STRUCTURAL ENGINEERING

QUARTERLY JOURNAL OF

INDIAN SOCIETY OF STRUCTURAL ENGINEERS

ISSE

VOLUME 9-2, APRIL-MAY-JUNE 2007

• E-mail : isse@vsnl.net • Website : www.isse.org.in

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Editorial

Structural Alterations

Umesh Dhargalkar

It has finally taken a few serious building collapses for the citizens of Mumbai to wake up to the menace called "Structural Alterations". Mumbaikars have once again realized that their homes and offices may not be safe – even if the condition of their building appears to be good – if a member carries out structural alterations out of sheer ignorance or callousness. This time around, people are showing significant awareness and anxiety as can be seen from the spurt of telephone calls received at some consulting engineers' offices. In some cases, this awareness and anxiety has led the members of the concerned cooperative housing societies to be more vigilant about any renovation works going on in their buildings.

Such collapses have also caused people to recognize the role of a structural engineer in the construction and repair/rehabilitation of buildings. People have started talking about load transfer, overloading, structural alterations and stability of buildings.

The root causes of internal alterations within flats are scarcity of space (and the ever increasing property prices), an urge to renovate one's home and of course human greed. Since none of these can be removed, it would be impossible to eliminate the possibility of alterations in the present social fabric and political system. We can, however, try and control or regulate them.

In the wake of the recent collapses, many cooperative housing societies have resolved to allow only those alterations, which are certified by a structural engineer. In this respect, we have to understand the professional and social responsibility that we have to shoulder in a field, which is closely associated with the life and living conditions of people. It is sad that sometimes even our own professional brothers take a simplistic view when such proposals are referred to them and fail to think of the long-term effects of the proposed alterations. Some common perceptions by them are:

- Since the RCC beams and columns would not be touched, the alteration would be safe … *How do we know that the structural members would not be touched? Who will check and verify that structural members were indeed not tampered with?*

- Removals of walls would actually reduce the loads and make the building safer or construction of walls directly on the slabs is acceptable if it is a lightweight masonry or modifications in internal layout or provision of fixed storage do not increase overall loads or provision of internal cutouts and staircases can be designed … *How about the loss of symmetry, loss of diaphragm action or infill wall action, shift of centre of mass and increase in the sagging bending moment in the neighbouring beam? How about wider crack widths, which would lead to loss of durability? How about the present structural capacities of structural members, which may be significantly less than those for which they were designed?*

Another issue is that while assessing the feasibility of such internal alterations, the consultant is likely to assess them in isolation without any regard to the flats in that vertical line or to the rest of the building, which may have already been loaded excessively. Similarly the consultant is likely to be so much preoccupied by the considerations of gravity loads that the seismic consideration may take a backseat.

As structural engineers, we should realize our professional and social responsibilities in carefully evaluating the proposals of structural alterations referred to us and think of the long-term effects of such alterations on the safety, durability and serviceability of the concerned before accepting or rejecting them.

Cable Stayed Bridge over River Yamuna at Allahabad

Cover Story

Introduction:

The concrete cable stayed bridge across river Yamuna at Naini/ Allahabad is the first of its type in India. It is a vital link between the twin cities of Allahabad and Naini, also connecting Allahabad to Mirzapur in Uttar Pradesh and Rewa in Madhya Pradesh. The bridge has a four-lane divided carriageway, complemented by footways and cycle tracks on both sides.

Description of structure:

Total project length including approach roads on either side is 5319 M. The length of the bridge portion including 365 M viaduct is 1510 M. The central part consists of a 630 M long concrete cable-stayed bridge across the deepest channel of the river. The main span of this cable stayed bridge measures 260 M, which is the largest concrete deck span in India to date using latest technology being adopted in the world. The concrete deck, 630 M long and 26 M wide, is tied by 104 locked coil stay cables hanging from top of twin pylons, 90 M high. The pylons are supported on 40 M deep Double-D well foundations. The cable-stayed module is followed by 515 M long, conventional 60 M continuous-span bridge, on shallow channel having deep well foundation and pre-stressed concrete superstructure. The bridge is flanked by approaches with flexible pavement connecting NH-27 on Naini side and having 365m long viaducts connecting old G.T. Road on Allahabad side.

Landscaping of the entire stretch has been done keeping with the environment. State of the art instrumentation work has also been executed on the bridge to allow study of structural response of the bridge, traffic on the bridge and weather parameters. The project was completed in 44 months and opened to traffic in August 2004. Completed project cost including design was Rs. 300 Crore.

Salient Features:

Total Length of Project:	5.319 Km
Bridge Length:	1510 M
Cable Stayed Portion:	630 M long with 260 M main span
Continuous Girder bridge:	515 M long [50 M + 60 M x 7 + 45 M]
Viaduct on Allahabad end:	365 M long [20 M + 25 M x 13 + 20 M]
Approach road:	748 M on Allahabad side
	3062 M on Naini side
Road Under Bridge:	85 M
Owner:	National Highways Authority of India
Design & Supervision Consultants:	
	COWI Consulting Engineers & Planners AS, Denmark
	In Joint Venture with SPAN Consultants Pvt Ltd, India
Contractor:	Hyundai Engineering & Construction Co., Korea
	In Joint Venture with Hindustan Construction Co., India
Suppliers:	
Pre-stressing system:	BBR India Ltd, Bangalore
Locked coil cables:	Usha Martin Industries, Ranchi
HT strands:	Tata SSL, Mmbai
POT-cum-PTFE Bearings:	METCO, Kolkata
Expansion Joint:	Maurer Sohne, Germany

Low Strain Integrity Testing of Piles

V. T. Ganpule and S. M. Gupte

Abstract :-

A large number of bored cast-in-situ piles are constructed every year in and around Mumbai as well as other parts of the country. At times, due to the errors in construction process, the errors in handling of bentonite slurry, concrete withdrawal of casing, lowering of steel and other factors, the defects are introduced in the pile shaft. It is therefore necessary to evaluate structural soundness or integrity of the pile shafts on completion by resorting to non destructive tests (N.D.T.) especially, where non-redundant piles are installed. Many such non-destructive test methods are in vogue in western countries. In India the Pulse Echo integrity test is popularly used these days. The limitations and advantages of the low strain tests are discussed in this paper.

Introduction :-

The low strain integrity test based on pulse echo integrity test was developed by the TNO Dynamic laboratory in Delft, Netherlands. The advantage of the method is that the test can be performed rapidly, without any internal intervention in the shaft at a relatively low cost. Due to cost effectiveness the method is finding wide acceptance. Many a times, the structural engineers are showing undue reliance on the interpretation of the test results by the testing agencies which create lot of confusion. The following paragraphs, present the procedure of the tests, the principle behind the test, as well as the advantages and limitations of interpretations.

Background Theory :-

In theory the method is simple. The head of the drilled shaft is struck with a hand – held hammer. A compression wave is generated that travels down the drilled shaft and is reflected from the base (bottom) of the shaft (or from a defect with in the shaft), and is picked up by an accelerometer or other appropriate transducer at the head of the shaft. In operation, however, the method is far from simple. The signal must be processed to eliminate unwanted wave forms (noise), such as waves reflecting from the sides of the shaft, and the resulting signal must be displayed rapidly for convenient analysis. The time for the wave to travel down the shaft either to the uppermost defect or to the base of the shaft and back again to the surface can be read from the signal as it is displayed on the screen of an oscilloscope or computer that displays accelerometer output versus time. With knowledge of the velocity of the compression wave C in concrete, the constructed length of the drilled shaft (or the distance from the top of the shaft to a defect) can be found out.

If there is defect in the shaft, the value of L obtained from the first reflection will be less than the constructed length of the shaft and will in fact be the depth to the defect. A computer can be programmed easily to plot instrument signal versus depth by just multiplying the recorded values of time by C/2. Most experts now use this way of displaying the data. The operators of the test use experimental evidence that correlates the shape of the curve that is found with various kinds of defects.

Inference From Signals :-

It is important to understand that most signals are not so simple as the reflection of wave off the base of a sound drilled shaft embedded in hard rock. But waves can be reflected from any number of locations along the shaft at which the resistance to wave propagation, or “impedance,”changes. Some of the recorded reflections in a test indicate defects and some are unimportant, as explained briefly below.

- Impedance changes occur at levels in the shaft where there is a change in cross – section (e.g., at the base of the drilled shaft) increases in cross section (bulge in a shaft, which would ordinarily be of no concern unless downdrag can occur) can be distinguished from decreases in cross section (either a defect or an inconsequential reduction in cross – section, as often occurs at the elevation of the bottom of a temporary casing) by observing the polarity of the reflected signal. A return pulse of opposite polarity to the incident pulse produced by the hammer,. for example, is an indication of increased impedance, such as may be caused by a bulge in the shaft or by a base embedded in rock that is stiffer than the concrete.

- Impedance changes occur when there is a change in concrete modulus or density or size and such a condition will often be recorded as an apparent defect. Such a situation can be caused, for example, by mixing of drilling slurry with the concrete or honeycombing in the concrete, but it can also result from changing ready mix trucks during a concrete pour in which the modulus and / or density change, although detectable, does not constitute a structural defect.

- Impedance changes also occur due to changes in geomaterial energy transmission conditions along the shift, which has no relation to the structural integrity of the shaft. For this reason the interpreters of pulse echo and similar data (impulse – response, impedance logs) need to have access to the boring data at the construction site.

Because there are many sources of recorded wave reflections, operators of echo equipment are not likely to report "defects" in the drilled shaft. Rather, they are normally in a position to report only "anomalies," or variations in the signal at could, but do not necessarily, indicate a defect. The final decision on whether to treat a shaft as defective is left to the engineer. Sometimes, the size, location and nature of the defect can be simulated using a one – dimensional wave equation program by varying the size, position and stiffness of the defect in the computer code and matching the computed velocity time history at the head of the shaft with that measured by the test. This is an important feature of the original TNO method and sometimes allows for a better understanding of the possible properties of the defect. In some cases the results of such a " curve matching ", procedure may not be unique, however.

Limitations of method :-

First, the further the wave travels along the shaft the more energy it loses, so that deep defects or deep bases are not likely to be detected. An upper limit to the depth to which such tests with modern equipment are useful is about 20m (66 ft). Some experts relate the upper depth limit to length – to diameter ratio and stiffness of the surrounding soil, with a maximum depth – to diameter ratio of about 30.

Second, wave energy is not likely to be reflected from defects unless the defect is either relatively thick or extends nearly across the entire cross – section of the drilled shaft. Schellingerhout and Muller (1996) show that a dramatic reduction in reflected energy occurs once the thickness of a defect drops below about one – quarter of the wave length of the propagating compression wave. For an average hammer impact, the wave length might be around 1.6m (4.6 feet), suggesting the it will be difficult to detect defects thinner than about 0.4m (15 inches). Many types of potential defects can be thinner than this. Samman and O'Neill (1997a) reported an experimental study in which defects that were about 25mm (1 inch) thick could not be reliably detected experimentally by this method. Baker et at. (1993) concluded from an extensive experimental study that low strain methods were not reliable in identifying thin defects that covered less than about 50 per cent of the cross – sectional area of the shaft. Beyond this observation that the defect must be of significant size to be detected reliably by this method, there is no way to tell with present technology how large the defect actually is.

Third, defects or shaft bases that are located below the topmost defect appear not to produce detectable reflections in most instances. For example, if there is a defect at a depth of one – half of the length of the shaft, the wave that reflects off the defect will return to the top of the shaft, reflect off the top of the shaft, return to the defect, reflect off of the defect and return to the top of the shaft a second time just as the reflected wave is returning from the base of the shaft, making it impossible to verify the length of the shaft using this method.

Forth, Samman and O'Neill (1997 a) concluded from a study performed by many consultants that false positives were frequently reported from the low strain tests on short drilled shafts.

Fifth, with the low strain test, it is not possible with present technology to determine in what direction relative to the centerline of the shaft, the defect is located, only the depth at which the defect is to be found. A small defect on the compression side of a laterally loaded drilled shaft will be more detrimental than one on the tension side .

In summary, the low strain test should be considered to be only a very crude screening method that is capable of locating only major defects such as major soil inclusions or bases of shafts that were drilled to the wrong depth. There are only a few types of defects that the low strain test would be able to detect.

It is really essential to know the bore details, soil properties and the pile characteristic before interpreting the result of low strain test, while most of the agencies do not even enquire about the soil report or the field record of piles before giving the report. It is a therefore confirmed opinion of Authors that the low strain test has limited use and should only be used as a sieve.

The Current Scenario:-

As on date the practitioners of low-strain integrity tests are drawing inferences without enquiring about soil investigation, piling records. The inference drawn by them is accepted by many of the structural consultants without applying engineering prudence. The piles are rejected left, right and centre. This creates lot of confusion and illusions about the entire work. From the study of the theoretical background and limitations of the test, it can be easily seen that it is difficult for the low strain practitioners to interpret correctly in absence of the proper theoretical background of geo-technical engineering. The test is conducted by using the same hammer for piles of the sizes varying from 400-1000 diameter. It implies that influence zone of the generated mechanical waves must be up to a radius of o/s (1200 mm-1500 mm dia) 600 – 750 mm. It leads to a conclusion that the effect of surrounding soil plays a significant role in the interpretation. The variation in soil profile can only be reflected that too approximately from the soil investigation data. The piling records shall enable to know the problems during boring like obstructions, change of strata and other problems faced during concreting etc.

As a mater of fact that no attention is paid to these facts, instead only on noting the waves forms and their variations comments are offered.

Moreover, the scale used, plays very vital role while comparing the weave forms of various piles and in such cases the due attention to the scaling is not given. On the whole the results are misleading. It is against this backdrop, it is emphasized that the low strain test only a crude sieve and not totally dependable. At times greener picture is potrayed while at other times a gloomy and dull picture is presented.

It is therefore very necessary to utilize low strain test with caution. There are cases where the reported necking of a pile when exposed showed buldging and vice-a-versa. The purpose of this exercise is not to discourage low strains testing but to highlight anomalies and limitations involved in the technique used. It is possible to provide a guideline by carrying out a detailed research programme wherein a precast-pile is used and the formations around it are changed then such a project can lead to some approximate methodology for rational interpretation.

The authors are contemplating on a schematic solution for the same. But, in the absence of any such guidelines the following procedure of testing and interpretation is proposed which may be of great help to the engineering fraternity.

Suggested Methodology for Testing :-

1) Remove top bad concrete layer and prepare a flat top surface.

2) On dry top surface ensure the contact of transducer by applying grease between transducer and pile top.

3) For pile diameter up to 600 mm place the transducer at centre and of the pile top and give impacts by hand held hammer all round till convergence in the reflectogramms is noticed.

4) For piles bigger than 600 place the transducers in centre of each quarter and hammer all round.

5) Always avoid hitting the reinforcement bars while hammering.

INTERPRETATION OF TEST RESULTS :-

1) The reflectogramme shows the reflected waves, their intensity and time of reflection.

2) As the main reflection should come from end of piles, either length of pile or the velocity of stress wave has to be fed in the programme.

3) The intensity of reflected wave gets reduced as distance travelled increases the intensity is modified (amplified) in logarithmic scale.

4) For good interpretation amplification is adjusted such that amplitude of reflectogram at zero distance and full length are equal.

5) The similar orientation of reflectogramme as starting indicates necking, intrusion of soft material crack or increase in impedence.

6) The reverse orientation indicates bulding or decrease in impedence.

7) For better interpretation, the pile chiseling (penetration) record, the soil profile, water table records have to be studied together with reflectogramme.

Ref. - Drilled shaft construction procedures and desig methods (Reese and Onivl)

1. F. P. S guide lines for load tests on piles.

2. Various publications of Federal Highway department U. S. A.

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2. S. M. Gupte is a practitioner in integrity and dynamic testing of piles.

Torsional Provisions in IS :1893 - 2002

Rudra Nevatia

ABSTRACT

New clauses were introduced for torsion of symmetric as well as asymmetric buildings with rigid diaphragms in the revised Indian seismic code. The treatment of torsional provisions is elaborated here along with a solved example. Torsion has been identified as one of the most prevalent contributors to seismic failure of buildings. Plan irregularities in buildings due to mass or stiffness result in significant torsional response. Table 4 of IS:1893(2002) defines such irregularities and requires dynamic analysis if the maximum storey drift, computed with design eccentricity at one end of the structure transverse to the axis is more than 1.2 times the average of storey drifts at the two ends of the structure. For buildings with nominal plan irregularity not covered by Table 4, Section 7.9 is applicable.

The operative Clause 7.9.2 reads as follows:

The design eccentricity, edi to be used at floor i shall be taken as:

edi = 1.5esi + 0.05bi or = esi - 0.05bi

whichever of these gives the more severe effect in the shear of any frame where

esi = Static eccentricity at floor i defined as the distance between centre of mass and centre of rigidity

bi = Floor plan dimension of floor i, perpendicular to the direction of force

Under seismic loads, structures experience lateral forces acting, in general, at a design eccentricity edi with respect to a neutral point, such that deflections on the side towards edi are higher than those on the other side of the neutral point which does not deflect under torsional loads. The sides towards and away from edi are known as the flexible and stiff sides respectively.

Multiplier 1.5 on esi in the first equation is the amplification factor to account for possible coupling of torsional and lateral modes of vibration and depends on the ratio of frequencies in the two modes. When frequencies in the two modes are far apart, dynamic amplification factor is 1.0 as in the second equation.

Accidental eccentricity due to possible variations of live load, stiffness and ground motion along the width of building is given by 0.05bi. Obviously, this factor can take positive or negative value.

Two cases are possible:

1. Lateral-torsional mode coupling occurs and accidental eccentricity is in the same direction as the static eccentricity which is reflected by the first equation. In general, this is the governing case for members on the flexible side.
2. Lateral-torsional mode coupling does not occur and the static eccentricity is in the direction opposite to the static eccentricity which is what the second part of the equation implies. In general, this is the governing case for members on the stiff side.

   Seismic force acting at the code specified design eccentricity results in torques at various floor levels. There are two approaches to account for this effect.

1. Floor Torques about Centers of Rigidity:

Static eccentricity is defined as the distance between the center of mass and the center of rigidity at a given floor. Centers of rigidity are points on each floor of a multistoreyed building such that lateral loads applied through them do not cause rotation of any of the floors [1].

In order to locate centers of rigidity, the following procedure is adopted:

1. The structural models constrained to deflect only in the direction of applied seismic loads along x and y axes are analyzed.

2. Free body diagram of each floor is taken along with storey shears vi+1,j and vi,j above and below that floor respectively where subscript i refers to storey and subscript j refers to shear resisting element of that storey.

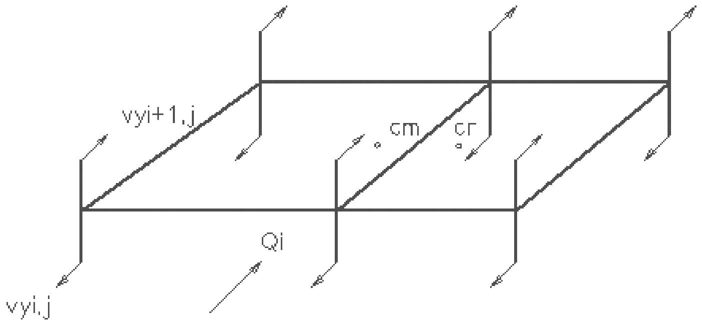

FIG. 1 Free-body Diagram at Floor Level

3. The point of intersection of resultants of net storey shears (vi,j - vi+1,j) along the orthogonal axes is the center of rigidity for the storey.

A pair of design eccentricities and the resulting floor torques at each storey can now be calculated. These floor torques are applied to a three-dimensional frame model taking due care of the fact that 3D frame analysis accounts for static eccentricity 1.0 esi automatically.

2. Storey Torsion about Shear Center:

Static eccentricity is defined as the distance between the center of cumulative mass from roof down to the level under consideration and shear center at that level.

In order to locate shear center, the following procedure is adopted:

1. The structural models constrained to deflect only in the direction of applied seismic loads along x and y axes are analyzed.

2. Free body diagram of the substructure from roof down to the level being considered is taken along with shears Vxi,j and Vyi,j at the cut where subscript i refers to level and subscript j refers to shear resisting element at that level.

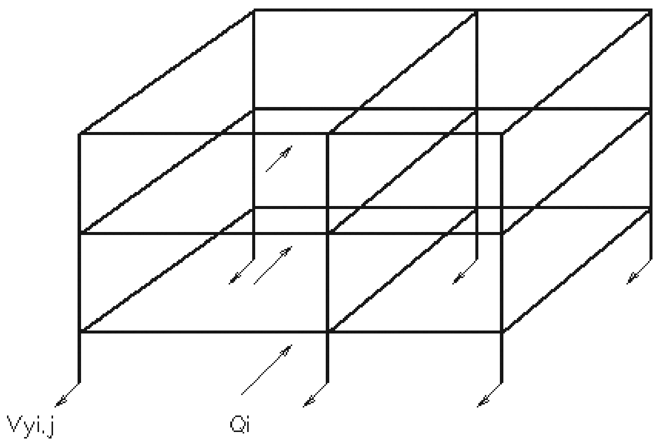

FIG. 2 Free-body Diagram at Storey Level

3. The point of intersection of resultants of shears Vxi,j and Vyi,j defines shear center at that level.

A pair of design eccentricities at the level under consideration can now be calculated.

Analogous to the quantity (1+6e/d) commonly used to calculate maximum pressure in an eccentrically loaded footing, magnification factors on all forces and moments obtained in step 1 above are given by:

äxi = 1 + ediy*yi,k/r_k^2
äyi = 1 + edix*xi,k/r_k^2

where

äxi = Magnification factor for frames in x direction at level i
äyi = Magnification factor for frames in y direction at level i

Vxi,j = Shear along x direction at level i in column j
Vyi,j = Shear along y direction at level i in column j
edix = Maximum additive design eccentricity at level i along x axis

ediy = Maximum additive design eccentricity at level i along y axis

xsci = x ordinate of shear center at level i = ÓVyi,j*xi,j / ÓVyi,j

ysci = y ordinate of shear center at level i = ÓVxi,j*yi,j / ÓVxi,j

xi,j = x ordinate of j-th column at level i

yi,j = y ordinate of j-th column at level i

r_k = Radius of gyration of stiffness
= (ÓVxi,j*(ysci-yi,j)^2/ÓVxi,j+ÓVyi,j*(xsci-xi,j)^2/ÓVyi,j)$^{1/2}$

It must be emphasized that, in general, location of shear center is different from center of rigidity. Tso [2] has shown equivalence of the two procedures given above.
The second approach is computationally simpler and will be used to illustrate the effect of torsion on the two storeyed building shown below consisting of ground, first floor and roof levels

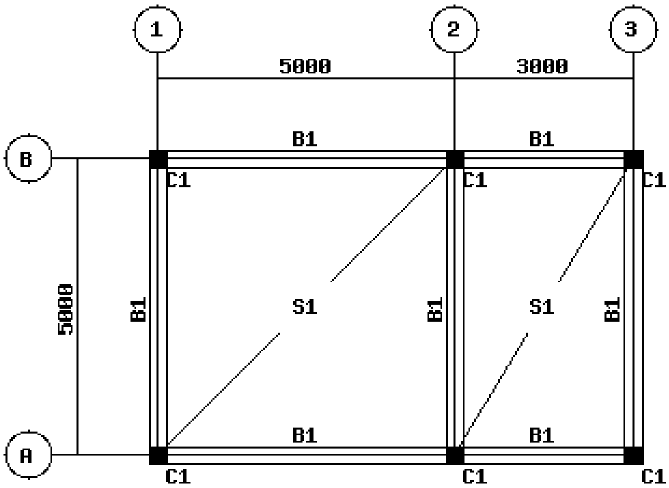

FIG. 3 Framing Plan

The data for this example is as follows:

Column Data			Beam Data				Slab Data			
Mark	Cx mm	Cy mm	Mark	bw mm	D mm	Wall Load kN/m	Mark	t mm	Loads kN/sq.m Dead	Live
C1	300	300	B1	300	500	6.25	S1	150	1.25	5.00
			B1R	300	500	2.25	S1R	150	3.25	1.00
			B1G	300	400	6.25				

Here, suffixes R and G on beam and slab marks refer to roof and ground levels respectively. Storey heights are 3.0 m and foundation depth is 1.5 m below ground.

Seismic Weight wi

Seismic weight at a particular level consists of:

- Dead loads of slab and beams including finishes at the level
- Proportional dead loads of walls and columns above and below
- Appropriate amount of live loads at the level as per code

The following table gives seismic weights at various levels along with x ordinate of center of gravity, xcg measured from lower left corner which will be used later for calculating center of mass. Due to symmetry in vertical direction, ycg is located at 2.5 m for all of the components. Density of reinforced concrete is taken as 25 kN/m^3 for this example.

	Roof	Floor	Ground	xcgm
Slab+finishes	280.00	200.00	0.00	4.000
Beams	116.25	116.25	93.00	4.161
Walls above	69.75	193.75/2 = 96.88	193.75/2 = 96.88	4.161
Walls below	193.75/2 = 96.88	193.75/2 = 96.88	0.00	4.161
Columns above	0.00	40.50/2 = 20.25	40.50/2 = 20.25	4.333
Columns below	40.50/2 = 20.25	40.50/2 = 20.25	20.25/2 = 10.13	4.333
Live	0.00	0.5*200 = 100.00	0.00	4.000
wi (kN)	583.13	650.51	220.26	

Center of Mass

Centers of mass at various levels with respect to origin at lower left corner are calculated as :

xcm = Ówj*xcgj/Ówj
ycm = Ówj*ycgj/Ówj

where subscript j refers to each component of seismic weight from roof downwards to the level under consideration.

xcm at Roof Level
=(280*4+(116.25+69.75+96.88)*4.161+20.25*4.333)/583.13
=4.090 m

xcm at First Floor Level
=(4.090*583.13+200*4+(116.25+2*96.88)*4.161+2*20.25*4.333+100*4)/(583.13+650.51)=4.094 m
xcm at Ground Level

=((583.13+650.51)*4.094+(93+96.88)*4.161+(20.25+10.13)*4.333)/(583.13+650.51+220.26)=4.108 m
ycm = 2.5 m at all levels

Seismic Analyses on Constrained Models

In lieu of analyses, it will be assumed here that all columns share storey shears equally at all levels.

Shear Center

Following the assumption of storey shears being shared equally by all columns, values of Vxi,j/ÓVxi,j and Vyi,j/ÓVyi,j are all equal to 1/6. Thus at all levels,

xsc = ÓVyi,j*xi,j/ÓVyi,j = 2*(0+5+8)/6 = 4.333 m

ysc = ÓVxi,j*yi,j/ÓVxi,j = 3*(0+5)/6 = 2.500 m

Static Eccentricity esi

The distance between center of mass and shear center gives static eccentricity along each of the axes at various levels.

Along x-axis: esix = xcm - xsc

Roof : 4.090 - 4.333 = -0.243 m

Floor : 4.094 - 4.333 = -0.239 m

Ground : 4.108 - 4.333 = -0.225 m

Along y-axis: esiy = ycm - ysc = 2.5 - 2.5 = 0 m (at all levels)

Accidental Eccentricity bi

Along x-axis = 0.05*bix = 0.05 * 8 = 0.40 m

Along y-axis = 0.05*biy = 0.05 * 5 = 0.25 m

Maximum Design Eccentricity edimax

Algebraic addition of static and accidental eccentricities gives maximum value of design eccentricity.

Along x-axis = 1.5*esix + 0.05*bix

Roof : -1.5*0.243 - 0.40 = -0.765 m

Floor : -1.5*0.239 - 0.40 = -0.759 m

Ground : -1.5*0.225 - 0.40 = -0.738 m

Along y-axis = 1.5*esiy + 0.05*biy = 0 + 0.25 = 0.25 m

Minimum Design Eccentricity edimin

Algebraic subtraction of accidental eccentricity from static eccentricity gives minimum value of design eccentricity.

Along x-axis = ediminx = esix - 0.05*bix

Roof : -0.243 + 0.40 = 0.157 m

Floor : -0.239 + 0.40 = 0.161 m

Ground : -0.225 + 0.40 = 0.175 m

Along y-axis = ediminy
= esiy - 0.05*biy
= 0 - 0.25
= -0.25 m

Radius of Gyration of Strength

r_k = (ÓVxi,j*(ysci-yi,j)^2/ÓVxi,j + ÓVyi,j*(xsci-xi,j)^2/ÓVyi,j)$^{1/2}$

The quantities Vxi,j/ÓVxi,j and Vyi,j/ÓVyi,j being equal to 1/6 at all levels,

$r_k^2 = \{3*(2.5-0)^2+3*(2.5-5)^2+2*(4.333-0)^2+2*(4.333-5)^2+2*(4.333-8)^2\}/6 = 17.14\ m^2$

Magnification Factors

äxi = 1 + ediy*yi,k/r_k^2

Frame along grid 1

Roof : 1+(0-4.333)*(-0.765)/17.14 = 1.193 or 1+(0-4.333)*0.157/17.14=0.960

Floor : 1+(0-4.333)*(-0.759)/17.14 = 1.192 or 1+(0-4.333)*0.161/17.14=0.959

Ground : 1+(0-4.333)*(-0.738)/17.14 = 1.186 or 1+(0-4.333)*0.175/17.14=0.956

Maximum : 1.193

Frame along grid 2

Roof : 1+(5-4.333)*(-0.765)/17.14 = 0.970 or 1+(5-4.333)*0.157/17.14=1.006

Floor : 1+(5-4.333)*(-0.759)/17.14 = 0.971 or 1+(5-4.333)*0.161/17.14=1.006

Ground : 1+(5-4.333)*(-0.738)/17.14 = 0.971 or 1+(5-4.333)*0.175/17.14=1.007

Maximum : 1.007

Frame along grid 3

Roof : 1+(8-4.333)*(-0.765)/17.14 = 0.836 or 1+(8-4.333)*0.157/17.14=1.034

Floor : 1+(8-4.333)*(-0.759)/17.14 = 0.838 or 1+(8-4.333)*0.161/17.14=1.034

Ground : 1+(8-4.333)*(-0.738)/17.14 = 0.842 or 1+(8-4.333)*0.175/17.14=1.038

Maximum : 1.038

As expected, frame along grid 1 which is farthest from shear center and on the flexible side experiences maximum magnification factor

äyi = 1 + edix*xi,k/r_k^2

= 1 + 0.25*2.5/17.14 = 1.037 for frames along A and B at all levels.

Storey-wise magnification factors or, conservatively, the maximum value of magnification factor for each frame in x and y direction is applied to all actions found from seismic analysis of constrained model.

CONCLUSION

A procedure is outlined for complying with torsion requirements of IS:1893. The solved example shows significant increase in forces and moments due to earthquake when requirements of Section 7.9 of IS:1893 are accounted for.

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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.

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Guideline Notes for RCC Work

Hemant Vadalkar

Introduction :

Structural engineers carry out structural analysis & design. The design information is converted in the form of RCC drawings for execution of the project. Drawings like framing plans, foundation layout, floor plans and sections along with the RCC details for foundations, columns, beams and slabs are prepared by structural engineers.

It is a good practice to incorporate additional information regarding the RCC work in the form of notes and sketches for the guidance of the executing agency. This should include basic design data, loading data, grade of concrete , grade of steel, cover to reinforcement and certain dos and don'ts. If all this information is given on the drawings, it can be quickly referred at site. This information is also useful in future as it is available along with the set of drawings.

Sample general notes for RCC work are presented here, which can be modified as per the project requirements.

(I) General:

- All the structural drawings shall be read in conjunction with all the architectural, other consultant's drawings, specifications and with such other written instructions as may be issued. All discrepancies shall be referred to the architects and/ or the structural consultants for decision before proceeding with the work on the site.
- Dimensions shall not be obtained by scaling any of the structural drawings. (i..e. No drawing shall be scaled.)
- During construction, the structure shall be in stable condition and no part of the structure shall be overstressed.
- The quality of materials and workmanship shall be in accordance with the latest Bureau of Indian Standard specifications except where varied by the contract documents.
- The overall safety at construction site is sole responsibility of the contractor.
- All dimensions are in millimetres and levels are in meters unless noted otherwise.

(II) Excavation and Foundations:

- The safe bearing capacity (SBC.) of the founding strata/ soil shall be as indicated in the foundation drawings.
- All dimensions and grid line locations shall be checked and correlated with architectural drawings.
- Unless noted otherwise, excavation is symmetrical with respect to the grid lines.
- Unless noted otherwise, foundations, columns and walls are located symmetrically with respect to the grid lines.
- Excavation safety shall be the responsibility of the contractor who shall comply with the requirements of the safety code for excavation work IS 3764 latest.
- If it is not possible to maintain the founding strata at one level, the founding levels shall be stepped down in consultation with the structural consultant.
- If the founding depths are varying, the top of the footing must be below the soffit of the plinth beams.
- Records of actual founding levels shall be communicated to the engineer immediately.
- Excavation shall be kept dry before and during laying of concrete.
- Plain Cement Concrete (PCC) shall not be placed prior to obtaining approval of the engineer.
- Excess excavation under footing shall be filled with well compacted M10 concrete after obtaining approval of the engineer.
- Any undulations in the hard strata must be filled up with PCC, the PCC to be provided below footings should be minimum 75mm thk and should have a minimum offset of 150mm.
- If footings are butting against each other, bituminous paper should be provided to separate their joining faces.

(III) Concrete:

- Materials:
  - Cement: fly ash / slag based composite cement should be used. Use ACC PPC / UltraTech P53/ Ambuja PPC
  - Aggregates: coarse and fine aggregates shall conform to either BE: 383-1963 or IS: 515-1959.
  - Water: potable water is generally considered satisfactory for mixing and curing concrete.
- All reinforced cement concrete (RCC) shall unless noted otherwise be M25 (25 MPa).
- All plain cement concrete (PCC) shall unless noted otherwise be M10/ 40 (i.e. Characteristic strength 10 MPa, aggregate size 40mm)
- Concrete mix in slabs and beams, under and around columns and walls covering an area of four times the gross area of the column or wall shall be of the same grade as that specified in the column or wall drawing.
- All concrete shall be machine mixed, shall be placed and compacted before setting commences and shall not be

subsequently disturbed.

- Curing: Concrete shall be protected against premature drying due to hot weather. The concrete surface shall be kept constantly wet by covering it with a layer of sacking, canvas, hessian or similar absorbent materials, for a minimum period of ten days from the date of placing (or casting) of concrete. Slabs should be cured by ponding for at least ten days.
- Concreting shall be carried out continuously. If construction joints are necessary their position shall be as determined by the structural consultant. When the work has to be resumed, the hardened surface of the concrete shall be roughened and cleaned thoroughly to expose the aggregates so that the joint is free of laitance, loose aggregates and any foreign matter. The joint shall be wetted and covered with 13 mm (1/2") layer of cement and sand mortar (1:2) freshly mixed and placed immediately before concreting. Preferably a good bonding agent shall be used to ensure adequate bond between old and fresh concrete.
- Immediately after removing formwork, any honeycombed and defective concrete shall be brought to the notice of the structural consultant and shall be repaired as directed by him.
- Sizes of concrete elements do not include the thickness of any applied finishes.
- Inverted beams shall be cast simultaneously with the slab but in no case shall the delay in casting the beam exceed 24 hours.
- Cantilever beams should be cast along with the tie-back beams.
- Concrete shall not be poured or placed from a height more than 2.0 meters (6.5 feet)
- Concrete shall not be placed during rains.

(IV) Reinforcement:

- "MS" indicates mild steel bars (IS: 226 or IS: 432) yield stress = 250 MPa.
- "Tor" indicates high strength deformed bars (IS: 1139 or IS: 1786) yield stress = 415 MPa. Use TMT steel: Tata steel / SAIL.
- Structural steel: rolled steel and sections made from structural steel shall conform to IS: 226
- Use of any other steel shall be subject to the specific approval of the structural consultant.
- Unless otherwise noted all reinforcing bar diameters are given in millimetres.
- Clear cover to reinforcement shall be as follows (unless noted otherwise):
    - Pile/ pile cap = 75mm
    - Footing ——bottom = 50mm; sides and top = 75mm
    - Column pedestals = 75mm to links
    - Plinth beams——bottom, sides and top = 40mm to stirrups
    - Columns = 40mm to links
    - Walls = 20mm to bars nearer to the wall face
    - Beams——bottom = 35mm to stirrups; sides and top = 25mm to stirrups
    - Slabs——bottom = 20mm, top = 15mm (to bars nearer to the slab face)
- Cover blocks for footing bottom, beam bottom (including plinth beam bottom) and slab bottom shall be of the u - shaped type, with 6mm maximum size aggregate, of same grade as that of surrounding concrete. For beams sides, (including plinth beam sides), columns and walls, cover blocks shall be of ring type, 25mm thick (mounted on extra horizontal bars spanning between links).
- Links in beam-column junctions: Additional set of links shall be provided at the bottom of the junction and at the top of the junction so that the spacing of links does not exceed 100mm for a distance of 500mm. Links shall also be provided in column within the beam depth at spacing not exceeding 250mm. Closed links may be replaced by u-shaped links in case placement of beam reinforcement presents a problem.
- Any reinforcing bars shall be joggled at a slope not less than 1 in 10.
- Unless noted otherwise noted lap lengths of reinforcing bars shall not be less than 50 times d (where d = diameter of bar)
- Lap lengths for epoxy coated reinforcing bars shall be 60 times d for M30 concrete and 70 times d for M20 concrete.
- Not more than 33% of the main reinforcing bars should be lapped at any section for beams & 50% for columns.
- For two way slabs, the short span reinforcement shall be placed in the outermost layers.
- For flat slabs, the long span reinforcement shall be placed in the outermost layers.
- Reinforcement is represented diagrammatically, it is not necessarily shown in true projection.
- Reinforcing bars shall not be bent after being embedded in concrete.
- Welding of reinforcement is not to be permitted under normal circumstances.
- All the reinforcement shall be clean and free from loose mill-scales, dust, loose rust and coats of paints, oil or

other coatings which may destroy or reduce bond.

(V) Formwork :

- General: The formwork shall conform to the shape, lines and dimensions as shown on the drawings. The timber used for formwork and centering should be of such quality and dimensions so as to, when constructed, remain sufficiently rigid and unyielding during laying, compaction and setting of concrete. It shall be such as not to deform and warp excessively when wetted or not to shrink so as to create large gaps in the joints.
- Ballies used as props shall not be less than 100mm (4") in diameter and shall be in one piece for work up to 4 meters high. All props shall be adequately stiff and they shall be braced laterally in both directions at mid-height. Props shall not rest on loose or made-up ground.
- Forms shall be sufficiently tight to prevent any loss of slurry from concrete.
- Runways for the concreting gangs should not be allowed to rest on reinforcement. Steel chairs should be used to support the runways.
- In normal circumstances, formwork should not be removed before the periods given below:-
  - Walls, columns and vertical sides (faces) of beams or any other structural member … 1 day (24 hours) to 2 day (48 hours) or as directed
  - Slabs spanning:
    - o upto 4.5 meters … 7 days
    - o more than 4.5 meters … 14 days
  - Beams spanning:
    - o upto 6.0 meters … 14 days
    - o more than 6.0 meters … 21 days
- Camber: Forms and scaffolding shall generally be cambered as indicated in the drawings (for beams and slabs of large spans). However, for moderate spans defined as under, camber need not be normally provided:

Span type	Cantilever		S. Supported		Continuous	
Element	Slab	Beam	Slab	Beam	Slab	Beam
Span	1.0m	2.0m	3.5m	5.0m	4.0m	6.0m

- Steel plates and tubular props used for formwork and centering shall be as per the manufacturer's specifications.
- The general arrangement shall comply with following:-
  - All pieces and panels shall be easily removable without causing disturbances to other members.
  - One side of column moulds shall be left open in case of columns higher than 2.5 to 3.0 meters. The open side shall be successively filled in as the concrete is placed and consolidated.
  - Wedges and clamps shall be used wherever practicable in lieu of nails to facilitate Striking.
  - All props shall be supported on double wedges. While removing the props these wedges should be gently eased.
  - Bolts and rods shall be preferably used for ties in case of beams, walls, etc. Wire ties should generally be avoided as far as possible.
- Cleaning and treatment of forms:
  - The inside of formwork shall be coated with nonstaining mineral oil or good quality deshuttering oil or any other approved material. The coating should be as thin as possible and any excess oil should be wiped off before the reinforcement is placed.
  - Temporary opening shall be provided at the base of column boxes and wall forms and other places where necessary to facilitate cleaning and inspection prior to laying of concrete
  - All rubbish, particularly chippings, shavings and sawdust, shall be removed from the interior of the forms before the concrete is placed.
  - All formwork shall be properly designed and erected by the contractors. Preferably the contractors should get the formwork approved by the consulting structural engineers. However, approval of the proposed formwork by the consulting engineers will not diminish the contractors' responsibility for the satisfactory performance of the formwork nor for the safety and co-ordination of all operations at the site.
  - The contractor shall confirm in writing that he has read and understood these specifications and that he shall be fully responsible for its provisions, arrangements, workmanship, care and maintenance till the time of removal of props and formwork.

(VI) Workmanship, Supervision and Testing:

- Mixing: Concrete shall normally be mixed in a mechanical mixer. Mixing shall be continued until there is a uniform distribution of the materials and the mass is uniform in colour and consistency, but in no case shall the mixing be done for less than one and half minutes.
- Adequate care should be taken to ensure that the maximum water-cement ratio does not exceed 0.5
- Placing and Compacting: Concrete shall be placed and compacted before setting commences and should not be subsequently disturbed. The method of placing should be such as to avoid segregation. Concrete shall be thoroughly compacted during the operation of placing and thoroughly

worked around the reinforcement, around embedded fixtures and into corners of the formwork. Concrete compacting should be done with mechanical vibrations using needle vibrators etc. Over-vibrations or vibrations of very wet mixes is harmful and should be avoided.

- When the concrete has not fully hardened, all laitance shall be removed by scrubbing the wet surface with wire or bristle brushes, care being taken to avoid dislodgement of particles of aggregates. The surface shall be thoroughly wetted and all free water removed. The surface shall then be coated with neat cement grout. The first layer of concrete to be well rammed against old work, particular attention being paid to corners and closed spots.
- Supervision: It is exceedingly difficult and costly to alter concrete once placed. Hence, constant and strict supervision of all items of the construction is necessary during the progress of the work, including the proportioning and mixing of the concrete. Supervision is also of extreme importance to check the reinforcement and its placing before being covered.
- Testing of materials: Copies of result of all the testes shall be forwarded to the structural consultants for record and perusal.

- Cement: before casting the concrete, genuineness and quality of cement as to setting time and strength shall be ascertained as per required standards from approved material testing laboratory.
- Concrete: Concrete cubes of size 15 cm x 15 cm x15 cm shall be cast, cured and tested as specified in IS: 456-2000. However, not less than six cubes shall be taken for every concreting operation. Three cubes out of this shall be tested for seven days age and the centering shall be removed only after seven days strength results meet the required standards. The balance three cubes shall be tested for 28 days age.
  - Reinforcement: Testing of reinforcing steel shall be carried out as per the relevant Indian standards as specified in IS: 456-2000 for every lot of steel received.
  - Concrete mix design: Mix design should be carried out periodically at site and at the time of change of source for the concrete ingredients. Mix design should be carried out every month to ensure the consistency of concrete.

(VII) Caution:

- Structural drawings can be executed at site subject to necessary permission from competent authority.
- The responsibility of structural consultant is limited only to accuracy of design calculations. The design is based on data provided by the client. Client should engage independent supervision agency to ensure quality control at site.
- Client should ensure that
  - No addition and alteration is to be carried out without consulting the structural engineer.
  - Structural members should not be damaged / tampered with for any reason what-so-ever during and after construction.
  - Overloading of the structure, changes in the structural system, lack of maintenance, or any act that is detrimental to the safety & stability of the structure as a whole must be avoided during the life time of the structure.

(VIII) Design Loads:

The structure is designed for the following loads:

- Dead Loads:
  - Floor Finish Typ. Floor = 125 kg/Sq.M.
  - Internal Walls 100mm Thk = 300 kg/Sq.M.
  - External Walls 230mm Thk = 540 kg/Sq.M.
  - All toilet walls shall be in lightweight Siporex material
  - False Ceiling Load = 25 kg/Sq.M.
  - Brickbat Coba 150thk. on Terrace = 300 kg/Sq.M.
  - Reinforced Cement Concrete = 2500 kg/Cu.M.
- Live Loads:
  - General floor and passage = 400 kg/Sq.M.
  - Stair = 400 kg/Sq.M.
  - Terrace = 150 kg/Sq.M.
- Wind Loads:
  - Wind Pressure P = 150 kg/Sq.M.
- Earthquake Loads:
  - Earthquake Loads calculated as per IS: 1893-2002.
  - Zone-III, I = 1.0, R = 4, Founding Strata = Hard

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Effect of Aggregates on Quality of Concrete

Prof. R. G. Limaye

Introduction:

Use of higher grade concrete, say M40 and above is becoming more popular in the present time. However, experience has shown that the quality of concrete actually reached is much lower than the required. It is observed that the results of strength on cubes taken in normal course are lower. So non-destructive testing is carried out on the structural members as also cores are taken out. Both these indicate lower results. The actual strength of concrete may be as low as 50% of the characteristic strength required. It is necessary to understand the possible reasons for such low results.

Production of Concrete:

There are three major stages in the production of concrete:

1. **Composition :**
   It depends on mix design and correctly following the mix proportion.
2. **Compaction :**
   It is required to ensure that there are no air voids left in the volume of concrete filling the formwork. The workability of concrete and the means of compaction have to be mutually compatible to achieve this goal. Of course it is very difficult to remove the last traces of air from concrete and that's why a practically fully compacted concrete is defined as one having not more than 1 to 2 % air voids.
3. **Curing :**
   Curing essentially maintains the conditions in concrete conducive for hydration reaction to continue. This practically involves supply of water periodically to compensate for the loss of water from concrete by evaporation.

A deficiency in any of these three stages could result in lower quality of concrete. There are many parameters affecting these three stages and discussion of all these would be too elaborate. In the present paper, only one aspect, namely the effect of moisture in aggregates, is discussed as many times it is not given due attention in the field.

Wet or Dry Aggregates ?

Aggregates are normally stored in open, directly exposed to sunlight. Therefore, they are dry and hot during daytime when most of the concreting work is carried out. It is only when water is spread on the aggregates or during rain that they are in a wet condition. Moisture content in aggregates depends upon atmospheric conditions. It differs from time to time and is also different in different parts of the same stack.

In concrete mix design, it is always advisable to use saturated surface dry aggregates. This is to ensure that neither the aggregates absorb any water from the mix nor do they release additional water to the mix. The mix proportions are then properly defined. In actual practice a suitable correction needs to be applied depending upon the actual state of moisture in the aggregates. If the aggregates are relatively dry, additional water has to be used. If the aggregates are wet, less water needs to be added. The objective in either case is to ensure that the net amount of water in the mix is correct on the basis of Saturated Surface Dry condition (SSD) of aggregates as determined in the mix design. If the aggregates are wet, normally tests are carried out to determine the exact correction required. This is relatively easier. However, if the aggregates are dry, normally no such tests are carried out. As a result, the exact proportion of water in the aggregates is not known. Moreover, dry aggregates are relatively hot leading to higher evaporation of water and increase in temperature of concrete. This makes the correction for water more indeterminate.

Theoretically, aggregates with any state of moisture content could be used provided necessary correction is applied. Proper tests need to be carried out to determine the exact amount of water to be added to any batch of concrete as determined by the mix design. If aggregates are wet, excess water above the SSD condition needs to be reduced from the amount of water for the batch. During the preliminary test on the aggregates prior to mix design, absorption of aggregates is also determined and thus it is available for calculating the necessary correction. Surface Water = Total Water – Absorption in case of wet aggregates. The correction is relatively simpler.

If aggregates are dry, the amount of water for the batch needs to be increased to account for absorption by the aggregates. In case of dry aggregates no tests are normally carried out to determine the moisture content. The aggregates may not be in a perfectly dry condition. A more serious problem is that of higher temperature of dry aggregates, which is many a times overlooked. The resulting higher temperature of concrete increases the rate of evaporation of water from the mix.

In a standard absorption test, the aggregates are directly covered by water whereas in the actual practice, they are covered by cement and water. As a result, the actual rate of absorption is lower than that in a standard test. Workability of concrete thus reduces with time and could be much lower at the actual time of placement of concrete, thereby making compaction more difficult. If absorption and evaporation continue even after the concrete has set, further air voids are formed and the hydration reaction is adversely affected. The quality of resulting concrete is lower than required.

This difficulty could be overcome, to some extent, if the curing process is started immediately after the setting of concrete. Usually there is a time lag even up to 24 hours before the curing is started. The problem becomes more severe in summer when the ambient temperature is on the higher side. All precautions for hot weather concerting are then warranted but hardly implemented in the field. The situation becomes more critical in case of higher grades of concrete wherein the cement content is also on the higher side. Some times mix design is carried out on the basis of dry aggregates. This is not at all desirable.

Concluding remarks:

It is advisable to use wet aggregates in the field applying necessary correction for surface water rather than using dry aggregates, so that the correction is more accurate resulting in better quality of concrete.

Author: Prof. R. G. Limaye retired as a professor of civil engineering from IIT Bombay. He specializes in concrete technology. As president of “Aryan Engineers” he currently practices as an expert in non-destructive testing of structures.

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.

If 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
   For details visit www.icaci.com

READERS' RESPONSE

Words of appreciation

Dear Mr. G. C. Oak,

I read your article "Consulting engineers can guide and lead reconstruction of buildings", published in the ISSE quarterly journal – Jan-Feb-Mar 2007 volume-9-1. Indeed you have presented a unique and interesting case study for reconstruction work. The work can be called as the first of its kind wherein the society members themselves got their buildings reconstructed under the guidance of expert professionals. As a member of ISSE, I am proud to know that you yourself were the structural engineer and project management consultant for the said work. As rightly pointed out by you in the article, this endeavor has resulted in overall economy and effective control over all the activities. I am sure that housing societies planning to take up reconstruction work would be immensely benefited if they follow your path.

Best wishes and warm regards,
Yours sincerely,

K.L. Savla
Consulting Engineer, Mumbai