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# Soil-Structure Interaction and Aseismic Design of a Stadium Building

V. D. Miglani
*Regional Engineering College, Kurukshetra, India*

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# Soil-Structure Interaction and Aseismic Design of a Stadium Building

**V. D. Miglani**
**Professor of Civil Engineering, Regional Engineering College, Kurukshetra, India**

SYNOPSIS: The paper presents case history of a reinforced concrete stadium building which had been structurally designed for a particular component configuration and also constructed upto seating level and which was referred to the author for suggesting structural modifications and redesigning for different configuration which meant curtailing middle two main columns each above the seating level out of four columns in each of left and right halves of the building. The required modifications necessitated analysis of the modified frame under static loads taking into account soil-structure interaction. The other problem to be tackled was ensuring lateral stability with reduced number of main columns, which are slender and have restriction in size, under earthquake conditions. Since the structure could have free vibrations in coupled translation and yawing, advantage has been taken of stiffness of rear columns whose size was not restricted.

## INTRODUCTION

It is not uncommon for civil engineers to face problem of redesigning a structure requiring modifications after a large part of construction has already taken place. The solutions are sometimes simple and at some occasions not so simple as can be adopted without some detailed analysis.

One such problem was that of a reinforced concrete frame building 'DRONA STADIUM' at Kurukshetra, which, after having been constructed upto seating floor level and with reinforcement of columns appearing above the floor, was referred to the author for suggesting modifications in the structure and for redesigning the same without dismentling the constructed footings. It was pointed out by the district sports authority that the upcoming eight main columns which were to support the roof and its large cantilever would be too many to afford an unobstructed view to the spectators behind them and wanted the middle two columns out of four in each half to be curtailed at seating floor level. The solution meant transfer of load of eight roof beams to the four retained columns and also distribution of the enhanced load on each of these columns back to the eight columns already constructed upto seating floor and their foundations. It also meant ensuring aseismic safety of the structure.

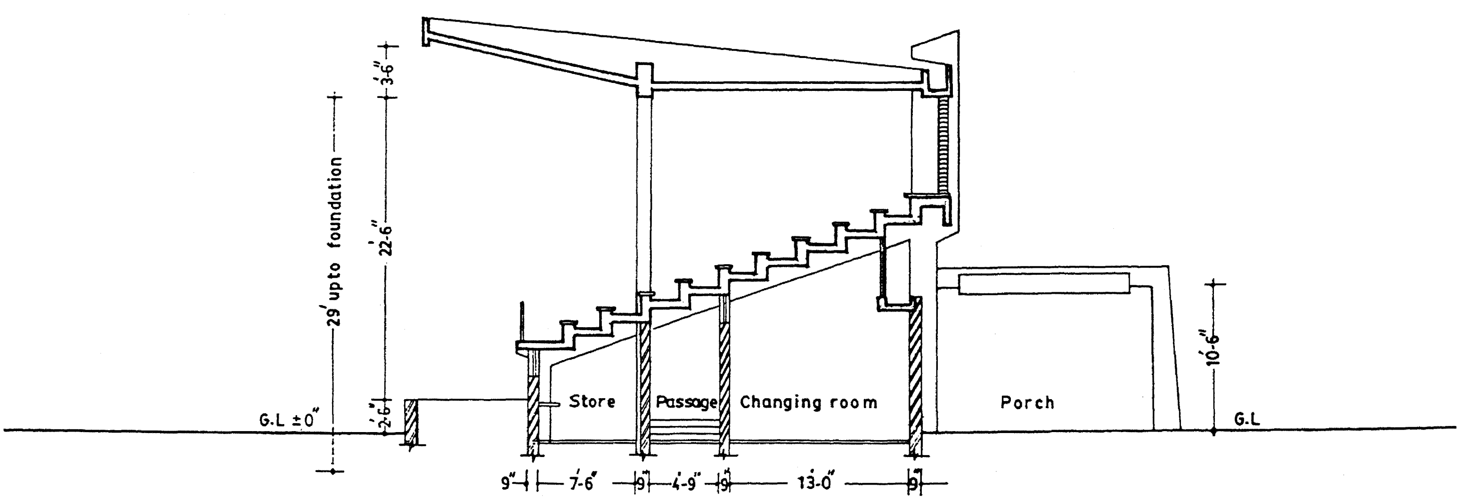


Figure 1 - Sectional Side Elevation of the Building.

DETAILS OF THE BUILDING

Figures 1, 2 & 3 show the side elevation and plans of the building as per the original design. The roof having size 38.5 feet x 93 feet (11740 mm x 28350 mm) overall consists of slab monolith with eight 38.5 feet (3960 mm) centre to centre upturn beams. These beams are named RB-1 to RB-8 from left to right. Each beam has 20.6 feet (6280 mm) effective supported span and 16.40 feet (5000 mm) cantilever. The roof beams as per these plans were to be supported on two rows of eight columns each; main columns MC-1 to MC-8 at the junctions of cantilever and supported span, rear columns RC-1 to RC-8 at the other ends of supported spans. About 90 per cent of the roof load is carried by the main columns. The seating floor consists of stepped slab supported on beams FB-1 to FB-8 which in turn are supported on the earlier

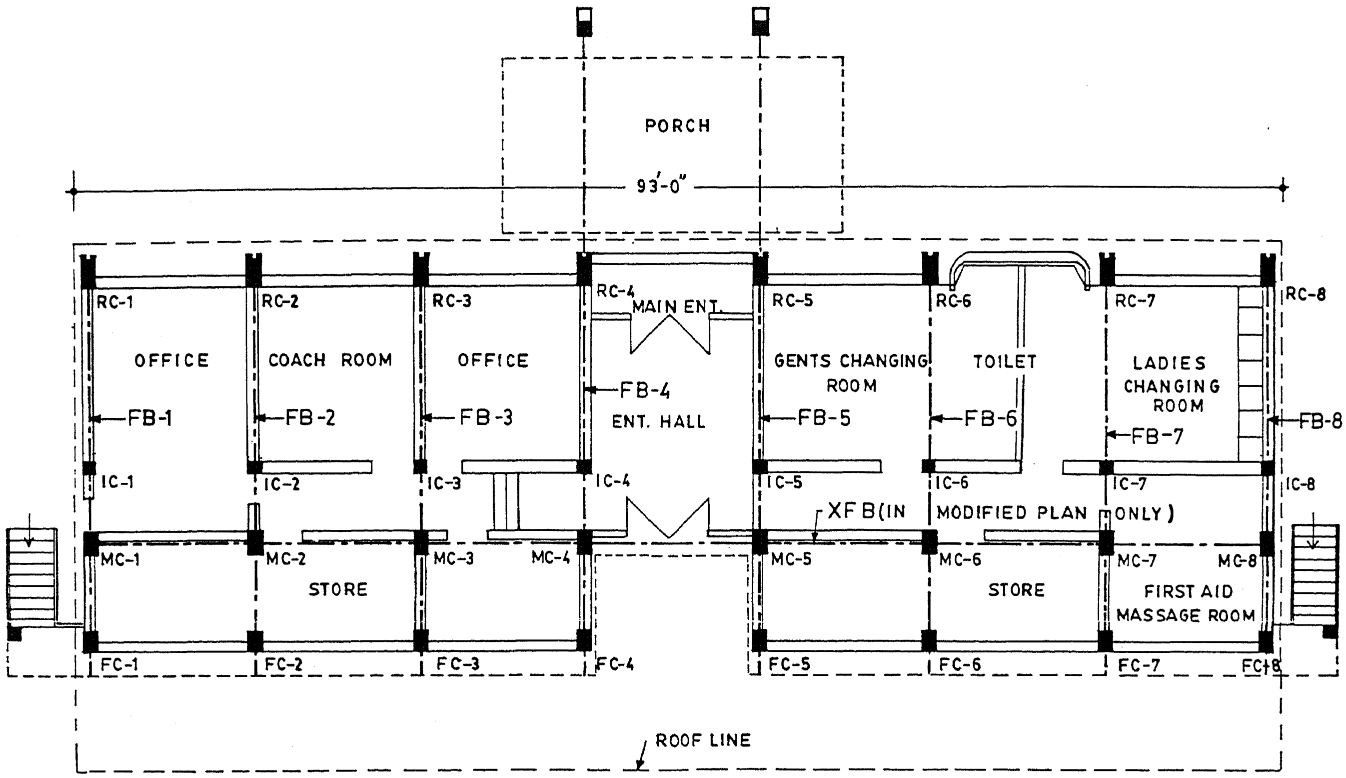


Figure 2 - Ground Floor Plan of the Building.

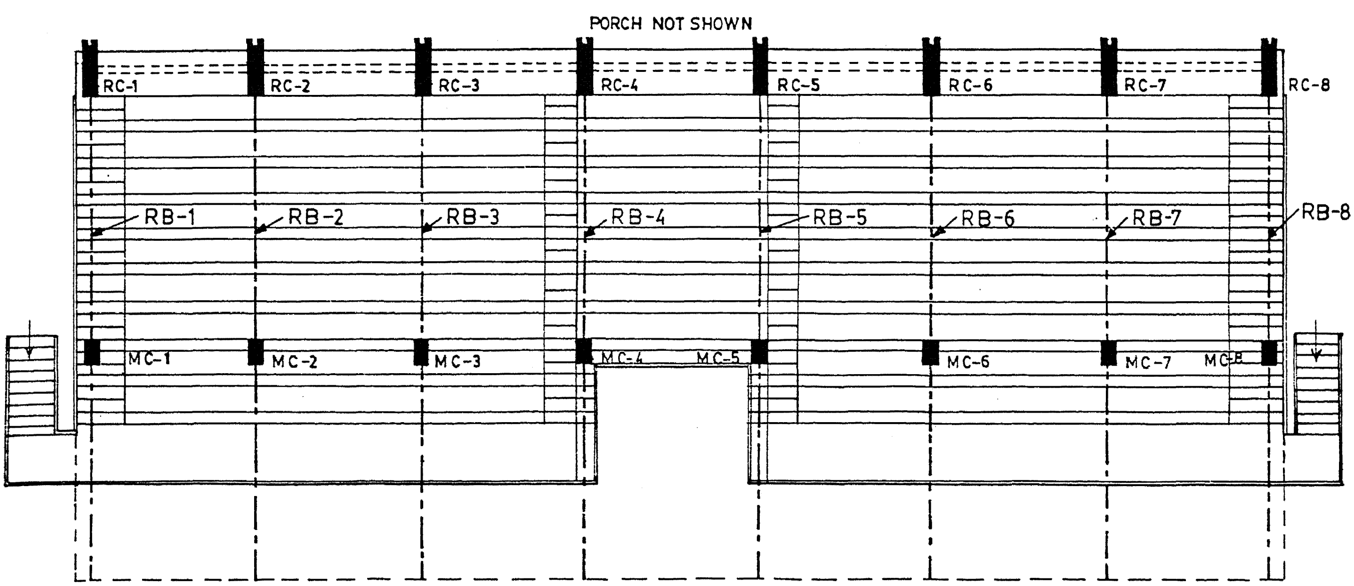


Figure 3 - Seating Floor Plan of the Building (Original).

mentioned columns (MC-1 to MC-8, RC-1 to RC-8) and also on other two rows of columns, one in the front consisting of columns FC-1 to FC-8 and the other row IC-1 to IC-8 between rows of MC and RC columns. The two rows of FC & IC columns do not extend above the seating floor. The space below the seating floor has been uilized for various purposes.

## THE PROBLEM

The construction of the building started with laying of foundations of all the thirty four columns including those of main columns as per the original design. This was followed by casting of the columns and arrangements for laying of the seating floor. The reinforcement of 'FC' and 'IC' columns was either curtailed or bent into the floor as per the structural design as these columns were required only in the lower storey. At this stage only, when the reinforcement of the eight main columns (MC-1 to MC-8) as also the rear columns was protruding above the seating floor, did the sports and district authorities realise the drawback in the functional utility of the building. The problem was referred to the author for any possible solution without demolishing the already constructed work.

It was felt that the eight number of main columns would be too many for unobstructed view by the spectators behind them. It was desired to retain the end main columns and the middle two main columns (i.e., MC-1, MC-4, MC-5 and MC-8 from left to right) and curtail the remaining main columns (i.e., MC-2, MC-3, MC-6 and MC-7) above the seating floor. Modified seating floor plan in that case would be as shown in the Figure-4.

## THE SOLUTION

The solution meant the following:

(a) Modifying the structural system of roof and and redesigning the same for transfer of roof loads to the retained columns.

(b) Redesigning the main columns MC-1, MC-4, MC-5 and MC-8, as these would now be required to take up almost twice the earlier design loads.

(c) Suggesting structural system for transfer of the roof load coming through the retained columns on the already constructed foundations as at this stage it would not be economically feasible to redesign and reconstruct new foundations for the retained columns.

(d) For an economical solution, it would be necessary to ensure a better picture of modified structural frame taking into account soil-structure interaction.

(e) Ensuring a safe aseismic design of the modified structure.

Steps (a) to (d) are dependent on each other and cannot be taken in isolation.

## THE MODIFIED STRUCTURE

Although several alternatives, such as waffle type roof slab, various beam-slab arrangements were available, it was decided to retain the original roof beams (RB-1 to RB-8) and slab arrangement for architectural reasons. An upturn beam XRB of constant section from column

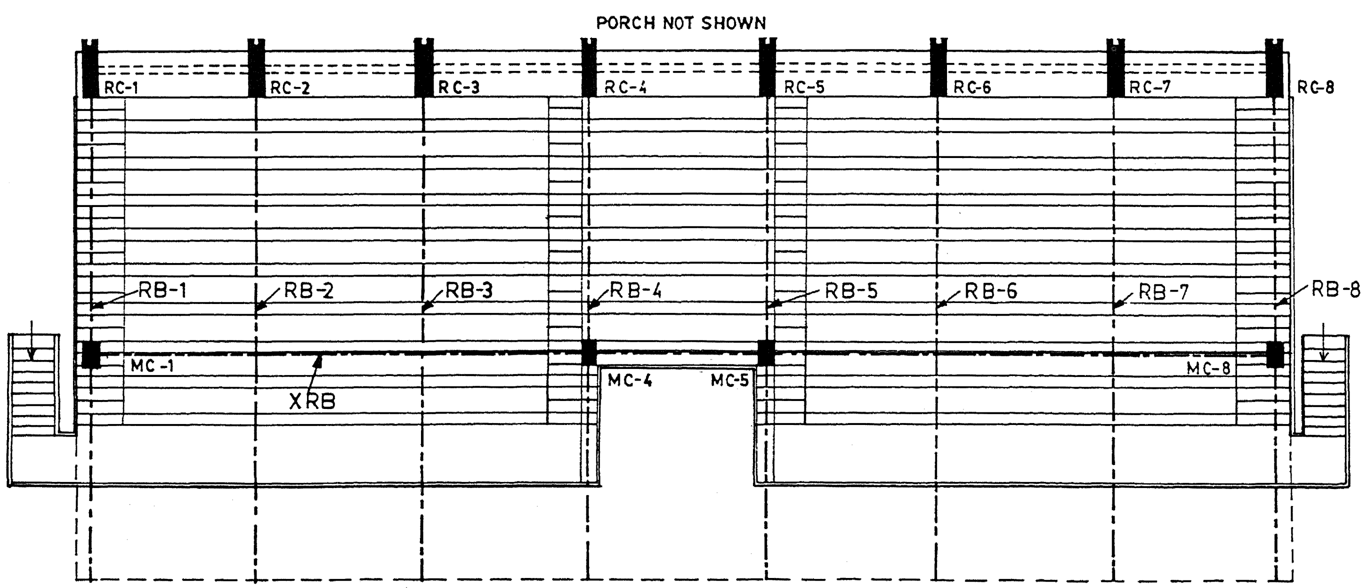


Figure 4 - Seating Floor Plan of the Building (Modified).

tops MC-1 to MC-8 was introduced. Similarly a down turn beam XFB with its top at seating floor level between MC-1 to MC-8 was also introduced. On account of passage under the seating floor, size of the beam XFB between columns MC-4 and MC-5 was restricted. This arrangement entailed only little demolishing of freshly laid concrete in the main columns down to the level of bottom of beam XFB.

## ANALYSIS AND DESIGN OF THE STRUCTURE UNDER STATIC LOADS

To start with, cross sections for various components of the structure are decided on the basis of experience. The structural frames subjected to loads are analysed for bending moments, shear forces and direct loads. Based on the analysis, the sections are revised and analysis repeated, if necessary. This is a normal procedure for design. In this particular case, sizes of the members already constructed cannot be revised.

Static loads were considered as per the Indian Standard Code of Practice (IS:875-1964) and limit state method as per IS:456-1978 was adopted. Detail of these aspects is not within the scope of this paper.

All the structure frames have been analysed by the usual process of moment distribution. Only one frame (hereafter called main frame) consisting of all main columns and horizontal) beams could not be solved without the consideration of soil-structure interaction.

## SOIL-STRUCTURE INTERACTION OF THE MAIN FRAME

Since the main frame and its loading are symmetrical, only half the frame has been used for analysis by modifying the half frame as shown in Figure-5. Carry over factors for the

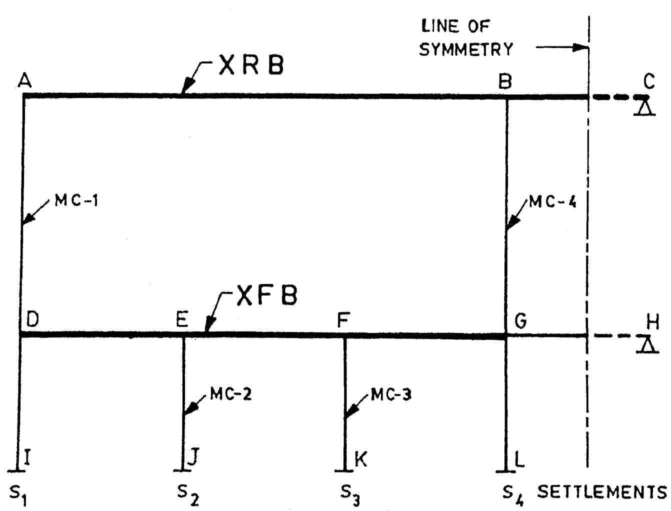


Figure 5 - Line Diagram of Half Main Frame

beams BC and GH are taken as zero, while their stiffness factors are taken at half their values in the total frame. For this purpose, reference may be made to any standard text book on structural analysis or on moment distribution, e.g., by Lightfoot (1961).

Concept of subgrade reaction (Terzaghi and Peck, 1969) has been used for considering soil-structure interaction. Vertical reaction '$R_n$' at any foundation 'n' is given by

$$R_n = K_s S_n B_n D_n \qquad ...(1)$$

where

$K_s$ = Coefficient of subgrade reaction

S = Settlement of the foundation

B = Width of the foundation in the plane of main frame

D = Width of the foundation perpendicular to the plane of foundation.

Using the subgrade reaction concept and taking into account the compliance of foundation to application of moments, stiffness factors and carry over factors at the other end of columns can be derived to the following form:

$$k_{ab} = \frac{12EI}{H} \times \frac{1 + \beta}{3 + \beta} \qquad ...(2)$$

$$C_{ab} = 0.5 \times \frac{1}{1 + \beta} \qquad ...(3)$$

where

$k_{ab}$ = Stiffness factor of column 'ab' at joint 'a', end 'b' being supported on foundation of size B X D.

$C_{ab}$ = Carry factor from 'a' to 'b'

$= \frac{\text{Induced moment at 'b'}}{\text{Applied moment at 'a'}}$

E = Young's modulus for concrete

I = Moment of inertia of the column section about axis passing through c.g. of foundation and normal to plane of frame.

$\beta$ = Compliance factor of the foundation

$= \frac{36\ EI}{K_s\ B^3\ D\ H}$

H = Length of the column between 'a' and 'b'.

In the absence of directly determined values of coefficient of subgrade reaction $K_s$, a range of values of $K_s$ was selected from table of values given by BOWLES (1968) and modified for length to width effect as proposed by Terzaghi (1955).

The governing equations for analysis are :

$$R_{1W} - F_{11}S_1 + R_{12}S_2 + R_{13}S_3 + R_{14}S_4 = K_s S_1 B_1 D_1$$

$$R_{2W} + R_{21}S_1 - F_{22}S_2 + R_{23}S_3 + R_{24}S_4 = K_s S_2 B_2 D_2$$

$$R_{3W} + R_{31}S_1 + R_{32}S_2 - F_{33}S_3 + R_{34}S_4 = K_s S_3 B_3 D_3$$

$$R_{4W} + R_{41}S_1 + R_{42}S_2 + R_{43}S_3 - F_{44}S_4 = K_s S_4 B_4 D_4 \qquad \ldots(4)$$

where

$R_{1W},\ldots,R_{4W}$ = Reactions (upward) at 1,..,4 due to given loading on the frame with supports not settling.

$F_{nn}$ = Force (downward) required at nth support to cause unit downward displacement at the nth support.

$R_{mn}$ = Reaction (upward) at mth support due to unit downward displacement at the nth support.

$S_1, S_2, S_3 \& S_4$ = Settlements at 1,2,3 & 4.

All the above forces have been determined by considering frames with supports unyielding in vertical direction but compliant in rotation and simultaneous equations (4) solved for $S_1,\ldots,S_4$. Bending moments, shear forces and direct forces in various parts of the main frame have been determined by the method of superposition.

Analysing the main frame as above using maximum and minimum values of $K_s$ in the selected range reveals that higher value of $K_s$ gives more critical results of column loads (hence critical in distribution of loads to the foundations) and also critical in bending moments in the beams XRB and XFB near the middle two columns. On the other hand, lower value of $K_s$ is critical for bending moments in these beams midway between main columns: MC-1 and MC-4, MC-5 and MC-8. Table-1 shows a comparison of design load carrying capacity and the load coming on the foundations through this arrangement using critical results of analysis.

TABLE-1:

Comparison of Design Load Capacity 'C' and Actual Redistributed Load 'R' on Existing Foundations

| | MC-1<br>MC-8 | MC-2<br>MC-7 | MC-3<br>MC-6 | MC-4<br>MC-5 |
|---|---|---|---|---|
| C ≐ (tonnes) | 28 | 36 | 34 | 34 |
| R (tonnes) | 31.02 | 32.06 | 34.10 | 37.15 |

ASEISMIC DESIGN OF THE BUILDING

Restriction in size - 15" x 15" (380 mm x 380 mm), length of main columns - 29' (8840 mm) and their reduction in number to carry about 90 per cent roof load posed the problem of aseismic design. The most critical mode of vibration of the structure is horizontal translation in the longitudinal direction (x-direction) coupled with yawing motion. Free vibration analysis has been done by solving eigen value problem in the following set of four equations in the matrix form:

$$\begin{bmatrix} E11 & E12 & E13 & E14 \\ E21 & E22 & E23 & E24 \\ E31 & E32 & E33 & E34 \\ E41 & E42 & E43 & E44 \end{bmatrix} \begin{Bmatrix} X_1 \\ \phi_1 \\ X_2 \\ \phi_2 \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix} \qquad \ldots(5)$$

Where the elements of the square matrix are:

$E11 = - M_1 \omega_n^2 + S01$
$E12 = - S02$
$E13 = - S01$
$E14 = S03$
$E21 = - S02 = E12$
$E22 = - M_{m1}\omega_n^2 + S05 + S07 + S10$
$E23 = S02$
$E24 = - (S04 + S08 + S10)$
$E31 = - S01 = E13$
$E32 = S02 = E23$
$E33 = - M_2\omega_n^2 + S01 + S11$
$E34 = - (S03 + S12)$
$E41 = S03 = E14$
$E42 = - (S04 + S08 + S10) = E24$
$E43 = - (S03 + S12) = E34$
$E44 = - M_{m2}\omega_n^2 + S06 + S09 + S10 + S13 + S14 + S15$

The other quantities are :

$X_1$ = Translation of the roof mass at its c.g. in x-direction.

$X_2$ = Translation of the floor mass at its c.g. in x-direction.

$\phi_1$ = Rotation of the roof about vertical axis through its c.g.

$\phi_2$ = Rotation of the floor about vertical axis.

$M_1$ = Mass of roof.

$M_2$ = Mass of floor.

$M_{m1}$ = Mass moment of inertia of roof about vertical axis through its c.g.

$M_{m2}$ = Mass moment of intertia of seating floor about vertical axis through its c.g.

$\omega_n$ = Circular natural frequency of the structural system.

$a_1, b_1$ = Coordinates of the column tops in x & y directions w.r.t. c.g. of the roof mass.

$c_1$ = No. of columns in the upper storey.

$a_2, b_2$ = Co-ordinates of column tops in lower storey in x & y directions w.r.t c.g. of the roof mass.

$c_2$ = No. of columns in the lower storey.

$$S01 = \sum_{c_1} k_{x1}$$

$$S02 = \sum_{c_1} k_{x1}.b_1$$

$$S03 = \sum_{c_1} k_{x1}.b_2$$

$$S04 = \sum_{c_1} k_{x1}.b_1.b_2$$

$$S05 = \sum_{c_1} k_{x1}.b_1^2$$

$$S06 = \sum_{c_1} k_{x1}.b_2^2$$

$$S07 = \sum_{c_1} k_{y1}.a_1^2$$

$$S08 = \sum_{c_1} k_{y1}.a_1 a_2$$

$$S09 = \sum_{c_1} k_{y1}.a_2^2$$

$$S10 = \sum_{c_1} k_{\emptyset 1}$$

$$S11 = \sum_{c_1} k_{x2}$$

$$S12 = \sum_{c_1} k_{x2}.b_2$$

$$S13 = \sum_{c_1} k_{x2}.b_2^2$$

$$S14 = \sum_{c_2} k_{y2}.a_2^2$$

$$S15 = \sum_{c_2} k_{\emptyset 2}$$

$k_{x1}, k_{y1}, K_{\emptyset 1}$ = Stiffnesses of columns in upper storey in x, y and $\emptyset$ direction.

$k_{x2}, k_{y2}, k_{\emptyset 2}$ = Stifgfnesses of columns in lower storey in x, y & $\emptyset$ directions.

After determining the modal quantities, the design seismic forces have been determined as per Indian Standard Code - IS:1893-1975. Only the fundamentasl mode of vibration has been considered as the contribution of the higher modes to the final forces is very small and can be neglected for all practical purposes. Advantage has been taken of stiffness of the rear columns, size of which only could be varied.

More details of soil-structure interaction analysis and aseismic design are given elsewhere (Miglani-1984).

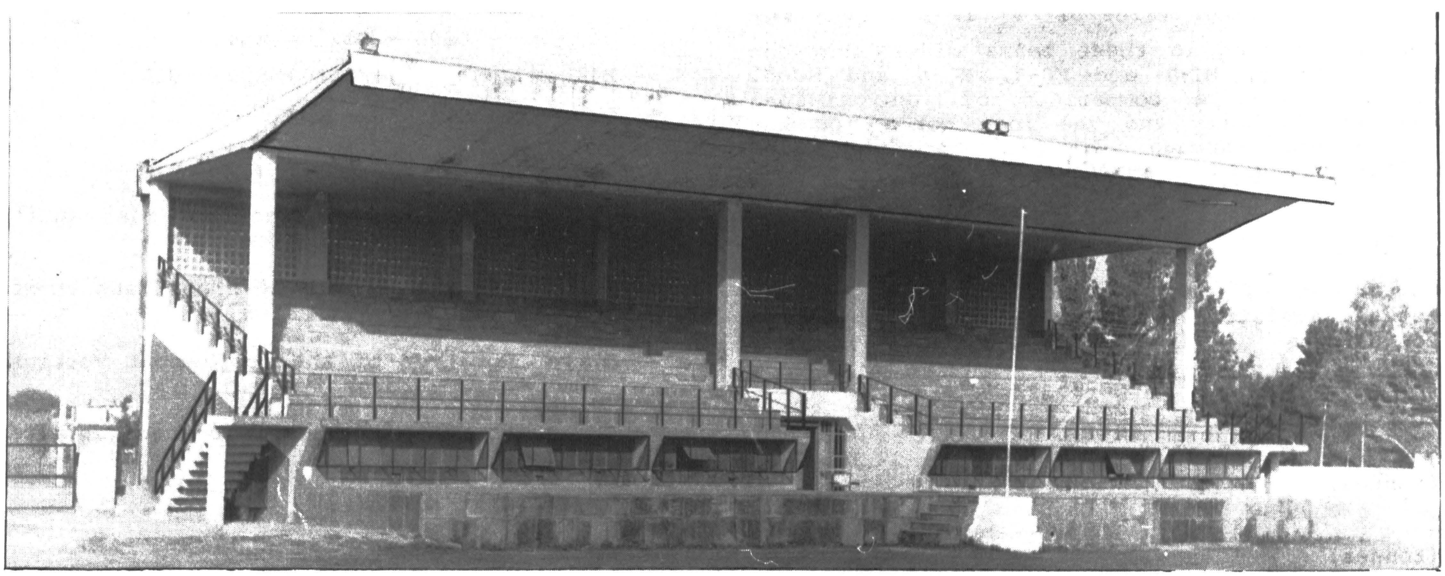

Figure 6 - A View of Finally Constructed Stadium Building.

A view of finally constructed stadium building appears in Figure 6.

## CONCLUSIONS

Moment distribution method has been used for solving the soil-structure interaction problem.

Constructed in 1984, the stadium building has stood the test of time so far and also withstood the October 20, 1991 earthquake of magnitude 6.2 on Richter scale and of intensity VI on the Modified Mercalli Scale in the area without even a hair crack whereas many buildings in the vicinity including even some newly constructed ones developed cracks or had cracked plaster. This gives an idea of adequacy of the aseismic design of the building.

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