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Editorial

Structural Alterations

Umesh Dhargalkar

Bridge Near Itarsi Over Railway Lines

Cover Story

A challenging job presented here was to provide temporary work for Road Over Bridge, crossing heavy traffic on railway lines near Itarsi (Madhya Pradesh) on the Central Railways.

One of the conditions was that no block in Railway operations would be available. Moreover, there was no place for casting yard on either side of the bridge to consider precasting option.

The bridge consisted of six prestressed concrete girders in one span of 40 m, each weighing 140 T. It was decided to make a staging up to pier cap level on one side beyond railway tracks for precasting one girder at a time, partially prestressed and slide sideways in its designed position. Girders were physically pulled along pier cap by means of a winch placed on ground with a provision of idle rollers for change of direction. Interesting design of centering, shuttering and movement of the precast girders over 45 degrees skew piers, was the main theme of this scheme (ref. dwg. and photographs). Side shuttering in the lengths of about 6 m in between girder diaphragms was designed to open out horizontally over hinge provided at the bottom. This shutter then served as a working platform for assembling of high tensile steel cables and reinforcement. The bottom hinge level was so provided that after opening the shutter horizontally, P.S.C.Girder ready for moving out had a clear unobstructed passage. Overhead monorail was provided for raising and lowering of heavy side shutters effortlessly as and when necessary. Walkway was also provided at the top level of the P.S.C.Girder to facilitate movement of wheel barrows for concreting. Above arrangement helped to reduce time frame substantially as under:

1. Working platform was available at the level of pier cap. This arrangement avoided lifting of 140 T girder for erection in place.
2. Girder was provided with steel plate shoe and pier cap was provided with steel plates. Interface was properly greased for easy side shifting. Hand Winch was placed on the ground and through idler, pulling effort was transferred at pier cap level.
3. 6 m long side shutters on one side could be made horizontal (hinged below girder level to platform) after 24 hours of girder casting. These shutters fabricated in one piece and hinged at bottom end were lifted or lowered by monorail above at ease. Only half load effort for lifting was required as half load was supported by hinge. If shutters were conventionally erected, they would have been in pieces of about 1200 mm wide x 3500 mm high. Handling and assembly would have been extremely difficult. Overhead monorail and hinge at bottom did all the trick. Manual effort was minimal. Accurate, fast and safe assembly at ease was the KEY ACHIEVEMENT. This platform when opened was a very good and safe working place for reinforcement and prestressing cables assembly.
4. Rail Traffic below did not hamper work progress at any point of time.

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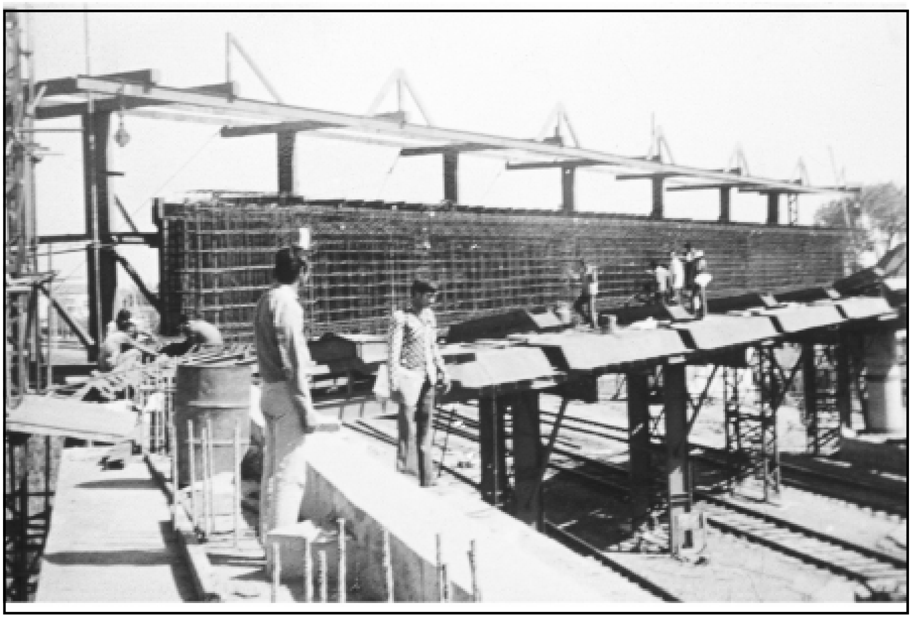
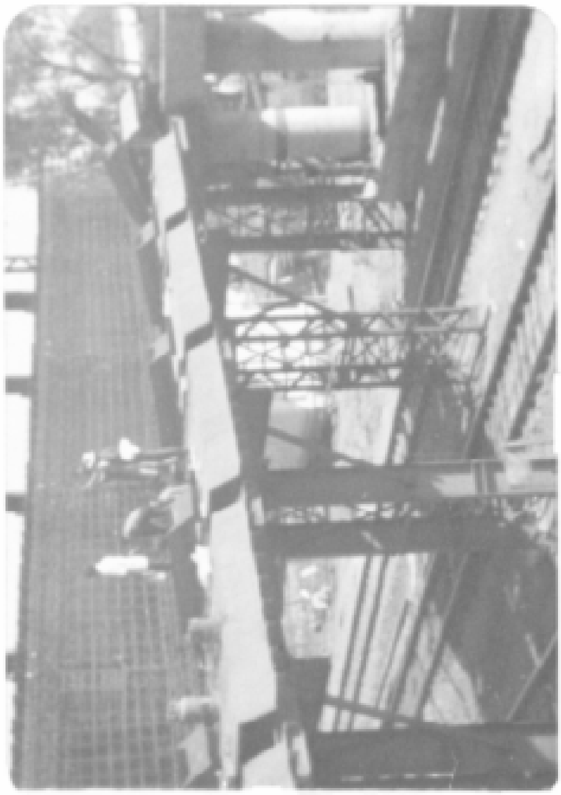


CASE 7



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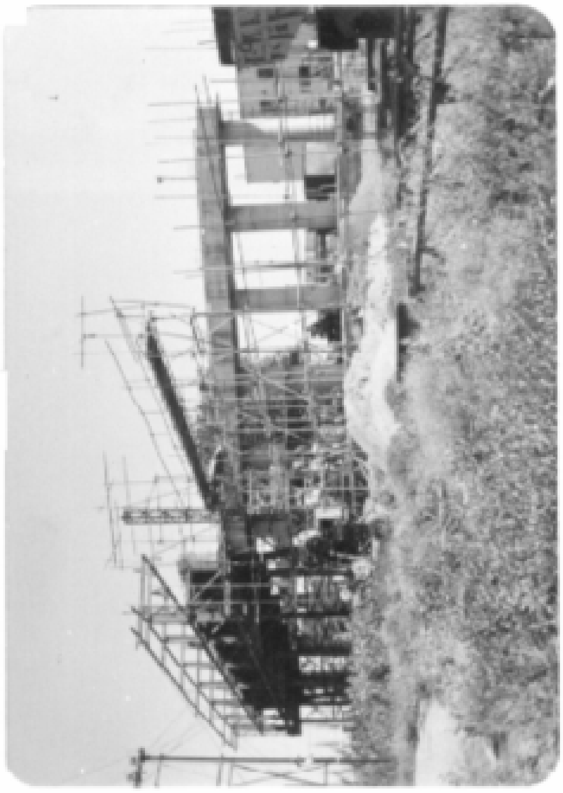

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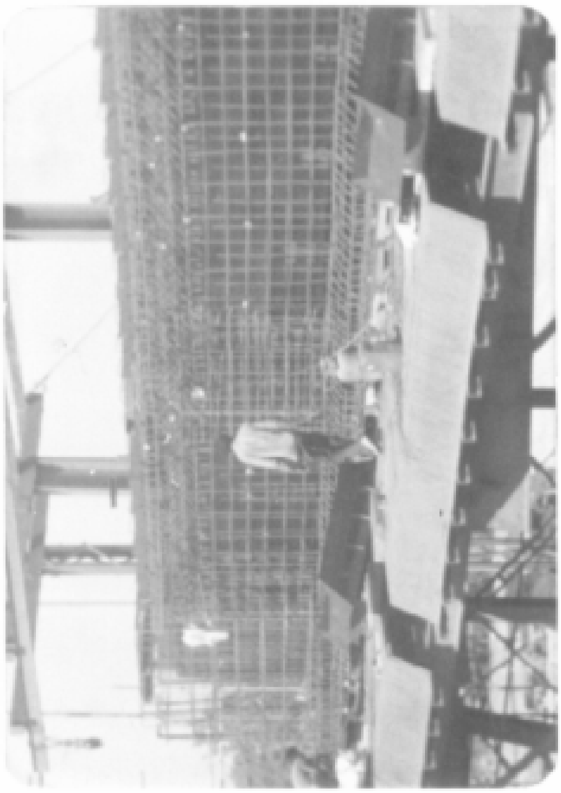

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General Principles of Earthquake Resistant Design

G V Khanolkar

The fundamental goals of any structural design are to ensure safety, serviceability and economy. Designing structures to resist earthquakes has become very important as earthquakes cause tremendous loss of life and property. At the same time, it should also be admitted that it is very difficult to design structures to resist earthquakes with complete assurance of safety of life and property. The main reason being difficulty in accurately quantifying the action of earthquake load on the structure and also in predicting the response of the structure to the action of earthquake.

Earthquakes have their origin in the sudden release of accumulated energy in some zones of the earth's crust and the resulting propagation of seismic waves. It is the sliding movement between the two continental plates that causes this release of energy at the corresponding plates' boundary. It results in sudden shaking of ground. Inertial forces, in the direction opposite to that of the ground movement come into action on the structure supported by this ground. As the ground movement takes place in all directions in a small interval of time, the structure above also experiences inertial forces of similar nature. The sudden vibrations in the structure may damage the non structural components in the structure and structural collapse can also occur if there is a loss of equilibrium between the action of inertial forces and resistance offered by the structure.

Magnitude of the ground acceleration caused due to the earthquake needs to be determined in order to be able to calculate the magnitude of the inertial force experienced by the structure. The magnitude of ground acceleration depends on other parameters like severity of the seismic source, epicenter distance, depth of soil above bedrock etc.

Magnitude of ground acceleration is also known to be influenced by the type of seismic action, hypocentral distance, wave propagation path, orientation of the site with respect to the fault line, local site conditions and local topography. Data collected from the earthquakes in the past, in the form of accelerograms (graph of actual acceleration values of the ground measured during an earthquake in relation to its time duration. It is recorded with a device called Acclerograph.) is used to correlate and predict values of the peak ground accelerations to be considered in the analysis & design.

It is the responsibility of the national authorities to provide the magnitudes of peak ground accelerations to be considered by engineers to design earth quake resistant structures. Different regions with different seismic activities, different probabilities of different return periods of earthquakes of different magnitudes, different levels of risk acceptance and need to have a common set of guidelines leads to adoption of probabilistic approach in deriving these values of peak ground accelerations for the purpose of design. The entire region is divided into different seismic zones on the basis of knowledge of past seismic events and understanding of the local seismic activity. The values of peak ground acceleration to be considered in the design are defined, zone wise, in the form of numerical coefficients which along with other parameters, that influence the response of the structure, are to be used to arrive at the value of acceleration to be used to determine the magnitude of the inertial forces experienced by the structure during an earthquake.

Other parameters that influence the response of the structure are

1) Energy dissipation capacity of the structure: Materials, type and characteristics of the structure influence the energy dissipation capacity of the structure through ductile behavior. Magnitude of inertial force needs to be modified using these response reduction factors. These are also specified by the national authorities for different types of structural configurations.(This factor accounts for nonlinear response of the structure)

2) Importance Factor: The values of seismic input, in the form of peak ground acceleration, can be increased or enhanced according to the different levels of risk acceptance. National authorities have specified these modification factors which a designer can use according to the intended importance (in terms of public safety & national security etc.) of the structure under consideration.

3) Response Spectrum: It is the most important information to derive the magnitude of the acceleration experienced by the structure during an earthquake. It is presented as a graph of measured values of peak acceleration of different oscillators of single degree freedom, when they are excited with a common source (specific ground motion) to vibrate with their respective natural frequencies. Time periods (inverse of natural frequency) of these oscillators are plotted along X axis of this graph and the corresponding acceleration, expressed in terms of gravitational acceleration, along Y axis. As the ground acceleration is known to be influenced by the nature of soil (Hard, medium, soft) in a defined manner, this relationship is also incorporated in the graph (Response Spectrum) provided by the national authorities. (Fig. 1)

The task of determining the magnitude of acceleration (and therefore the force) experienced by the building during an earth quake can be thus accomplished by combining seismic properties (peak ground acceleration values corresponding to different seismic zones) and structural properties (acceleration corresponding to structure's natural frequency and the level of ductility of the structure) as defined by the national authorities.

The design procedure can now be organized through the following steps.

1) Identify seismic zone and select the Zone coefficient, Z.

2) Select Importance factor

3) Select Response reduction factor

4) Calculate Time period (inverse of natural frequency) of the structure under consideration

   For regular (regular stiffness and mass distribution) and smaller buildings the national authorities provide a formula to calculate the 'Time Period' of the building structure under consideration.

5) Find the value of acceleration corresponding to the 'Time Period' from the

   Response Spectrum

   It must be stated at this stage that national authorities allow the use of a simplified method called 'Equivalent Static Force Analysis' for analyzing regular and small buildings. The Dynamic (time related) response of the structure is assumed, in this method, as a static response.

   It must be understood that the building structure has its mass distributed as lumped masses at different floor levels (Fig.2) and the structural model is in fact a combination of different degrees of freedom systems. When excited by varying frequencies of the ground motion in a very short interval of time, the structure will vibrate at different natural frequencies corresponding to different degrees of freedom and will go through different deflected shapes (Modes). Inertial forces will be experienced by different lumped masses at different levels in different directions, as per the directions of displacements of these masses, in different modes (Fig.3). The simplest form to handle is the first mode of vibration which causes displacements of all lumped masses in one direction and allows linear distribution of inertial forces along all floor levels to match the deflected shape of the structure under the action of these forces. 'Equivalent Static Force Analysis' is done by distributing the inertial force, maximum at the highest lumped mass location where displacement is maximum and zero at the base where displacement relative to the ground is zero, using a specified formula.

6) Calculate the total inertial force experienced by the building mass by multiplying

   'Acceleration' with the 'mass' of the building and values of other parameters as defined by the national authorities. This is called 'Base Shear' as it is resisted by the building structure in the form of a shear force at its support.

7) Distribute the total inertial force (Base Shear) at different floor levels using the

   Formula provided by the national authorities.

   Simple static analysis can be carried out of this configuration of the forces and the structure and structural elements can be designed accordingly. .

However application of this method is limited to regular and small structures. In case of non regular structures higher modes of vibration also become significant and the corresponding effects need to be considered in the analysis. Normally, in Response Spectrum Method of design, first two or three modes are considered to be of significance. Computer analysis (modal analysis) may be used to determine these modes (mode shapes) and the time period (T) of each mode of the vibration of the structure. The values of acceleration corresponding to different Time Periods are obtained from the acceleration response spectrum. Inertial forces experienced by different lumped masses (at floor levels) are then calculated and so also the effects of these forces. It may be noted that in higher modes, some of the inertial forces act in opposite directions according to direction of displacement of different masses at different levels thus reducing the overall effect of the horizontal force considerably as compared to that in the first mode wherein direction of all inertial forces is one. Effects of the horizontal inertial forces acting on the different floor levels of the structure of all the modes (mainly first two or three) are combined to arrive at the magnitudes of design parameters such as bending moment, shear and axial force.

Response Spectrum Method also has its limitations and it is not considered satisfactory for buildings which are too irregular, too tall or of high importance.

Although used widely, this method assumes linear elastic behavior of the material and gets restricted with the elastic limit of the materials.

For very complex or very important or very tall buildings it would not be appropriate to rely on the generalized seismic input given in the form of zonal seismic coefficients. Also for such structures a detailed analysis involving estimates of component displacements for each degree of freedom in the model, in response to combinations of specific ground motion records (natural as well as machine generated) needs to be carried out. As the nature of response is dynamic (varying with time), a rigorous dynamic analysis by considering the non linear properties of the structure needs to be carried out.

Advance mathematics, analysis capabilities of the 'Finite Element method' and very high computational power of the present day computers has helped the development of software programs that carry out these complex looking earthquake resistant design calculations very quickly and efficiently. However it requires sound engineering judgment and experience to handle this subject and blind use of computer software can lead to wrong results.

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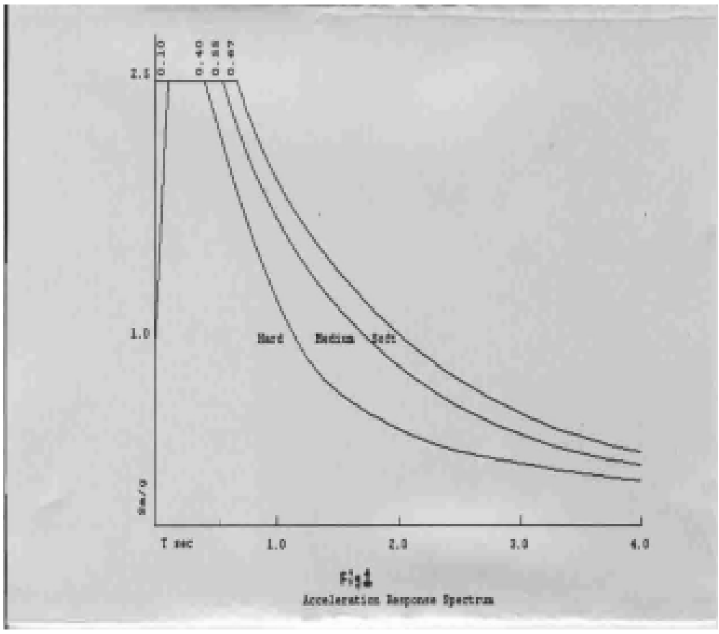
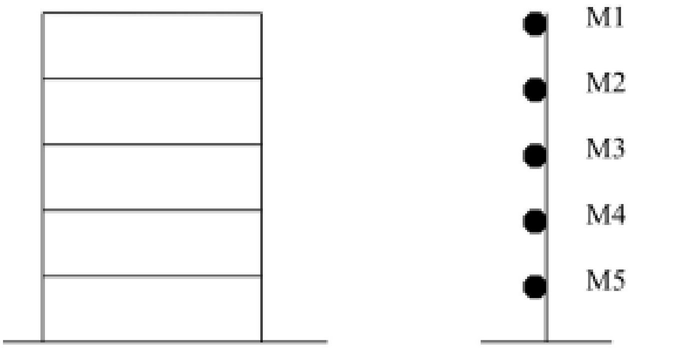


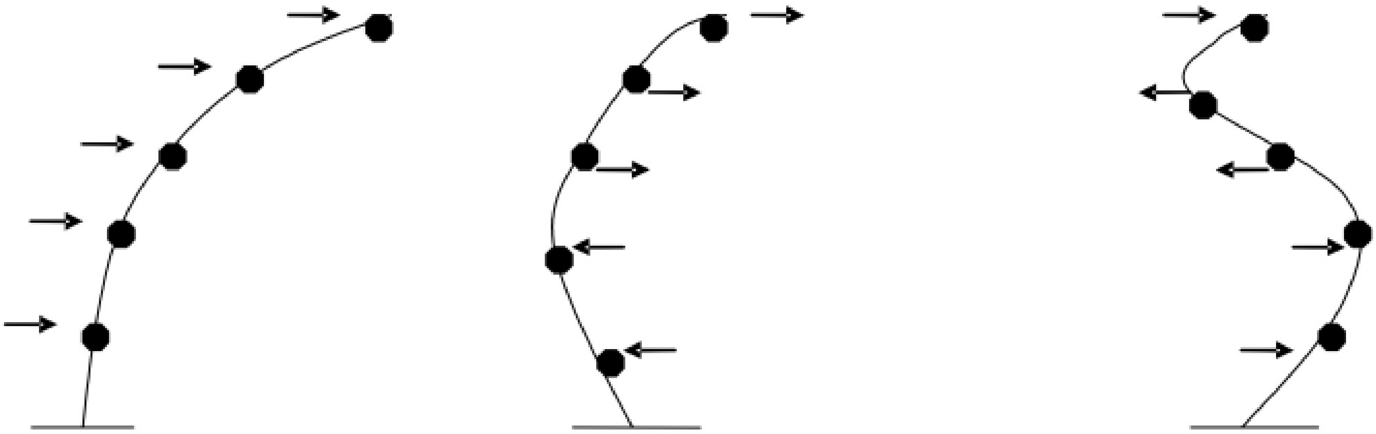
Fig1
Acceleration Response Spectrum



Building Structure
(Frame – Elevation)

Lumped Mass System

Fig. 2



Mode 1
(Time Period T1)

Mode 2
(Time Period T2)

Mode 3
(Time Period T3)

(Direction of ground motion) (Direction of ground motion) (Direction of ground motion)

Location of Spices for Longitudinal Reinforcement in R. C. Column Use of Weld Able - Couplers

Prof. M.D. Mulay

Introduction :

In R.C framed-structures subjected to earthquake loads, where ductile detailing is necessary as per I- S- 13920 / 1993, the splices for the longitudinal reinforcement bars in the columns are required to satisfy following two conditions.

1. The splices are required to be provided in the central half portion of the clear height (H_C) of the column.
2. Not more than 50% of the bars should be spliced at a given level / section in the column.

In many of the residential buildings, as per the architectural requirement the available clear height of the column is about (2.2) meters. Hence the central half portion will have the available length = 1.1 meter, bars of Fe 415 in concrete of grade M-30, the lap length required is nearly 1200 mm, which is more than the available length of 1.1 meters. In case grade of concrete is (M-20), the Lap length required is nearly (1290) mm, where by about of 10% of the lap length of the bar has to be extended beyond the central half portion of the Column on both side. It is not clearly understood form the code, whether such extension of lap in end-portions of column is permissible. Further for the remaining 50% of the vertical reinforcement, the lapping – splice is then necessarily required to be provided in the upper – storey where by required the length of reinforcement bars are necessarily to be more than two the storey height. This aspect is also explained qualitatively in section (5) of the text - book on "Design of R.C. Structures for Earthquake-resistance" published by I.S.S.E.

For buildings where clear storey height is (4.4) meters or more, it is possible to accommodate two lapped type of splices for the adjacent reinforcement bars in the central half length of (2.2) meters, if each of the lap length (~ 40 x diameter) is less than 1100 mm, so that these adjacent laps will not overlap. It is necessary that the distance between Centre to Centre of two-splices for adjacent bars in the column should have been specified in the codal provision. However it may be rational and safe to assume that the lap-length of splices for two adjacent bars should not overlap so that there is no stress concentration in the region over a given length of lap.

Use of Mechanical – Coupler:

Today mechanical couplers are being used, where in the c/c distance between two splices of adjacent bars can be reduced significantly depending on the length of such couplers, which is substantially smaller as compared to length of lapped-bars. Thus it is possible to portion of the column, ever if available length of this portion is about 1100 mm.

Proposal for Weldable – Type Coupler:

As an alternation to the mechanical couplers, it is also possible to design and develop a Weldable Coupler based on the following construction aspect. Since this is a new concept, it is necessary to conduct laboratory tests for its tension capacity to verify its validity and standardization.

Fig- (5A) shows the plan and elevation of an R.C. Column, where alternate reinforcement bars marked.

(1) In plan, are shown with splice location level (Y, Y) in elevation. The remaining bars are marked.

(2) With their splice-level at section (Y_2 Y_2). The distance between centers of splices of these adjacent bars (1) and (2) is indicated as (C_5) The length of the coupler shown dotted in the elevation is indicated as (L_c) and the centers of these two adjacent splices are at (M_1) and (M_2) respectively.

This wieldable coupler is basically a collar of hollow-circular section with a designed thickness (t_c) with a maximum length L_C = about (350) mm for 32 mm dia bar as shown in Fig (5 – B). The inner diameter (D_c) of the coupler is slightly more than the external diameter of the reinforcement bar to be spliced about less than (1) mm, so that the collars can be just passed over the reinforcement bar easily arranging insert – length of half the length of the coupler. The two bars to be spliced are to be aligned co-axially and their ends butting each other at (M) inside the coupler as shown in Fig – (5B). The coupler is provided with dia metrically opposite two vertical notches along its length, with a width of about <u>20 to 25 mm</u> as shown in plan section (Y_3 Y_3) and over two portions marked (T_1 T_2) in the upper part of collar and (B_1 B2) in the lower portion, each length T_1 T_2 = B_1 B_2 = 150 mm or less. The collar remains without any notch in the central portion of length = (50) mm marked as (T_1 B_1). The notches are meant for providing welded connection of a maximum of 10 mm size fillet weld between the reinforcement bars and the collar. Thus the welded connection of the collar along two vertical edges on each of the two notches with the reinforcement bars establishes the splice-joint – tested as " Weldeable-Coupler".

The two important properties required for the coupler are as follows.

a. The material of the coupler should have a good – weld eability property,

b. The material of coupler should have a high strength preferably of grade (F_e 350) with minimum yield strength of about 350 N/mm $^{2.}$

Design – Concept:

For the purpose of designing the thickness (t e) of Weldeable-coupler and deter mining the length of weld of a given-size, we may adopt the following philosophy- Assuming the reinforcement bar of Fe 415 grade, it is preferable to choose coupler of yield strength of 350 N/mm^2 or more. The yield capacity of the coupler should be at least equal to (1.2) times the yield strength of reinforcement – bar. The corresponding equation can be written as

$$A_{c\,(net)} \times f_{y\,(c)} \;//\; 1\text{-}2\, A_B \times f_{y\,(B)} \text{———— (I)}$$

Where: -

$f_{y\,(c)}$ = yield stress of coupler, say (350) N / mm^2

$f_{y(B)}$ = yield stress of reinforcement bar,

A_B = effective Area of cross – section of the bar.

A_C = Net area of Cross – section of the coupler after deducting area lost in two-notches assuming its maximum width to be 25 mm.

Considering a reinforcement of 32 TOR (Fe – 415 grades), it can be observed that a 12 mm thick coupler of grade (Fe-350) is satisfactory as shown in typical calculations in Appendix (I-A)

Further considering fillet weld of (10) mm size and assuming the working permissible shear stress in the site weld to be 85% of (108) N/mm^2 (i.e. 91 N / mm^2), the strength of the weld per mm run can be computed as follows.

Working-strength of 10mm fillet/$_{mm}$ = (10 x 0.707 x 91) N/$_{mm}$

= 643 N /$_{mm}$

Limit value of this should be greater than (1.2) times the yield strength of the bar, which is the right-hand side of equation (I). It is observed that the length of weld required along each edge of the notch is about (125) mm available including the two end-returns and hence the for notch length of (150) mm is adequately safe 32 mm diameter bar.

Conclusion:

It is therefore observed that even if the clear vertical distance between the two adjacent couplers is Kept as (350) mm, both the splices for 32 mm diameter bars can be easily accommodated in the central half clear length of the R.C. Column, if this even available distance is (1100) mm as shown in Fig (5A) – Elevation along section (X, X,). The distance (Cs) between Centre to Centre of the adjacent 8 / (27) splices can be about (700) mm for 32 mm.

It is worth-stating here that tension test should be conduct upto failure in a well-equipped Laboratory to verify that the strength of the splice is more than ultimate strength of reinforcement bars. Thus the welded connection of the coupler should remain intact and the bar should fail in the test indicating that the coupler connection is stranger than ultimate strength of the bar. At least about six samples of splices for a given diameter of bars, should be tested for validation so that certain practical considerations like a) Tolerance in the measurement of thickness of collar, b) Tolerance in the measurement of size of weld, c) Clearance between the coupler and the bar etc, can also be accounted for and the dimensions of the collar can there be suitably modified / finalized. The recommended dimensions of the collar are presented in a Table (I) in Appendix (I – B). It is also felt that if the two bars of different diameters are to be spliced (say 25 ϕ bar with 20 ϕ bar), it may be possible to use a Weldeable coupler corresponding to the higher size bar (25ϕ). Even though the clearance for 20 ϕ bars happens to be more by about 2.5 mm, the tension test of welded connection will indicate the suitability of its use. This could be a are point with respect to a mechanical coupler, case of lapped type of splices, the length of the lap required depends on the grade of concrete which is not the case when couplers are used. Further since the placement of the two-bars to be spliced are co-axial, it maker a more efficient splice, with respect to lapped-type splices. Table (I) shows that the length (L_C) of the coupler can be in the range of (250 To 350) mm.

APPENDIX – (1 – A)

Typical calculations for design of Weldeable coupler of grade (F_e 350) for a reinforcement bar of 32 mm diameter of grade F_e 415.

$A_B \approx$ (804.2) mm^2 (Approximately)

R.H.S. of Equation (I) = 1.2 x AB X $fy_{(B)}$

= 1.2 X 804 X 415

= 400,492 N

Assuming internal diameter (DC) of the

Collar to be = 32 + (ribs + clearance)

$\approx$ 36^{mm}

Adopting Thickness of collar = 12 ^{mm}

DO = External Diameter of Collar ≈ 36 + 2 x 12 = 60 ^{mm}

$$A_C(\text{net}) = \frac{\pi}{4}(60^2 - 36^2) - 2 \times 12 \times 25$$

= 1809 – 600 = 1209 mm^2

L.H.S. of Equation (I) = $A_{c\,(net)} \times f_{y\,cc)}$

= 1209 x 350 = 423,150 ^{N}

> R.H.S. of Eqn – (I).

Hence 12 mm thickness of coupler is O.K.

Length of weld requirement: -

Assuming 10 mm size fillet – weld

Limit strength of weld / $_{mm}$ = 1.5 x 643.4 $^{N/mm}$

⁂ Total weld Length required = $\frac{400{,}492}{965}$ = 415 ^{mm}

Weld length required on each of the edge of Two-Notches = 415/4

= 104 mm

Considering two end returns = 2 x 10 = 20mm

Required total Weld length of weld in notch = 150 mm

Hence certain amount of buffer is also available to account for various practical – consideration mentioned in conclusions. Further exact cutting of notches of required width will also require a precision in cutting system / equipment.

APPENDIX – (I – B)

Table (I) showing recommended Dimensions in mm of the collar (Fe 350) for various diameters of Reinforcement bars of F_e 415 grade.

Dia of bar (Nominal) (F_e 415) in mm	Thickness of Collar (t_c)	Aprons x 1. Dia. of collar		Size of fillet weld (mm)	Width of Notch (B_N)	Length of Notch (L_N)	Length of collar (L_C)
		(D_o)	(D_i)				
32	12	60	36	10	25	150	350
25	12	53	29	8	25	125	300
20	10	43	23	8	20	125	300
16	8	35	19	6	18	100	250

Note:- Dimension can be suitably modified based on experimental results on a set of six – samples of a given diameter and satisfying equation (I).

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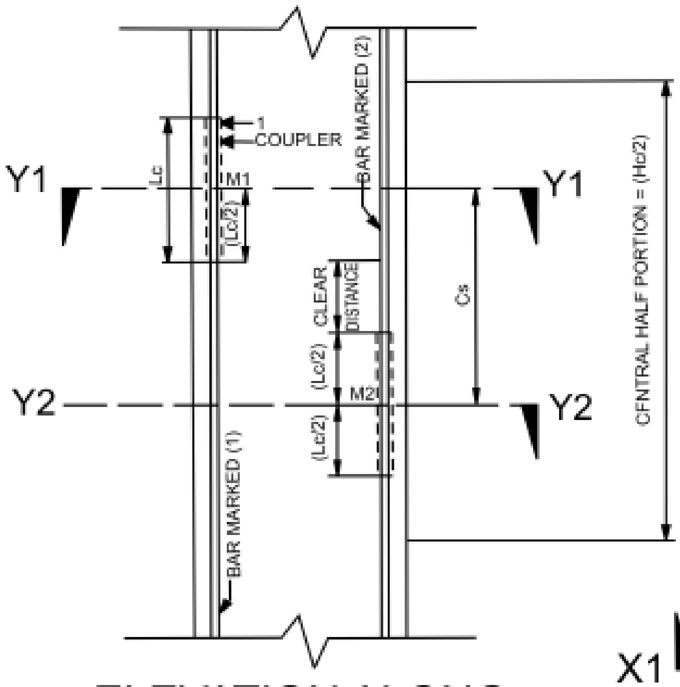
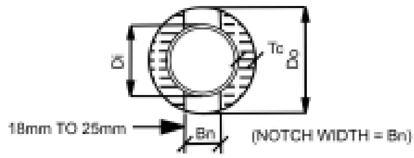
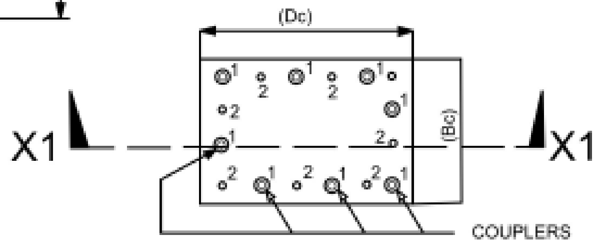


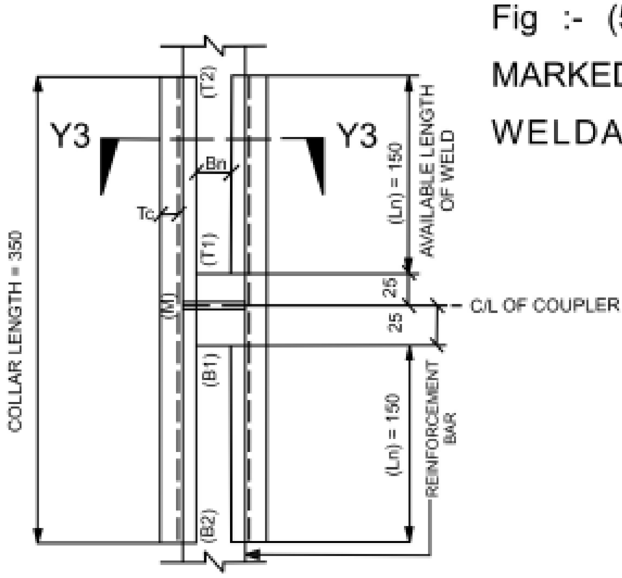
ELEVATION ALONG SECTION (X1X1)



PLAN OF COLLAR SECTION (Y3Y3)



PLAN AT (Y1Y1) OF R.C.COLUMN Fig :- (5A) :-MALTERNATE BARS MARKEDN(1) IN PLAN (Y1Y1) WITH WELDABLE COUPLER SHOW◎



ELEVATION

Fig :- (5B) :- DETAIL OF WELDABLE COUPLER FOR BAR - 32 Φ REFER APPENDIX - (1-B)

Facing Challenges of Common Bad Structural Systems Part - I

G. C. OAK

Bad structural systems root in peer pressures which are caused by nature of our system of building industry. The system consists of Flat purchasers- Builders – Architects – Municipal Building Rules and lastly Consulting Engineers. Though unpleasant, it is grim reality that Engineer is not only last but also ineffective link. Therefore while allowing formulation of structural systems, more importance is given to aesthetic aspects, than to aspects of effective / efficient structural systems for the buildings.

Obviously as a result, consulting engineer is left to accept the resulting structural system (good or bad, nothing is of his choice) and still has responsibility to evolve safe structure out of it. Those who can face this ordeal can survive the professional competition and others who cannot do that are likely to become drop outs!

Insisting on full compliance to codal provisions and on ideal structural systems, is practically impossible in real world of our building industry. That is possible at the most in the dreams of academicians!

The intention of this article is to bring out what are such common bad structural systems and also to present exhaustive study to fellow engineers to face the challenges posed in such scenarios, and also in remedying such scenarios, where ever possible.

I intend to cover following such systems:

A] Avoiding beam below internal wall (at the most by allowing “hidden beam” in the slab)

B] Avoiding tie beam between columns in the enclosed balcony

C] Allowing long cantilevers

D] Providing eccentric footing for boundary column

E] Providing floating column

F] Turning orientation of column above plinth (rotating through 90 degrees), at some higher floors

Now let us study and find out how to face these challenges.

<u>The challenge of Providing wall on slab, without providing beam below the slab</u>

Challenge Details: 2.5 m X 3.5 m slab, simply supported and one-way, 100 thick (600 kg/sqm inclusive of live load of 200 kg/sqm), to support 2.9 m high 2.5 m long 150 th brk wall of 300 kg/sqm, centrally located on the slab.

For very preliminary study, imagine the effects by simple considerations.

Assuming total wall thickness of 150 mm and allowing dispersal of 75 mm, the effective width of strip below wall could be considered as 300 mm. Consider adjacent unloaded strip of same width of 300 mm. Comparative values of maximum B.M., maximum deflections and section design are given below.

FOR 300 WIDE STRIP WITH BRK WALL
WITHOUT WALL VALUES DUE TO

BRK WALL ALONE

LOAD t/m 0.99 0.18
0.87

Max B.M. tm 0.77 0.14
0.68

Max Defln mm 1.00 0.182
0.88

Reinf for 300 width At = 538 sqmm

Ac = 364 sqmm At = 85 sqmm

The strip of slab below the brick wall (spanning 2.5 m) will be very heavily loaded compared to the other similar strips. The excessive deflections of this strip with reference to adjacent strips will tend to cause separation of the heavily loaded strip from them and this strip will also tend to crack under flexure. If sufficient reinforcement is placed across the span (more than the distribution steel) then splitting of this strip will be saved as the adjacent strips will also share some load, redistribution can take place (isotropic action) and effectively more width than 300 mm will tend to resist the heavy loading. With this redistribution, maximum short span moment will get reduced but some moments will develop for long span and deflection curve will develop in both directions!

These phenomena are proved by FEM analysis of the same slab. Details of results of analysis by FEM are displayed on the adjacent page. (Thanks to Con Eng Hemant Vadalkar who carried out the FEM study of this problem and provided results to me.)

For FEM study 2.5 m X 3.5 m slab supported on long edges (one way) is considered as isotropic plate subjected to central brick wall (no loads like self weight, live load etc considered).

Important conclusions from the results of FEM analysis:

Due to the central brick wall

1. If M1is moment for short span and M2 is moment for long span per m width (which is nil for one way slab) additional M1 and M2 due to the central wall for design would be 0.237 and 0.136 tm/m.

2. Without wall the design moments M1 & M2 would have been 0.466 & 0 tm/m, requiring 8 T @ 175 c/c as short span steel and 6 P @ 230 c/c as distribution steel for long span

3. The additional maximum deflection due to the central wall would be 0.73 mm.

4. For small bay of 2.5m X 3.5m, steel of 8 T @ 150 c/c for short span & 8T @ 250 c/c for long span would be required.

5. For larger bays, similar considerations will have to be made and allowance will have to be made for higher values of bending moments **in both directions**

Solution for providing wall on existing slab:

If proposed wall is of full length without opening for door, first provide inverted beam of thickness same as wall thickness and of suitable details. Then construct the wall. Alternatively instead of separate inverted beam, 2 nos of 12 T can be provided on slab in bottom layer of the wall. But for this improvisation, slab needs to be propped before constructing wall, for proper load transfer and thus developing deep wall-beam behaviour.

Generally door opening is needed and in that case above solution based on deep wall-beam concept is not valid. In that case remove existing floor finish and provide at least 600 mm long pieces of 8 T @ 150 c/c across the wall alignment, provide 600 X 600 tiles on 40 thick concrete bedding and 12 T-2 nos at bottom of wall and then construct the wall.

Solution for Providing wall at Design Stage

1. It is very common practice to suggest provision of hidden beam of say 2-12 T at top and bottom. This reinforcement would suffice for the wall, subject to above discussions about deep wall beam action. Without wall-beam action this would not be adequate as could be seen from earlier discussions
2. Alternatively provide additional reinforcements for the additional moments which would be caused in both the directions by the future wall as mentioned above.

IF NO PROVISION FOR WALL CAN BE MADE BY ANY OF THE ABOVE METHODS, MINIMUM SOLUTION IS TO INSIST ON SIPOREX WALL 75 TH WITH SINGLE COAT PLASTER (loading caused would be @ 100 kg/sqm)

The Challenge of Avoiding Beam between columns in enclosed balcony

Because of municipal restrictions on location of columns, Architects frequently resort to gimmicks of “revas projection”, flowerbeds”, “box treatments”, etc for legal (even illegal) gain in floor areas in addition to permissible balcony areas. Thus cantilevering out of floors to the extent of 2.5 m to 3 m becomes necessity in Architectural Planning. Frequently balconies are then enclosed for increasing habitable area of the adjacent room. The room if it is bed room, the presence of beam over bed is undesirable, if it is living or kitchen room, then it is undesirable from the aesthetical aspects! If the municipal rules are changed and if cantilevers on any account and or enclosing of balconies are not allowed, such scenarios will not arise.

Structurally that beam is very much advisable because it ties the columns. Therefore as far as possible engineer should insist on allowing at least small beam of 230 X 300 size. It reduces the unsupported length of columns, helps in creating framing which is good for imbibing / augmenting lateral resistance of building as whole. Therefore the challenge is when the beam is to be avoided.

For allowing deletion of this beam:

1. Check the adequacy of breadth of concerned columns in the direction of the beam. It should be more than 1/12th clear floor height. If the clear floor height is 2.9 m, breadth required is more than 242 mm. Thus for 230 mm width, relevant strength reduction factor will have to be allowed while designing column sectional details.
2. Without framing action, as the column cannot be adequate to become cantilever for full height of building, the column will virtually be supported by diaphragm action of floor slabs when subjected to lateral forces. As a corollary this means that we should have other adequate frames formed by other suitable columns and shear wall (very much preferable). Shear walls can be available at locations of lift, stair, or some dead walls noticed from room plans. While taking advantage of shear wall as well as other frames for lateral loads, care needs to be taken to ensure that adequate dead loads are transferred to them, without which they would not be stable under overturning moments caused by lateral (seismic) loads. This aspect is many times not borne in mind while formulating structural system for building.
3. Though the untied columns will be laterally supported by diaphragm actions of floor slabs, the floor slabs themselves will sway under lateral loads like seismic loads .The relative difference in the sway of two consecutive floors will generate bending moments in those columns, despite the fact that those columns do not form any frames.

The Challenge of Allowing long cantilevers (more than 1.5 m)

1. The reasons for needing long cantilevers are discussed above.

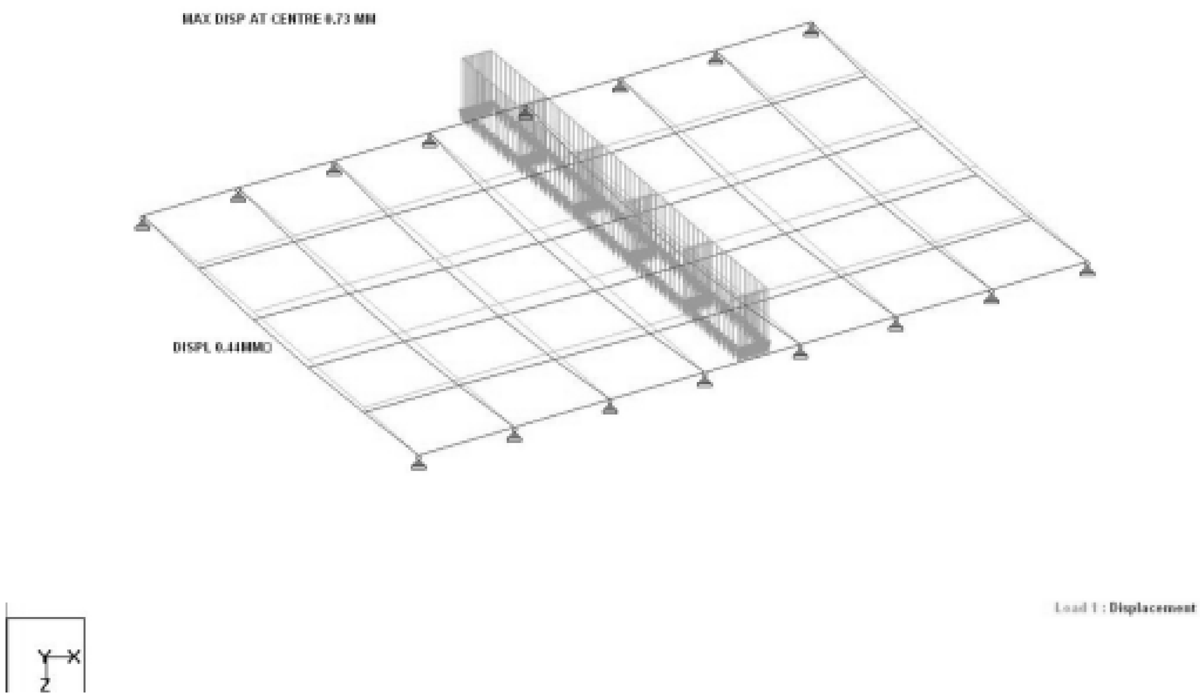
2. For avoiding effects like torsion on supporting beam or moment on supporting column, adequate anchor span (beam behind the cantilever) needs to be envisaged. The anchor span should be of adequate length with suitable dead load to ensure no uplift even for the worst combination of loads on cantilever and anchor span. Generally the length of anchor span should be about 1.5 times that of cantilever and should carry slab load as well as wall load. This aspect is likely to be not possible when longer cantilevers are required to be taken out from smaller rooms like kitchen, bed room etc. In that case special study needs to be made for either avoiding or allowing likely uplift in evolving stable structural systems.

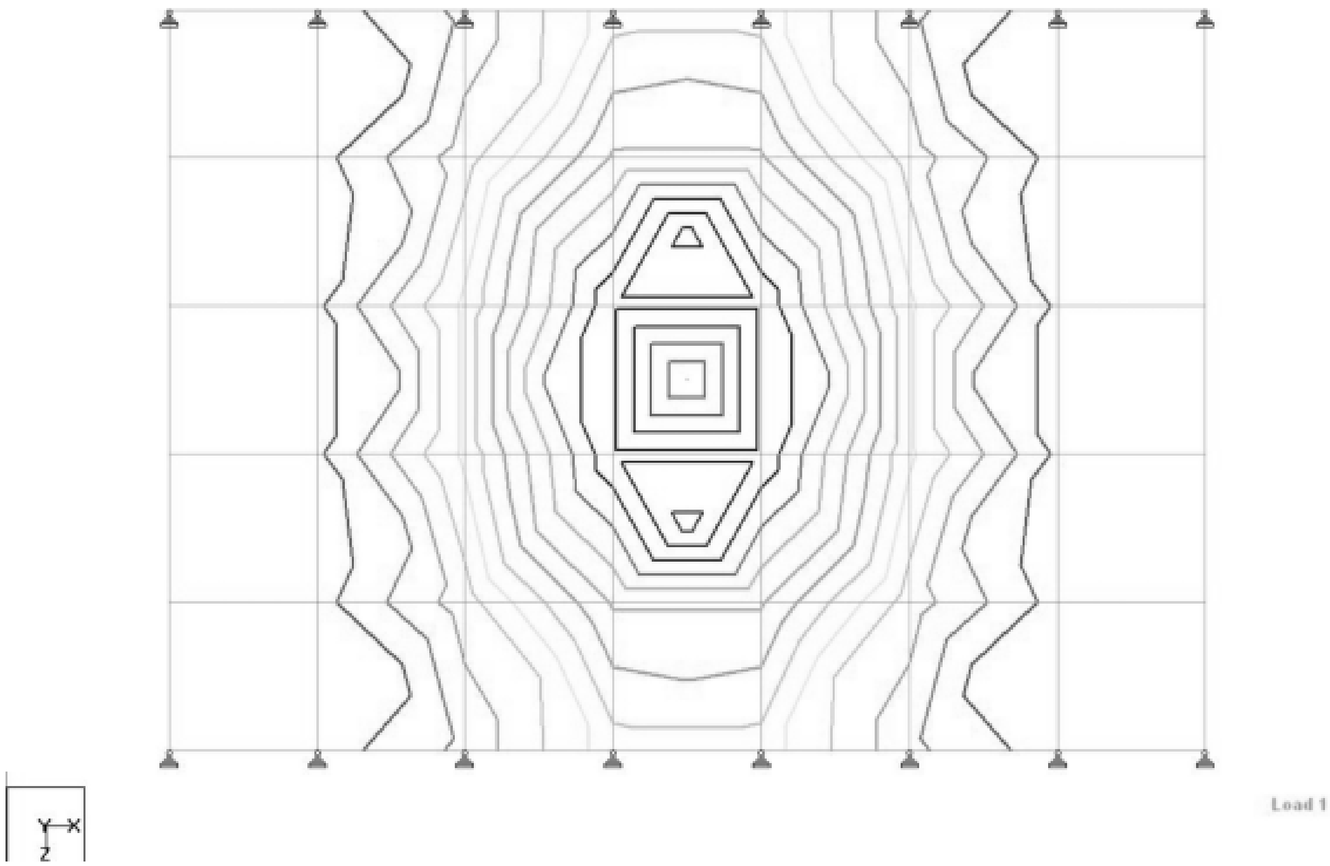
3. The effective depth of anchor span beam should be not less than that of cantilever. If it is less, the top steel of cantilever will not be fully utilized.

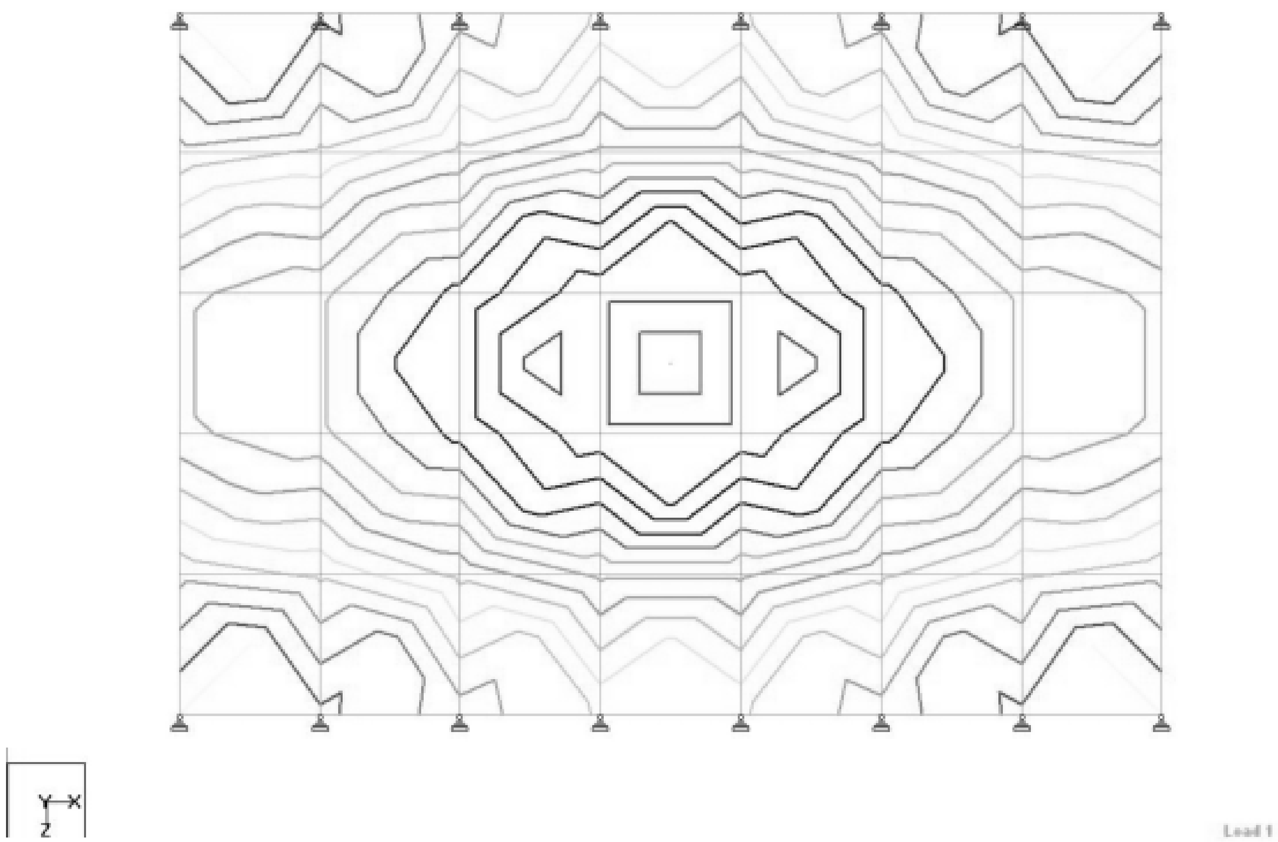
4. While designing cantilever IS clause of considering 5 times vertical seismic force along with other forces should be remembered.

5. In some exceptional cases of cantilevers of either longer spans or heavily loaded spans, where the cantilever depth is required to be 900 mm or more, providing sufficient anchor span becomes problem. For such cantilevers it would be better to develop bracket of full floor depth (providing R.C.C. paradi) and treat it like floor to floor high deep bracket beam . In that case tension and compression components could be provided by upper and lower floors by diaphragm action. I had used this technique for cantilevering out 11th floor by 6.3 m for one prestigious building at Pali Hill, Mumbai.

6. Lastly it should be remembered that cantilever beams (being generally more exposed to air / moisture) are more vulnerable to corrosion damage than the internal beams. Down standing ribs of all beams are generally more vulnerable to corrosion, especially so of the cantilevers as mostly those are on external faces. Moreover in the corrosion damage, the corrosion of bottom steel (generally provided as bent back bars for rings) destroys bottom concrete of compression zone and the cantilever fails / sags due to lack of compression forces. So special precautions in mix of concrete and suitable concrete cover over the bottom steel will be advisable. If the bottom steel is of sufficient large quantity – enough to provide necessary compression force in absence of concrete- then even in corroded state sudden failure would not take place. This of course costs more than the cost of providing steel as per normal design.




Y X
Z
Load 1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Methodology for Soil Investigation

D. J. KETKAR

Introduction :

Site Investigation is the basic requirement of any civil construction activity.

A properly conducted soil investigation should furnish:

1. Background of the previous use of the site and existing structures,
2. General geology and site topography, and surface drainage
3. Methods used for investigation,
4. Clear picture of subsoil strata and engineering properties,
5. Details of the proposed structure,
6. Recommendations of various foundation types and design parameters,
7. Suggest methods of construction for foundations / excavations, and
8. Highlight problems of ground water and any chemical attack on construction materials.

Currently, majority of the site investigation reports mainly contain (1) Details of investigation procedure, (2) Quantities of work done, (3) Subsoil strata with properties and (4) brief details of bearing capacities.

Geotechnical designer solely depends on this Site Investigation Report. However, it is vital that the Geotechnical designer frequently visits the site, during progress of the field work of the Site Investigation, and when necessary he should add more field tests or modify the method of investigation in the field itself.

Eventually we have to have a system of a accreditation of (1) Site Investigation Agencies, (2) Soil / Rock / Water Testing Laboratories and (3) Geotechnical Design Engineers.

The end – product should be a reliable and useful Site Investigation Report

Work of the Site Investigation should involve the following steps :

a. Desk Study
b. Detail plan (of field and Laboratory work)
c. Elements of the Proposed Structure
d. Field Work
e. Testing in the Laboratory
f. Preliminary assessment of Foundations, Construction aspects, etc
g. Additions / Modification to the Field Work
h. Final assessment and recommendations
i. The Soil Report

Each of the above steps will be detailed hereunder and will be useful as a check – list.

Desk Study:

Visit site for surface observations, collect data from enquiries and literature including:

a. Site topography and surface drainage
b. Site Geography
c. Site Geology
d. Past usage of the site
e. Existing Wells / Ponds / Seashore
f. Existing structures / foundations at and around the site
g. Underground pipelines
h. Nearby foundation Types
i. Overhead service lines
j. Vegetation & Trees
k. Layout, Loads and importance of the proposed structures
l. Earthquake zone classification
m. Roads around the site

Planning for S. I.:

Scope and extent of the Site Investigation is to be planned based on (1) past experience of the concerned Geotechnical Design Engineer, (2) Importance and Type of the proposed structures, and (3) Broad Guidelines form the codes.

Broadly the S. I. plan will include:

a. Selection of climatic season for field work
b. Number and Positions of Primary Trial Bores (Cone Tests) Load Tests
c. Type of Boring / Coring / Cone test Equipment and Load Test Equipment .
d. Boreholes for Ground water observations
e. Provision for secondary Trial Bores / Trial Pits / Field Tests
f. Type method and frequency of Sampling (soil / rock / water)
g. Type and frequency of In – Hole Tests (SPT, Vane, etc)
h. Frequency of ground water level observations.
i. Testing schedule for Lab. Tests on samples (soil / rock / water)
j. Guidelines for Preliminary and Final Soil Investigation Report
k. Specifications for each field activity
l. Time frame for various stage of work, viz. Preliminary Trial Bores, Preliminary Report, Secondary Trial Bores, Lab. Testing & Final Report.

Proposed Structure :

Collect a layout drawing at start of the field work. This drawing should also show positions of basements and heavily loaded areas

Field Work : Activities in the field will include

a. Observations of site surface features
b. Observations during Boring / Drilling / Cone Tests, (Such as presence of cavities, ground water levels, side collapse of boreholes, loss of drill water during drilling “)
c. Difficulties experienced in the Filed work (social, natural, climatic)

d. Visual study of soil / rock samples (disturbed, from SPT cone, undisturbed)
e. Packing, Labeling, Storage and Transportation of samples
f. Observations of Ground water level fluctuations (at 6 hour intervals), any artesian pressure and any interconnections.
g. Sealing and backfilling of Boreholes
h. At site, prepare subsoil cross sections from borehole data

Laboratory Testing:

a. Prepare a Table giving sample Nos. and schedule of Tests and due dates
b. Specify Type of Tests (on soil / rock / water)
c. Keep records of all Lab. Test readings
d. Prepare a summary of Test Results
e. Test on concrete cubes / beams with respect to Ground Water

Preliminary Assessment:

This is to be done when about 60 % of the field work is completed.

It should involve assessment of:

a. Type of Likely foundations
b. Safe Bearing Capacity (8 Settlement)
c. Ground Water problems – levels and chemical attack
d. Methods of open excavations
e. Type, depth and Capacity of Piles
f. Likely problems of concrete attack (Surface / piles)
g. List of additional / special Field Trial Bores / in – hole Tests / Sampling / Lab Tests.
h. Hold discussions with owner / consultant for any major changes to the structures

Supplementary Filed Work and Lab Testing:

Generally as per (6) and (7) above, but including changes as per (8) above

Final Assessment:

This should be based on all data from the Field work and the Lab Tests Results. It should include:

a. Recommendations and Design data for Type and capacities pf Foundations.
b. Construction Procedure, Limitations and precautions.
c. Recommendations for Type and use of Cement concrete and steel.

Site Investigation Report:

Broadly the Report should include the following:

- Part A – Site Location and details of the Proposed Structure
- Part B – Geology and Past use of the site
- Part C – Existing under / over ground structures, structures in the vicinity
- Part D – Methods of Field operations, and quantities of work done.
- Part E – Lab. Tests – Methods and Type
- Part F – Plan showing Locations of Trial Bores and other field Tests
- Part G – Borelogs
- Part H – Subsurface profiles across the site
- Part I – Ground Water levels and chemistry
- Part J – Subsoil Layers and properties
- Part K – Recommended Foundation Types, Methods and Design Values
- Part L – Precautions and observations required during Foundation Construction
- Part M – Excavation method and precautions
- Part N – Summary and Remarks

One must remember that the Report should be Reliable and useful to the Geotechnical Designer.

Concluding Remarks :

a. The site Investigation activity needs continuous involvement of a Geotechnical Designer.
b. Proper plan for the Site Investigation Work is essential.
c. Scope of the Fieldwork may have to be modified, based on the early data from the fieldwork. This may involve change in the building layout / basement locations – in consultation with the owners.
d. Once all the field work is completed, nothing can be done to get further data or needed information.
e. Site Investigation work has to be assigned to a competent and reliable Agency. Otherwise one can be in trouble during actual construction (of the structure) which may lead to loss of time & money and litigation. Sometimes, it may compromise safety for the structure itself. Prevention is always better than cure.

As a simple definition, Grouting can be described, as

a. A Process of Introducing
b. Appropriate materials (Called Grout)
c. In the existing formations (soil / rock / masonry / concrete),
d. To improve properties (strength, permeability) of the formations,
e. To achieve desired performance of the Engineering Structures (Dams / Bridges / Foundations).

Sometimes, the Grout itself can be used as a construction material (Colcrete, Gunite, Anchors).

Execution of the Grouting process has many similarities with the practice of Medicine such as.

a. Examination of the Symptoms,
b. Diagnosis,
c. Selection of Medicines ,
d. Administering the Medicines,
e. Observing the progress of curing
f. Modifying the Treatment
g. Confirming the successful Results

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PRODUCT REVIEW

We have now introduced a new section, "Product Review" into the ISSE journal. This is where manufacturers and dealers can introduce their products such as construction materials, chemicals, equipment, software etc, through a technical review. Only one product review may be printed in each issue. A space of up to two pages of the journal may be allocated to this feature.

The main purpose of this feature is to introduce the newer products available in the market to our readers, and therefore, the review should be technically intensive. The manufacturers and dealers can highlight the advantages and uniqueness of the featured products in the review.

The review should cover one or two products only and may include their technical specifications, method of installation/ application, available product range, unique features, advantage, photographs etc. It should not be a direct commercial promotion of the products. However, the contributor may include his contact details at the end of the review. Matter received may be suitably edited and modified in consultation with the contributor.

For details please call the editor.

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UPDATE YOUR CONTACT DETAILS ON
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Our website www.isse.org.in contains the latest particulars of our members. The website also offers you a facility to update your own particulars in the database. However, we have observed that many of our members have still not updated their contact details such as postal and email addresses and telephone numbers. As a result, we can neither send them the Journal nor mail them information about our forthcoming events.

We earnestly request you to visit www.isse.org.in and update your particulars as soon as possible so that you do not miss the Journal or important announcements.

MISSING YOU

In the past few months, ISSE journals and other correspondence sent to some of our members have been coming back undelivered. It appears that they have moved to a new address. A list of such members is given below. We are very keen in having their latest contact details with us so that we can reach them in future.

***I**f you know any of these members, please ask them to get in touch with us (Phones: 022-24365240, 24221015) or send an email to isse@vsnl.net or update their contact details through our website www.isse.org.in.*

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Seminar on Structural Detailing in RCC Buildings

A Report by Rupali Joshi

Indian Society of Structural Engineers (ISSE) organized a one-day seminar on “Structural Detailing in RCC Buildings” at the auditorium of the Institution of Engineers, Mumbai on May 26, 2007. The seminar was organized in association with the Institution of Engineers (I), MSC and was sponsored by CADS Software India (P) Ltd.

Theme:

Detailing of structural components, joints and supports is very important for achieving the desired structural performance and load transfer in an RCC building. There are two requirements of good detailing. First, it should ensure that the assumptions made during structural modeling are properly translated into construction. Second, it should ensure unambiguous instructions to the contractor so that very little is left to “good construction practices”. Detailing is both, an art and a science. For years, detailing has been largely synonymous with reinforcement detailing. However in today’s context, it encompasses a lot many other things such as seismic considerations, provisions for services & equipments, connections, construction & expansion joints, construction sequence, soil-structure interaction and use of special products. Good structural detailing is the hallmark of a good consulting engineer.

Proceedings:

At the beginning Mr. S.G. Dharmadhikari, President, ISSE gave the welcome address. Thereafter there were five technical sessions and a session by the sponsors. This was followed by a questions and answers session. Mr. Umesh Dhargalkar, Advisory Trustee of ISSE conducted the proceedings of the seminar. Mr. Shantilal Jain, Advisory Trustee of ISSE, offered vote of thanks. About 200 participants attended the seminar.

Technical Sessions:

The first technical session was by Dr P.C. Basu, Director, Civil & Structural Engineering Division of Atomic Energy Regulatory Board (AERB), India. He dealt with earthquake resistant design of RCC buildings and the requirements of detailing for such design.

The second session was by Mr. S.C. Ghate, a consulting structural engineer. Mr. Ghate stressed the highly demanding requirements of detailing and documentation in international design assignments and provided illustrations in form of typical drawings.

The third session was a brief talk by Prof M.D. Mulay a professor of civil engineering (retd.), VJTI, Mumbai and Past President, ISSE. With the help of sketches and notes, he dealt with some aspects of the codal provisions applicable to RCC buildings.

In the fourth session, Mr. Nitin Doshi, a consulting structural engineer, stressed the importance of good detailing by discussing many case studies from his practice as a consultant in structural rehabilitation of buildings.

This was followed by a technical session by the sponsors. On behalf of CADS Software India (P) Ltd, Mr. Muniraja and his associates gave a presentation on their detailing software. They illustrated the capabilities of the software to automatically make drawings, replicate them in parts or whole and quickly generate the bills of materials.

The last sessions was by Mr. Hemant Vadalkar who dealt, at length, with various aspects of structural detailing including codal requirements, preparation of drawings, site related problems, dos and don’ts. He discussed these aspects with the help of several sketches in his presentation.

Forthcoming Events

1. ***Forensic Engineering: Failure Diagnosis and Problem Solving***
   International Conference organized by India chapter of ACI to be held from 6th to 9th December 2007 at InterContinental The Grand, Mumbai, India
   Call for Papers – Abstract / Synopsis: Submission deadline 30th June 2007
   Full Text Submission Deadline: 15th September 2007
   Contact Phones: 91-22-24469175/ 32957023, Email: icaci@vsnl.net
   For details visit www.icaci.com

2. ***Sustainable Concrete Construction***
   International Conference organized by India chapter of ACI to be held from 8th to 12th February 2008 at Hotel Kohinoor Samudra, Ratnagiri, Maharashtra, India
   Call for Papers – Abstract / Synopsis: Submission deadline 31st August 2007
   Full Text Submission Deadline: 30th November 2007
   Contact Phones: 91-22-24469175/ 32957023, Email: icaci@vsnl.net
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Some Facts about Ready Mix Concrete

S. H. Jain

Introduction :

Ready mix concrete popularly known as RMC is being used worldwide and is now becoming common in our country also. Earlier RMC was mainly used in infrastructure projects involving bridges, roads etc. However, now it is progressively being used in building construction as well.

Manufacturing Process:

RMC is manufactured in an RMC plant using a mixer having computerized control over the inputs of various materials. In major projects, RMC plants are installed at the site itself. However, in case of small projects it is not economical or feasible to install an RMC plant at site. In such cases, concrete produced at a plant is transported to the site using a revolving mixer fitted on truck otherwise.

RMC received at site is placed at the required location normally using a pump to ensure proper flow of concrete. The cost of RMC is primarily governed by the following factors:

1. Quantity and quality of cement
2. Plasticizers
3. Materials like Fly-Ash / GGBS
4. Transportation from plant to site
5. Pumping and height of placement

Current Scenario :

When RMC was introduced in Mumbai for the first time, its performance was fairly satisfactory. However, with the increase in the demand and consequent increase in the number of RMC manufacturers, substandard quality RMC is being introduced in the market. Currently many projects are suffering due to substandard quality RMC. Commonly observed defects are as follows:

1. Through and through cracks in slabs visible with naked eye
2. Harmful chemical content beyond permissible limit as specified in the IS code.
3. Concrete poured does not set for a long time sometimes even after 7 days
4. Low compressive strengths reported in cube tests

Probable Causes:

The probable causes may be as follows:

1. Cement manufacturers have stopped producing OPC (Ordinary Portland Cement) and have started manufacturing PPC (Portland Pozzolana Cement), in which fly ash is mixed with OPC. PPC is not found to be suitable for achieving higher grades of concrete.
2. Plasticizer to be added to RMC needs to be compatible with type of cement used. With OPC the compatibility issue is not as serious as it is in case of PPC. Not all plasticizers are compatible with PPC.
3. Prolonged Mixing: When the site to which RMC is to be delivered is very far away from the RMC plant, it is a common practice to add water and retarders to concrete to keep it wet so that it can be pumped. The time taken to reach the destination site may be considerably longer than the time assumed while adding retarders. A possibility of water or additional retarder being added by the transporters (drivers etc) also cannot be ruled out.
4. Lack of proper mix design at the RMC plant
5. Scarcity of qualified concrete technicians.
6. Further addition of fly ash which may be of substandard quality
7. Overdosing of retarders and water

We are aware that due to the use of bad quality cement, many buildings constructed during 1980 to 1985 are coming for major repairs or reconstruction. If the RMC manufacturers do not take necessary care to deliver good quality concrete, the future of all the concrete structures would be at stake.

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Congratulations

Universal Construction Machinery, Pune has started a training center (UCMCT) at its Shiware plant with an objective to provide training to the machine operators. This center has been established in association with and with support of Construction Industry Development Council (CIDC, New Delhi), established by Planning Commission.

Concrete batching plant operators, concrete mixer operators, mini dumper operators etc will be trained and certified through this center. This will improve their operational and safety standards. This training center will also provide job opportunities to rural youth, which is an acute requirement in the context of the present volume of construction.

This center will have facilities like library, audio-visual aids, computers, internet etc. The course will include classroom training, workshops and on site training. The duration of the course would be 3 to 7 days based on the equipment. At the end of the course, the trained personnel will get a certificate and a skill card. UCMTC initially plans to train 2000 personnel per year from all over India. For more information you may visit www.uceindia.com

We heartily congratulate Universal Construction Machinery on their initiative.