analysis by assuming the footings to rest on a Winkler type subgrade. If the footings are considered to be rigid, the settlement and pressure according to the Winkler concept will be uniform under each footing and the settlement will be equal to the pressure divided by the modulus of subgrade reaction. Under these conditions, each footing may be substituted by a vertical spring as illustrated in Fig. 3. The spring stiffness will be equal to the modulus of subgrade reaction k multiplied by the footing area.

In this study, the formulation of the stiffness matrices of the frame elements is based on the concrete sections of the elements using the concrete dimensions and ignoring the reinforcement. This method is commonly used to obtain the forces and moments in the structural elements. Its use is allowed, for example, by the British Code CP110. A more elaborate method is, however, recommended by the code for deflection calculations of beams. It consists of using the properties associated with partially cracked sections as described by Kong and Evans (1980). This was not possible, however due to the lack of reinforcement data for the building. The only structural drawings available were actually bad copies of the original drawings. Reinforcement data were given in tables which were mostly illegible. However, since the main concern of the theoretical analysis was the pattern of deformations and the effect of soil compressibility on this pattern, the simpler method ignoring the reinforcement was considered satisfactory.

RESULTS AND ANALYSIS

The plane frame analysis was applied to the two exterior frames AD and CD intersecting at right angle at the fractured column, Fig. 1. The idealized representation of the two frames in the analysis are given in Fig. 3. The effective flange widths of T-beams and L-beams and the loads transmitted from the slabs to the beams were determined according to the Egyptian Code of Practice. The moment and shear just to the right of the nodal points marked a, b, c, d, e, f, g and h in Fig. 3a and b after column fracture were calculated in an approximate manner by treating the slab with the two beams originally supported by the column (Fig. 1) as a plate supported on two sides. The tables compiled by El-Behairy (1974) for plates under different support conditions were used in the calculations.

The frame analyses were conducted for a wide range of values of the modulus of subgrade reaction in order to investigate the effect of soil compressibility. Comparison with

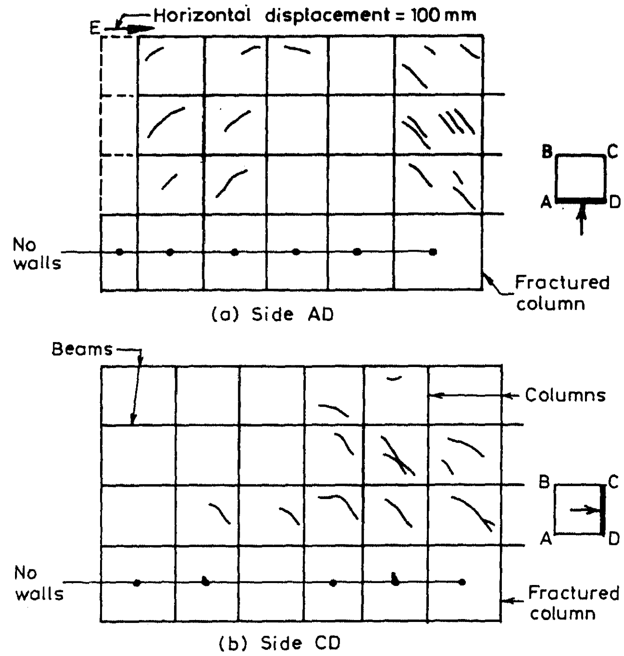


Fig.2 Cracks in Exterior Walls

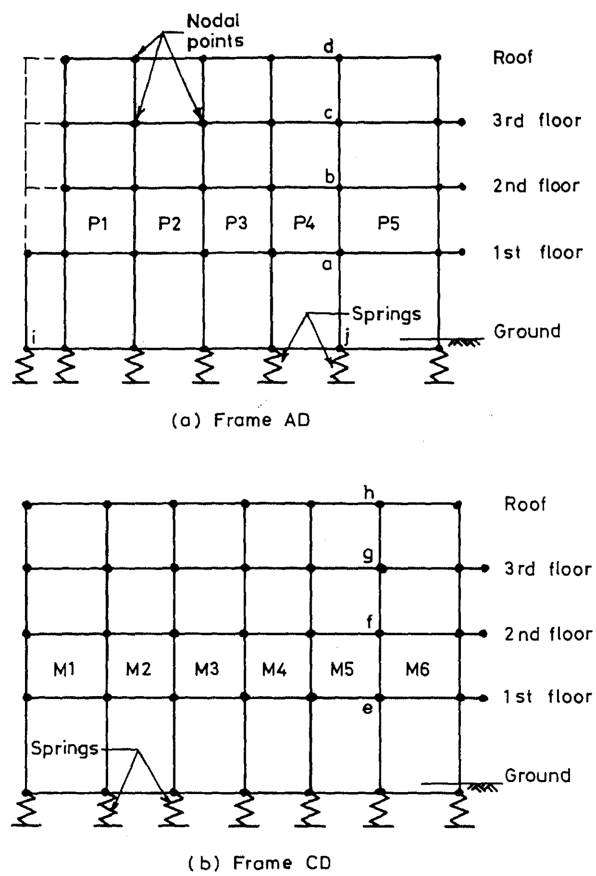


Fig. 3 Idealisation of Frames AD and CD in Frame Analysis

the observed building deformations and crack pattern are given below. All the given results were obtained from analyses with no restriction imposed on footing rotation. The effect of restricting rotation on these particular results was found to be insignificant.

Horizontal Movement

The predicted relationship between the horizontal movement at the top of frame AD and the modulus of subgrade reaction k after column fracture is given in Fig. 4. It can be seen that the horizontal movement decreases greatly with the increase of the modulus of subgrade reaction k. The limiting case corresponding to unyielding supports gives a value of 6.5 mm only which is considerably smaller than the measured value (100 mm). Referring to Fig. 4, the modulus of subgrade reaction corresponding to this latter value is 2.2 MN/m^3. The corresponding predicted settlements at nodes i and j, Fig. 3a are 2.7 and 13.2 mm respectively. These values are of the same order of magnitude of the observed ground settlement inside the building area.

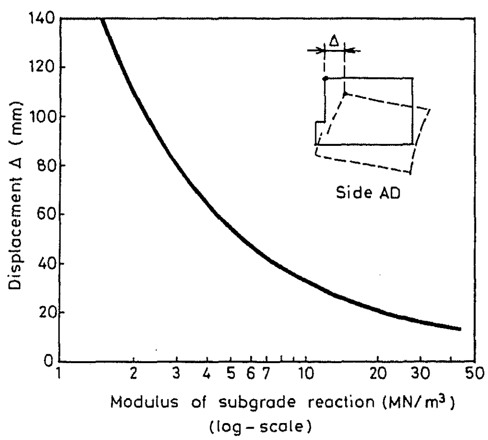


Fig.4 Relationship Between Modulus of Subgrade Reaction and Horizontal Displacement of Frame AD at Roof Level

The predicted horizontal movement at the roof level in the plane of frame CD is less than that in the plane of frame AD at all values of k. For example at k equal to 2.2 MN/m^3, the former value is 51 mm while the latter is 100 mm. Unfortunately, verification of the ratio between the horizontal movement of frames AD and CD was not possible since no measurement was made of the movement in plane CD. Surveying instruments would have been required for taking the measurement.

Crack Pattern

The development of cracks in buildings is usually related to some measure of differential settlement. Typical examples of this approach are the settlement criteria of Skempton and MacDonald (1956) and Polshin and Tokar (1957). Tilt (or rigid body rotation) is usually assumed not to contribute to the distortion of the structure and hence it is eliminated before differential settlements are determined. The tilt of a framed structure is defined as the angle between the initial and final positions of the line joining the two bottom end points of the frame. Burland et al (1977) stated that this might be acceptable for raft foundation, but quite inappropriate for a frame building on isolated footings. It may be added that further complications will arise if the structure suffers from horizontal sway in addition to tilt as is the case of the building in the present study. The development of cracks will be also related to the sway in this case.

In order to overcome the difficulties involved in relating cracks solely to foundation settlements, the method shown in Fig. 5 is used. Each wall panel is considered as an element undergoing shear strain. The potential direction of cracks in the wall will depend on whether the angle θ, Fig. 5a, increases or decreases with reference to its initial value (90°). The increase or decrease of θ is assumed equal to the shear strain experienced by the wall element. Various modes of deformations and tilt are shown in Fig. 5 together with the corresponding potential directions of cracks and the sign convention adopted.

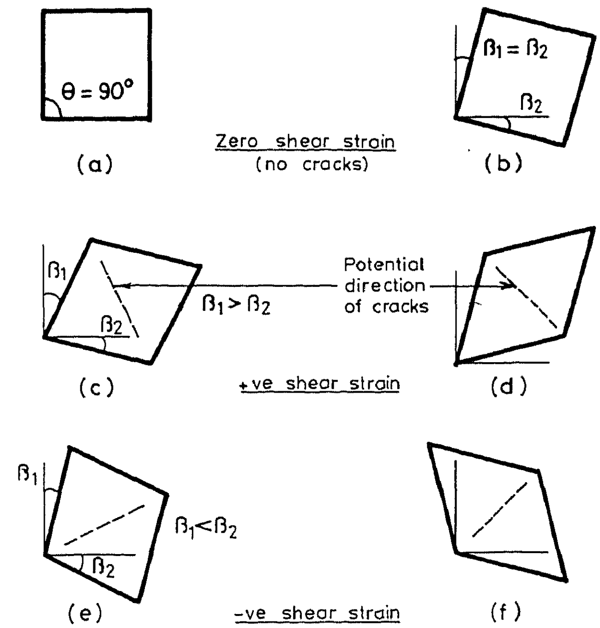
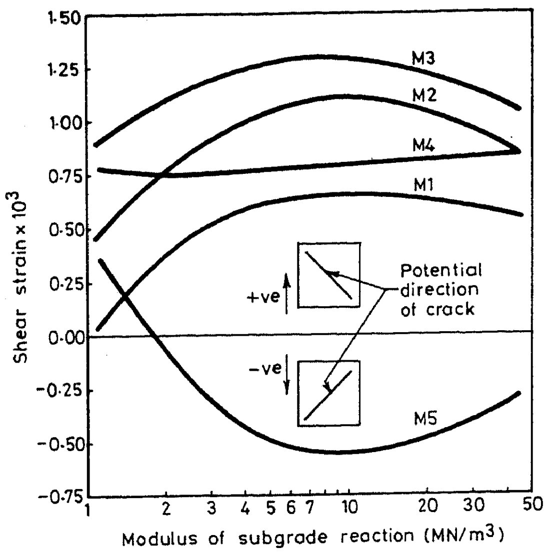


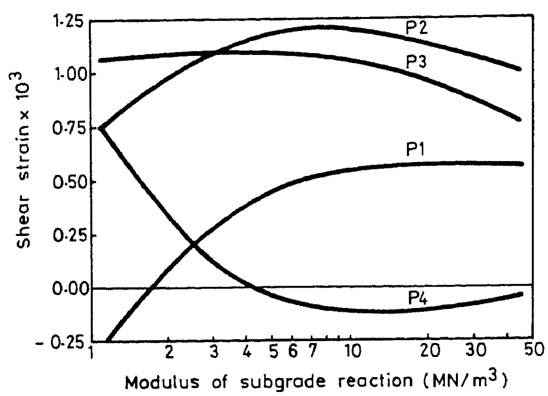
Fig. 5 Wall Distortion and Potential Direction of Cracks

The predicted relationships between the shear strain and the modulus of subgrade reaction k for the first floor wall panels of frame CD (marked M1, M2, M3, M4 and M5 in Fig. 3b) and those of frame AD (marked P1, P2, P3 and P4 in Fig. 3a) are given in Fig. 6a and Fig. 6b respectively. The predicted horizontal and vertical displacements of the nodal points at the corners of each wall panel are used in determining the shear strains. As already mentioned, the beams to the right of nodes a and e in Fig. 3 were treated in a simplified manner in the frame analysis. The shear strains for the wall panels above these beams could not, therefore, be determined. The relationships between k and the shear strains for the wall panels above the second and third floors are found to follow the same trend as those for the first floor wall panels.

Referring to Fig. 6a, it can be seen that the shear strain is greatly dependent on soil compressibility. The shear strain sign and hence the potential crack direction may differ according to the value of k. Comparison with Fig. 2b indicates that the potential crack directions become in accordance with the actual crack pattern as soil compressibility tends to high values. Agreement between predictions and observations of horizontal movement is also obtained at high soil compressibility as previously noticed. As the modulus of subgrade reaction decreases, the shear strain of panel M5, for example increases until it changes its sign, thus leading



(a) Frame CD



(b) Frame AD

Fig. 6 Relationships Between Modulus of Subgrade Reaction and Shear Strains in Walls

to the same direction of crack as that observed. Also the shear strain of panel M1 decreases sharply which conforms with the absence of cracks in this panel.

The predicted results shown in Fig. 6b for the wall panels in frame AD follow in general the same trend as those of frame CD. The observed crack pattern shows, however, some difference. Cracks are absent in panel P3 and P4 and the inclination of the crack in Panel P2 (Fig. 2a) contradicts that predicted. This is probably due to the high differential settlement between the columns in this zone. As previously mentioned, ground settlement in the region between columns F and G, Fig. 1 is greater than that in the surrounding area. Higher settlement of the columns in this zone with respect to the neighbouring columns could cause shear strains of signs opposite to those predicted in Fig. 6b and hence could be responsible for any local deviation from the overall pattern of deformations of the frame. It may be noted that the predicted direction of crack in wall panel P1 is in agreement with that observed, again, at low values of the modulus of subgrade reaction.

No predictions are made for the crack directions in panels M6 and P5 in Fig. 2a. It is interesting to note, however, that the direction of the cracks in these panels would have been at right angle to those observed if related only to the deflection of the supporting beams which are acting more as cantilever beams after column fracture. This shows that the walls are probably subjected to the deformation mode shown in Fig. 5c as a result of the horizontal sway suffered by the building.

Finally, it may be concluded that the predicted and observed pattern of deformations are generally in good agreement at low values of the modulus of subgrade reaction k. These low values are compatible with the order of magnitude of the ground settlement experienced by the building. Differential settlement due to factors such as soil nonhomogeneity which are not included in the theoretical analysis can cause local deviations from the overall behaiour. These local effects should be taken into consideration in assessing predicted forces and moments in the frame elements.

SUMMARY AND CONCLUSIONS

A case study of a damaged building is presented. Damage is caused by fracture of one of the ground floor corner columns. The whole building is affected by the column fracture as evidenced by the cracks, deformations and ground settlement suffered by the building.

Soil-structure interaction analyses are performed using the stiffness method of structural analysis for plane frameworks. Soil compressibility is simulated in the analysis according to the Winkler concept. The study indicates that soil compressibility has an important effect on the pattern of cracks and deformations. Using the appropriate modulus of subgrade reaction in the analysis, reasonable agreement is obtained between predictions and observations. Differential settlements due to factors not included in the analysis such as soil nonhomogeneity could cause local deviations from the general behaviour of the structure. These local effects should be taken into account in analysing the predicted results.

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