Name of work: Structure Safety Audit of Existing Ghalib Apartment Co-operative Group Housing Society located at Parwana road Pitampura, Delhi

STRUCTURE AUDIT REPORT

CLIENT: -

The President
Ghalib Apartment
Co-operative Group Housing Societies Pitampura,
Pitampura, Delhi-110083.

CONSULTANT:-

M/S Swati Consultants
504, Sachdeva Corporate Tower, Plot
No. 8, Sector-8, Rohini,
New Delhi-110085.

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1.0 INTRODUCTION

M/s. Swati Consultant has been awarded the work of Carrying out Structural assessment of Ghalib Apartment Co-operative Group Housing Societies Pitampura, Delhi-110083. Non-destructive testing including chemical laboratory tests and concrete core tests have been conducted on existing RCC structure to assess the strength of the existing reinforced cement concrete, quality of concrete, extent of corrosion in the reinforcement, assessment of acid soluble chloride in the concrete and pH value of the existing concrete.

Available Building Information

Available Information	Availability
Architectural drawings	Received on 02nd December 2020 **Status of availability of drawings** 1. Ghalib Apartment
Structural drawings	Not Available
Date of NDT Testing	02nd December 2020 & 03rd December 2020.

Building Data

Information	Description
Structure Type	RCC framed structure.
Foundation Type	Not Available

2.0 SCOPE OF NDT WORK

The scope of work is to conduct following tests on locations identified by M/s Swati Consultants in consultation with the client.

a) Site visit to assess the surface damage of the existing structural elements like beams, columns, slabs, chhajja's and parapet walls by visual inspection & taking photograph of critical structural distress points. The visual inspection has been carried out for the sole aim of identifying the point of structural distress, exposed reinforcement and Spalling of concrete besides mapping of visible cracks in structural elements with sample digital photographs.

b) Detailed presentation of test result and incorporation of suggestion if any in the final report as per direction of Engineer-In-charge.

c) Marking of testing points in drawings for conducting various NDT tests as per scope of work.

d) Conducting following tests

1. Ultrasonic Pulse Velocity Test (UPV)
2. Rebound Hammer Test (RBH)
3. Half Cell Potential Tests (HCP)
4. Cover Meter Test
5. Carbonation Test
6. Determination of Acid soluble Chloride from the concrete mass collected from the specified locations.
7. Determination of pH value of existing concrete.
8. Compressive Strength Test on Concrete Core.
9. Submission of comprehensive report with conclusion and recommendations.

3.0 BRIEF DESCRIPTION OF THE WORK:

Ghalib Apartment Co-operative Group Housing Societies Pitampura, New Delhi- 110083 is Ground + 4 storied RCC framed structure. The structural element like columns, beams and slabs have been identified as having visible cracks or distress at several locations. RCC columns, beams and slabs are provided as structural element to withstand all the possible vertical dead load, live load and to resist earthquake load. Seepage, Blistering, Delamination and Dampness have been observed at several place in the structure.

Ultrasonic Pulse Velocity tests and Rebound hammer tests has been performed in the columns, concrete slab and RCC beams to check the strength and quality of concrete. Based on the results of various tests, the structure was analyzed and the various results are presented in this report. Remedial measures and conclusions are given. Details of various NDT tests and core tests including its procedure and results are presented in Annexure A of the report.

4.0 OBSERVATION BASED ON RAPID VISUAL INSPECTION

Various observation based on site inspection and preliminary evaluation carried out to assess the various structural and services aspects of the structure in relation to its suitability against various safety provision of the NBC of India and related IS codes are briefly given below:

1. The existing building consists of Ground + 4 Storey, RCC framed structure with beams, slabs and columns as structural supporting system.
2. The beams are approximately 230X400 & 200X450 in sizes.
3. The columns are approximately 230X230 in sizes.
4. Dampness has been observed at several places in the building.
5. Due to seepage from service pipe lines, rain water pipe and toilets the rusting has taken place in steel reinforcement, concrete strength got reduced and structure has weakened. Roof treatment done has been found to be damaged and leakage has been observed in top floor slabs and beams.
6. It has been observed that there are Cracks, Blistering, Delamination & Efflorescence, present in the walls.
7. Spalling of concrete has been observed at many places.
8. Algae and fungus also has been observed at outer periphery of the building.
9. The rusting of reinforcement has been observed in some exposed reinforcement.

5.0 RESULTS OF NDT TESTS AND ITS DISCUSSION.

A total of 54 points were tested with UPV/Rebound hammer testing by DIRECT & INDIRECT method in Ghalib Apartment Results are presented in Table 1 & 2. It is observed that UPV readings at 42 locations are in doubtful category and at 1 location it is showing in medium category and at 1 location it is showing in good category and 10 locations were showing

No Readings. At several locations velocity measured is even less than 1 showing very poor quality of concrete. At 10 locations where readings could not be taken shows delamination of concrete and is very dangerous from stability point of view. Over all it means at most of the locations, concrete quality is in very poor condition and large scale honey combing, voids etc. are present. This may be due to continuous degradation of concrete due to atmospheric moisture containing environmental pollutants. Presence of chloride content in concrete and use of saline water during construction may have resulted in the early degradation of concrete. Lack of proper compaction and inadequate quality control resulted in weak and porous concrete. Concrete strength in the locations tested are in the range of M10 grade of RCC as indicated by the results of rebound hammer whereas designed strength of concrete is not available but may be M20 (1:1.5:3) at the time of construction. Core test conducted at 1 location in column, indicates the strength of concrete is 10.4 N/mm^2. At two locations attempts have been made to take core but it was broken as rubbles due to very poor quality of concrete and core could not be taken. The results are given in table. On an average it can be safely estimated that the strength of the existing concrete varies and can be taken as M10 or less for strengthening and design purposes. The low strength is attributed to honeycombing of concrete and aging factor along with various other site factors mentioned above.

Half-cell potential readings were taken at 15 test locations as shown in Table. Values of potential at 2 locations are showing values of potential is more negative than -350 mV, indicating percentage of Probability of Corrosion to rebar in these areas is more than 90% at these locations. Values of potential at 13 location are showing values of potential between -200 to -350 mV, indicating percentage of Probability of Corrosion to rebar in these areas is UNCERTAIN. Values in the table are very near to -350mV showing that its very near to worst conditions and probability is more than 90% in almost all locations. From the results of half-cell potentiometer it can be concluded that corrosion has started in steel reinforcement and reached at the level of severe stage. pH value of concrete as tested are in the range of 6.5 to 11.6. Concrete have a basic pH of about 13, the pH values at exposed surfaces may fall as reactions occur between carbon dioxide from the atmosphere and alkalis in the concrete. This process is known as carbonation. Normally corrosion started in the reinforcement when the pH value reduces to 9.5 or less. This along with the results of half-cell potentiometer verifies the existence of corrosion in the steel and carbonation of concrete. Chloride content in concrete is more as compared to the permissible limit of 0.6kg/cum as per IS 456:2000. The value is even 4 to 5 times higher at some point which is a major cause of deterioration of concrete.

6.0 ANALYSIS OF THE STRUCTURE

Structural analysis of the building have been carried out by taking the material strength as evaluated from the various tests and earthquake parameters as per current BIS provision of IS codes. Details of all the parameters taken and analysis results are presented in the Annexure B of the report.

7.0 CONCLUSION

Based on the various tests and analysis carried out following conclusions are made.

1. The structure has not been designed with Seismic provision and do not conform to various provision of earth quake resistant design codes like IS 1893-2016; IS 4326-2013; IS13920-2016, and various other provision of IS 456-2000 and NBC 2016 applicable at present.
2. Grade of existing RCC can be taken as M10 for all practical purposes and for assessment of strengthening measures.
3. The concrete quality in columns, slabs and beams are poor as per the results of the UPV tests and is below par for structural strengthening.
4. The rusting of reinforcement or corrosion has reached alarming value and cannot be reversed at this stage.
5. Chloride content is very high making the concrete very brittle and weak.
6. The columns sizes are so less and jacketing with micro concrete will require a huge expenditure not viable to be jacketed. Since all columns and beams required jacketing it involves expenditure equivalent to new construction hence strengthening is not advisable.
7. Restoring of existing strength of concrete (M10) is not possible even by injection grouting or jacketing since concrete strength at present is just 60% of that minimum permitted by IS code (M25) required. Even core could not be taken during extraction of core due to very poor and porous concrete.
8. In general the concrete strength in beams, columns and slabs are less and quality of concrete is very poor and needs attention for strengthening and retrofitting. Injection

grouting of existing concrete members like slabs, beams and columns is to be done to improve its performance and remove the honey combing and porosity to some extent only for the time being.

9. The structure has been analyzed on Etabs software and the results are presented in Annexure B. From the results it is concluded that strength capacity of structural members such as beams and columns were found to be very less. Most of the safety criteria as per codal requirements have not been satisfied in the design results. The structure is not able to take the requisite loads due to dead and live load including earthquake loads.
10. Friction damper/additional RCC shear walls are required to be provided to meet the various provisions of present seismic codal requirements.

Therefore it is concluded that Building Blocks in Tower A, B, C, D and E Totaling 54 Blocks are unsafe for long term use in earthquake zone IV. Extensive seismic strengthening and retrofitting are required to make the building habitable for few years only by improving the existing strength to some extent. Due to very poor strength of RCC (M10) grade, very less sizes of columns (230x230) and very less percentage of steel in columns and beams it is strongly recommended to demolish the buildings blocks one by one and reconstruct it with the latest provisions of seismic code and new technology under the supervision of structural engineer and a team of qualified engineers. Some of the repair and Restoration methodology is given below for strengthening the structure to some extent to avoid any major accidents.

8.0 Repair and Restoration methodology

Based on the above conclusions following remedial methodology are suggested. Recommendations have been made to address the observed deficiencies in various components of the structure.

8.1 Repair Category-RC-I: Repair Category-RC-II: Restoration of Integrity and Strength of Concrete

It is recommended to undertake epoxy injection grouting for the repair and restoration of RCC structural elements identified to have inadequate in-situ concrete integrity and strength.

Therefore all those structural elements where inadequate in-situ strength of concrete or unsatisfactory UPV value was noted shall be chosen for grouting. It is believed that presence of voids and pores in the concrete have resulted in unsatisfactory homogeneity, integrity and strength of concrete in these members. Grouting shall restore the Integrity and homogeneity of concrete which shall also ultimately result in enhancement in strength of concrete.

It is recommended to undertake grouting using ultra low viscous, solvent-free, high strength epoxy resin. The epoxy grout shall conform to the relevant recommendations of ASTM C 881 [9]. Mechanical packers with non-return valve are recommended to be used for grouting. Manufacturer's recommendations shall be followed with respect to application procedure for various other aspects of grouting. It shall be ensured that the entire volume of concrete in the member under consideration is grouted to the extent possible. The repaired RC elements shall be tested using ultrasonic pulse velocity apparatus in order to verify the improvement in integrity and strength of concrete. While the integrity and homogeneity of concrete shall be considered as satisfactory if a UPV value of more than 3.5 km/s is obtained. The above mentioned minimum threshold UPV values for meeting in-situ strength of concrete were computed from the strength correlations for beams and columns discussed in previous sections. It is also recommended that the repair executing agency shall undertake UPV testing of all the remaining RC beams and columns in both the buildings and the members found to be deficient with respect to the above mentioned UPV criteria shall be epoxy grouted.

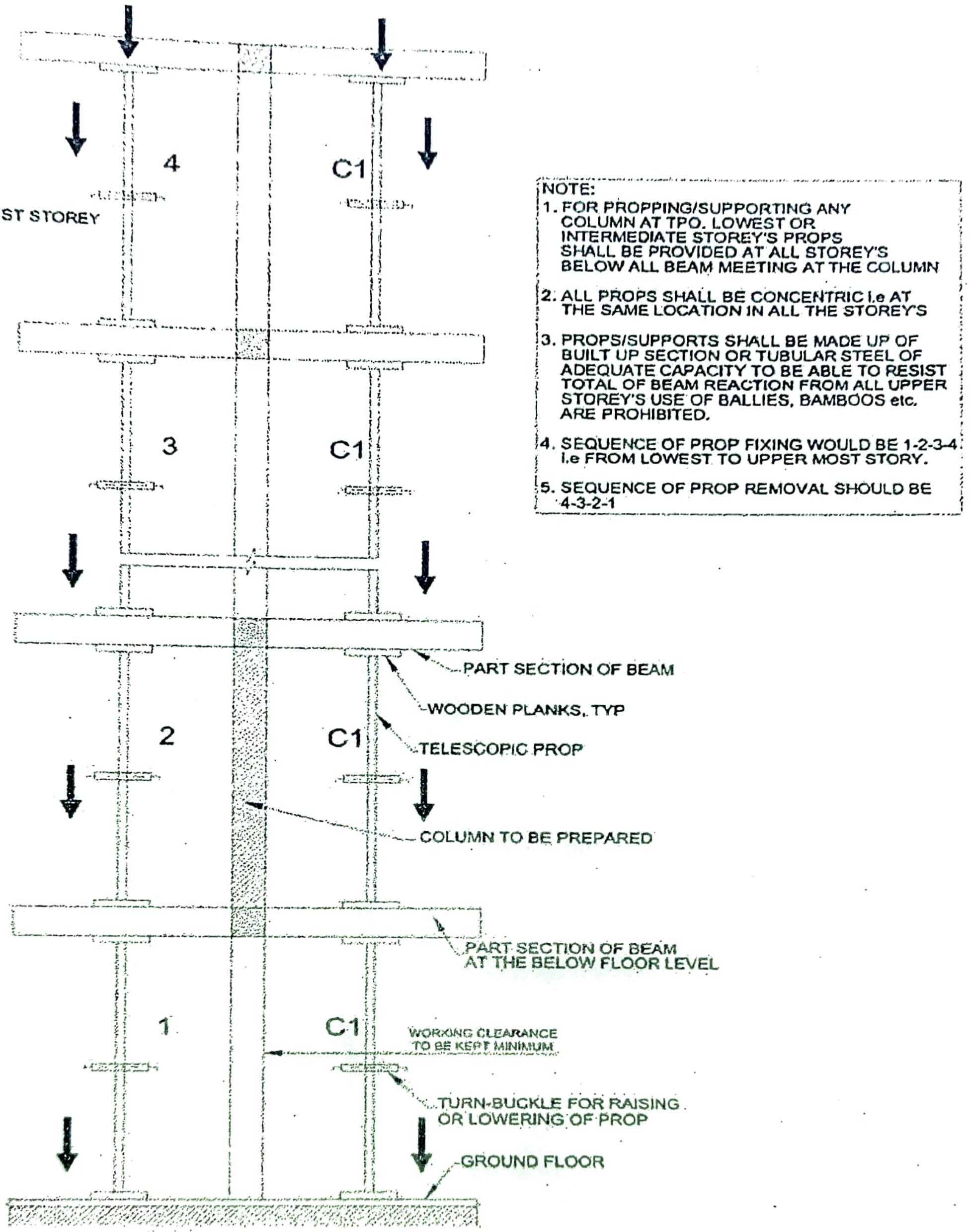
8.2 Repair Category-RC-III: Jacketing of columns and beams with micro concrete along with addition of reinforcement.

The structural elements which don't have significant visible damage but the structural analysis based on NDT testing show significant reduction in strength of column capacity check as per IS 1893-2016 and IS 13920-2016. Under this Repair Category-RC-II of repairs the procedure given as under-mentioned steps shall be followed.

1. Before attempting any repair, it shall be ensured that the live loads in the building, expected to be transferred to the member being repaired, shall be the minimum possible. Wherever necessary, temporary props and supports using structural steel sections shall be provided to unload the member being repaired. Generally where the thickness of

concrete removal exceeds the 50 mm on each of the two opposite of a member and section reduction in reinforcement is found due to corrosion, temporary unloading with propping is required before attempting any repair. Typical arrangement of unloading a beam by propping the slab on its either side is schematically illustrated in Fig It may be noted that a continuous transfer of load from the level of repair to ground floor or to columns below shall be ensured by placing the props concentrically at every level. On the ground floor the lowest prop shall be supported on firm ground. A local temporary R.C.C base shall be created and the lowest prop shall be firmly attached to the base. Remove plaster and finished all around the columns, and beams to be repaired. In columns, the plaster and the finishes shall be removed from all its four exposed faces along its entire height. Thereafter, using electric/ pneumatic chippers or any other concrete demolishing toll gently remove cover concrete over the entire affected portion of the structure member under consideration, to expose (corroded) steel reinforcement. It shall be ensured that the removal of cover concrete shall be done to such an extent that the longitudinal reinforcement in the columns stands exposed. It may be noted that in the case of columns, the cover concrete shall be removed from all the four vertical faces. Further, care shall be taken that the concrete shall also be removed from the inside/underside of the bars by under-cutting to a depth of 20 mm below/inside the (longitudinal) reinforcement bars. It shall also be ensured that concrete is completely stripped-off around the full circumference over the full length of the corroded bars, where ever encountered.

1. Ensure that where ever encountered, deteriorated or loosely attached concrete is removed from the member in question till the sound substrate is exposed. It is advised to remove the concrete from the entire height of length of the member in a given storey even if the distress is noticed only a patch. This precaution is advised because research show that patch repair of corroded R.C.C. remains unsuccessful due to formation of new anodes around the patched concrete and therefore repair of full height/length should be undertaken.



NOTE:

1. FOR PROPPING/SUPPORTING ANY COLUMN AT TPO. LOWEST OR INTERMEDIATE STOREY'S PROPS SHALL BE PROVIDED AT ALL STOREY'S BELOW ALL BEAM MEETING AT THE COLUMN
2. ALL PROPS SHALL BE CONCENTRIC I.e AT THE SAME LOCATION IN ALL THE STOREY'S
3. PROPS/SUPPORTS SHALL BE MADE UP OF BUILT UP SECTION OR TUBULAR STEEL OF ADEQUATE CAPACITY TO BE ABLE TO RESIST TOTAL OF BEAM REACTION FROM ALL UPPER STOREY'S USE OF BALLIES, BAMBOOS etc. ARE PROHIBITED.
4. SEQUENCE OF PROP FIXING WOULD BE 1-2-3-4 I.e FROM LOWEST TO UPPER MOST STORY.
5. SEQUENCE OF PROP REMOVAL SHOULD BE 4-3-2-1

(Fig. 1 Typical arrangement of propping for relieving a column of its axial load)

2. Clean the exposed concrete substrate by using wire brushes and/or sand blasting. Water jet may also be used. Remove rust (if any) from all around the surface along the length of the corroded steel reinforcement, using hand tools like chisel, hammers, wire brushes, abrading cloth/paper etc. remove the rust by sand blasting if directed by engineer-in-charge, due to unsatisfactory results of manual rust removal.

3. Upon removal of the cover concrete and rust from the corroded rebars/longitudinal bars, in the columns (If and where ever encountered) shall be inspected for loss in their section. Both longitudinal and lateral steel shall be inspected. If section reduction is less than 25% for one corroded bar and 20% for two more bars, no repair of reinforcement is required. If section reduction is found to be more than above mentioned limits then the member shall be repaired with addition of longitudinal bars as per drawings attached with this report. The additional reinforcement shall be rebar with HILTI chemical and lateral ties shall be provided before concreting.

   A structural drawings showing the methodology is attached with this report.

9.0 CLOSURE

We appreciate the opportunity to perform this investigation for you and have pleasure in submitting this report. Please contact us when we can be of further service to you.

Sadanand Ojha
04/01/2021

Dr. Sadanand Ojha.

MCD LIC NO: SE/0352

ANNEXURE-A

DETAILS OF NON DESTRUCTIVE AND CORE TESTS

NON-DESTRUCTIVE TESTING

A.0 VARIOUS NDT TEST METHODS

A.1 ULTRASONIC PULSE VELOCITY (UPV) TEST

The test involves measurement of transit time of an ultrasonic pulse generated through the emitter and measured by the collector or by receiver. Since the thickness of the member is known or can be measured, the pulse velocity can be calculated from the simple formula V=L/T.

The test involves measurement of pulse velocity by either direct or semi-direct or indirect (surface) transmission method as mentioned in IS 13311 (Part 1): 1992(Reaffirmed 2013). Direct method shall be used as far as possible, since that gives best results. Wherever direct method is not possible due to very high member thickness the semi-direct or indirect methods of testing shall be used. The locations at which test is desired shall be identified by the engineer in-charge before the test, so as to prepare the surface for the test. It is desirable that the transducers shall be directly in contact with concrete whose surface is clean and free from moisture and dust. The reliability of the results might be affected significantly in case readings are taken from plaster and/or rough surfaces. The thickness of the member or the length of shortest direct path within concrete shall be measured before start of the test. Any coupling agent like grease, petroleum jelly etc., shall be applied to the transducers and test surface to remove any entrapped air. The transducers shall then be placed against concrete in a direction depending on mode of transmission. The transit time shall be recorded onto the equipment. Repeated readings of the transit time shall be made until a minimum value is obtained and this should be the recorded value. Based on this, the ultrasonic velocity shall be calculated as described above and is presented in Table.

A.1.1INTERPRETATION OF RESULTS

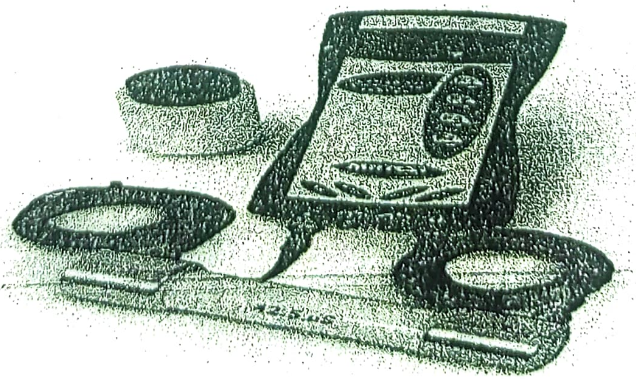
The interpretation of results has been done on the basis IS: 13311 (Part 1) (Reaffirmed 2013) the standard table of which Is reproduced below. The results are tabulated In Table. The method of testing is also mentioned along with specific remarks if any for a particular location. The pulse velocity through concrete usually gives a good indication of its quality. The pulse velocity is also affected by surface preparation, moisture content, temperature, percentage reinforcement etc., among other known factors.

TABLE 2A OF IS: 13311 (Part 1): 1992

PULSE VELOCITY (km/sec)	CONCRETE QUALITY GRADING
Above 4.5 km/sec	**Excellent**
3.5 - 4.5 km/sec	**Good**
3.0 - 3.5 km/sec	**Medium**
Below 3.0 km/sec	**Doubtful**

A.1.2 TEST LIMITATIONS

Test limitations for UPV testing are mentioned in IS 13311 (Part 1): 1992 (Reaffirmed 2013). To mention it briefly, the results are affected by age of concrete, percentage of reinforcement, method of testing, concrete composition etc. It is to be noted that if correlation is made between UPV values & compressive strength of concrete, then estimated strength may vary from actual strength by +/- 20 percent. Surface probing (Indirect method of UPV) in general gives lower pulse velocity than in case of cross probing (Direct method of UPV) and also depending on number of parameters, the difference could be of the order of about 1 km/sec Velocity as per IS: 13311 Part 1. The results are specific to tested locations only due to heterogenic property of concrete.



UPV APPARATUS

ULTRASONIC PULSE VELOCITY -TEST DATA

TABLE-1 (Ghalib Apartment)

Sr. No.	Test Location		Type of Method	Pulse Velocity (km/s)	Quality Of Concrete
	Location	Mark ID			
1	Tower A Stilt	AC/2	Indirect	0.3	Doubtful
2	Tower A Stilt	AC/5	Direct	2.4	Doubtful
3	Tower A Stilt	AC/7	Direct	2.4	Doubtful
4	Tower A Stilt	AC/8	Direct	0.4	Doubtful
5	Tower A Stilt	AC/9	Direct	-	-
6	Tower A Stilt	AC/12	Direct	-	-
7	Tower A Stilt	AB/1	Direct	-	-
8	Tower A Stilt	AB/3	Direct	-	-
9	Tower A Stilt	AB/4	Direct	-	-
10	Tower A Stilt	AB/13	Direct	-	-
11	Tower A Stilt	AS/6	Indirect	0.4	Doubtful
12	Tower A Stilt	AS/10	Indirect	0.4	Doubtful
13	Tower A Stilt	AC/11	Indirect	0.6	Doubtful
14	Tower B Stilt	BC/4	Indirect	0.6	Doubtful
15	Tower B Stilt	BC/7	Indirect	-	-
16	Tower B Stilt	BC/11	Indirect	3.9	Good

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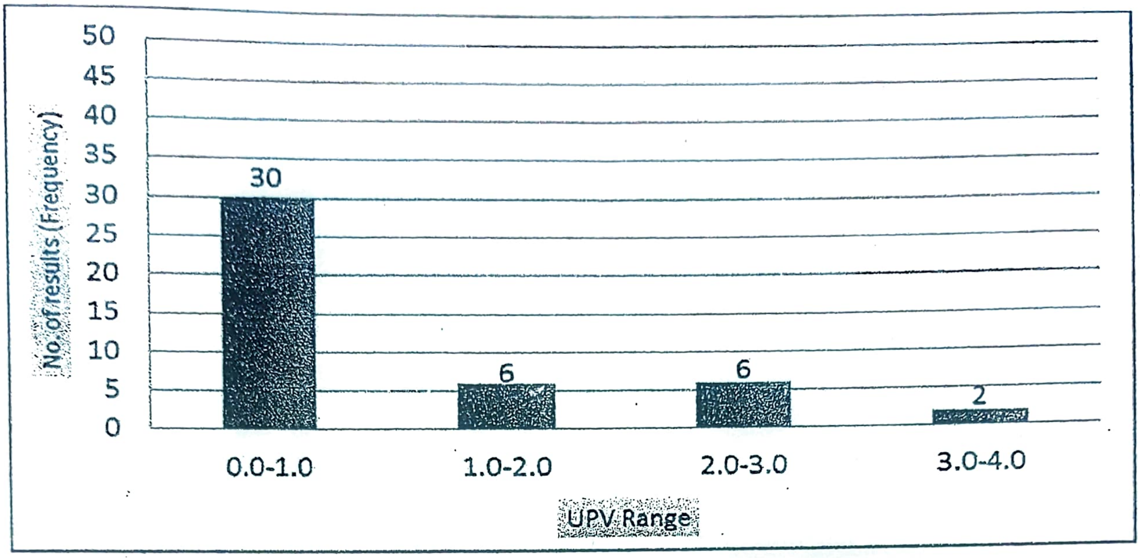
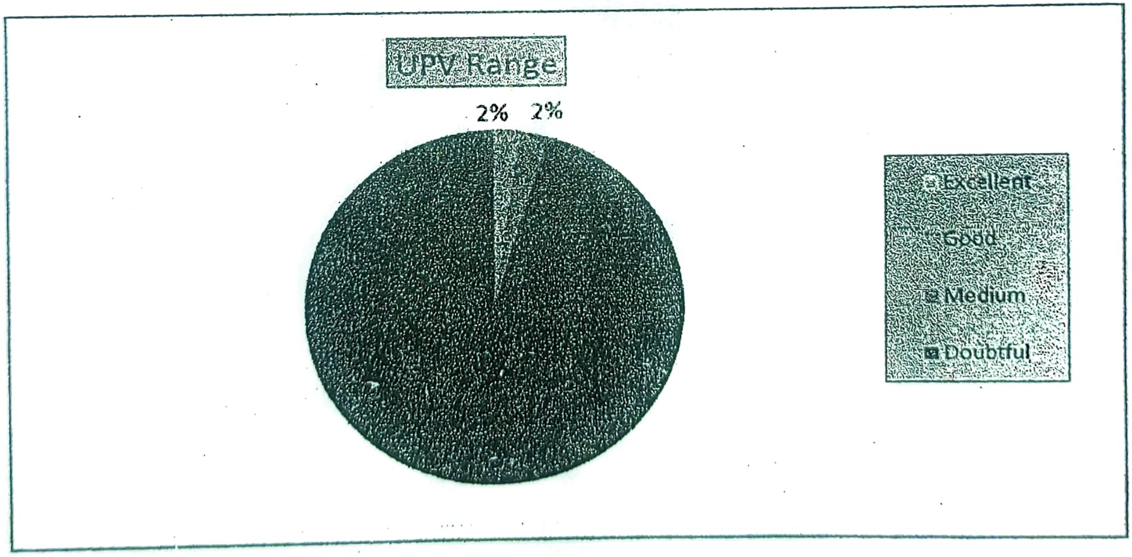
Sr. No.	Test Location		Type of Method	Pulse Velocity (km/s)	Quality Of Concrete
	Location	Mark ID			
17	Tower B Stilt	BC/12	Direct	0.9	Doubtful
18	Tower B Stilt	BC/13	Direct	3.1	Medium
19	Tower B Stilt	BB/1	Indirect	0.4	Doubtful
20	Tower B Stilt	BB/3	Indirect	0.2	Doubtful
21	Tower B Stilt	BB/8	Indirect	-	-
22	Tower B Stilt	BB/10	Indirect	-	-
23	Tower B Stilt	BS/2	Indirect	0.4	Doubtful
24	Tower B Stilt	BS/5	Indirect	0.5	Doubtful
25	Tower B Stilt	BS/6	Indirect	2.2	Doubtful
26	Tower B Stilt	BS/9	Indirect	0.4	Doubtful
27	Tower C Stilt	CC/1	Direct	1.4	Doubtful
28	Tower C Stilt	CC/4	Direct	0.5	Doubtful
29	Tower C Stilt	CC/6	Indirect	2.4	Doubtful
30	Tower C Stilt	CB/3	Direct	1.1	Doubtful
31	Tower C Stilt	CB/5	Direct	1.1	Doubtful
32	Tower C Stilt	CS/2	Indirect	0.5	Doubtful
33	Tower C Stilt	CS/7	Indirect	0.4	Doubtful
34	Tower D Stilt	DC/2	Indirect	0.3	Doubtful
35	Tower D Stilt	DC/6	Indirect	0.4	Doubtful

Sr. No.	Test Location		Type of Method	Pulse Velocity (km/s)	Quality Of Concrete
	Location	Mark ID			
36	Tower D Stilt	DC/8	Indirect	1.0	Doubtful
37	Tower D Stilt	DC/9	Indirect	0.4	Doubtful
38	Tower D Stilt	DB/1	Direct	-	-
39	Tower D Stilt	DB/4	Direct	0.6	Doubtful
40	Tower D Stilt	DB/7	Direct	0.8	Doubtful
41	Tower D Stilt	DS/3	Indirect	0.4	Doubtful
42	Tower D Stilt	DS/5	Indirect	0.4	Doubtful
43	Tower E Stilt	EC/7	Semi-Direct	1.7	Doubtful
44	Tower E Stilt	EC/9	Indirect	0.5	Doubtful
45	Tower E Stilt	EB/1	Direct	0.4	Doubtful
46	Tower E Stilt	EB/2	Direct	0.4	Doubtful
47	Tower E Stilt	EB/3	Direct	2.4	Doubtful
48	Tower E Stilt	EB/5	Direct	1.1	Doubtful
49	Tower E Stilt	EB/6	Direct	0.8	Doubtful
50	Tower E Stilt	EB/8	Direct	1.6	Doubtful
51	Tower E Stilt	EB/11	Direct	2.4	Doubtful
52	Tower E Stilt	ES/4	Indirect	0.9	Doubtful
53	Tower E Stilt	ES/10	Indirect	0.9	Doubtful
54	Tower E Stilt	ES/12	Indirect	0.7	Doubtful

HISTOGRAM PLOT OF UPV VALUES FOR GHALIB APARTMENTS



PIE CHART RESULT DISTRIBUTION OF UPV RESULTS FOR GHALIB APARTMENTS



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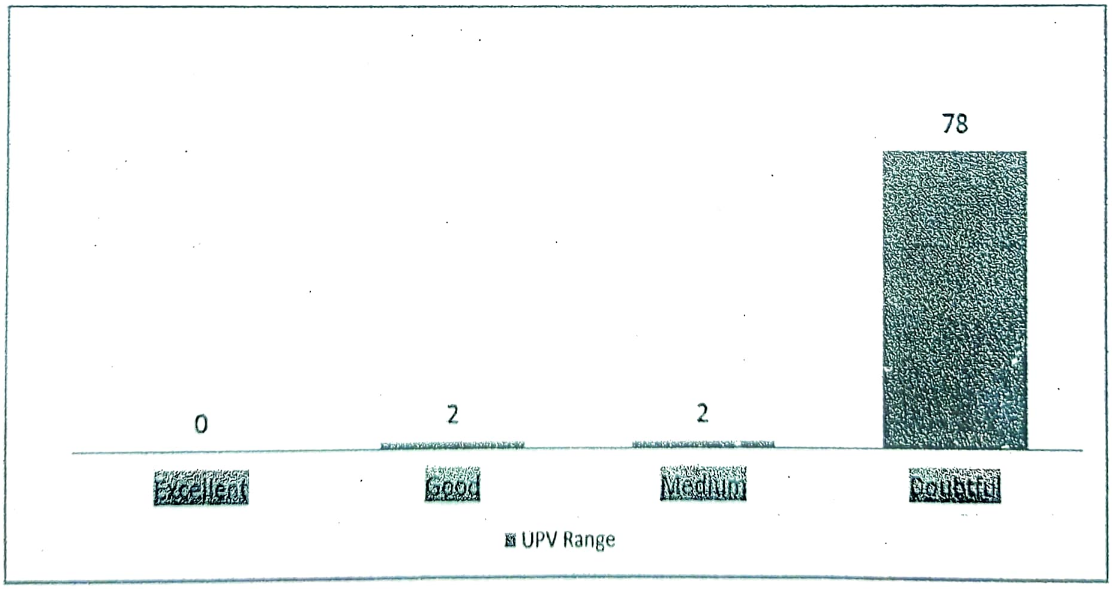
STATISTICAL ANALYSIS OF UPV TEST RESULTS FOR GHALIB APARTMENTS

PULSE VELOCITY (km/sec)	CONCRETE QUALITY GRADING
Above 4.5 km/sec	Excellent
3.5 - 4.5 km/sec	Good
3.0 - 3.5 km/sec	Medium
Below 3.0 km/sec	Doubtful



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A.2 REBOUND HAMMER TEST

The basic principal of Rebound hammer working is that when the plunger of rebound hammer is pressed against the surface of the concrete, the spring to control mass rebounds and the extent of such rebound depends upon the surface hardness of concrete. The surface hardness and therefore the rebound indices taken shall be related to the compressive strength of the concrete. The rebound distance is measured along a graduated scale and is designated as the rebound number or rebound Index. The test involves measurement of rebound number or rebound Index by placing rebound hammer at right angles to the surface of the concrete member as mentioned in IS 13311 (Part 2): 1992 (Reaffirmed 2013).

The test shall be conducted around all the points of observation on all accessible faces of the structural element. The point of impact shall be at least 20mm away from any edge or sharp discontinuity. The locations at which test is desired shall be identified by the engineer in-charge before the test, so as to prepare the surface for the test. For testing the selected surface shall be smooth and dry. Concrete surfaces shall be thoroughly cleaned before taking any measurement.

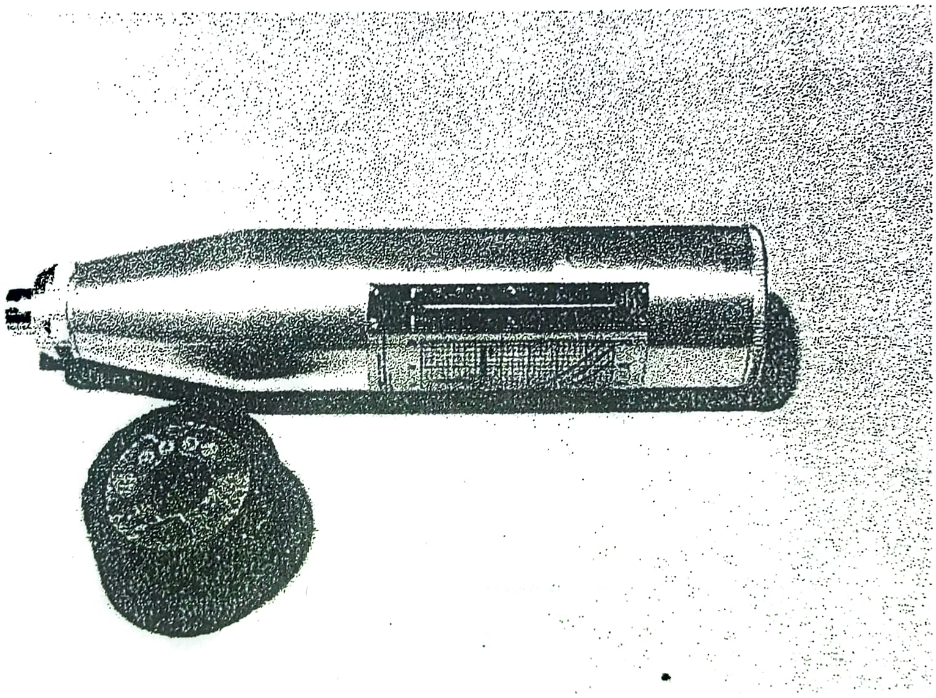
The reliability of the results might be affected significantly in case readings are taken from plaster and/or rough surfaces. The rebound number is affected by factors like type of cement and aggregate, surface preparation, moisture content, age of concrete and extent of carbonation of concrete. As per IS: 13311 (Part 2): 1992 the rebound indices are indicative of compressive strength of concrete to a limited depth from the surface.

A.2.1 INTERPRETATION OF RESULTS

The interpretation of results is evolved with reference to IS: 13311 (Part 2): 1992. The rebound hammer method provides a convenient and rapid indication of the compressive strength of concrete by means of established correlation between the rebound Index and the compressive strength of the concrete.

A.2.2 TEST LIMITATIONS

The test preliminary responds on surface hardness and is an indirect method to assess the strength of concrete. Factors that influence the readings as mentioned in IS: 13311 (Part-2), 1992 (Reaffirmed 2013) are affected by localized hardness, carbonation of concrete, surface smoothness, type of cement, type of the aggregates, moisture content of the concrete, concrete, age of concrete, concrete composition etc. The test results can vary up to +/–25 % as a limitation of this method of testing. The results are specific to tested locations only due to heterogenic property of concrete.



REBOUND HAMMER APPARATUS

REBOUND HAMMER –TEST DATA

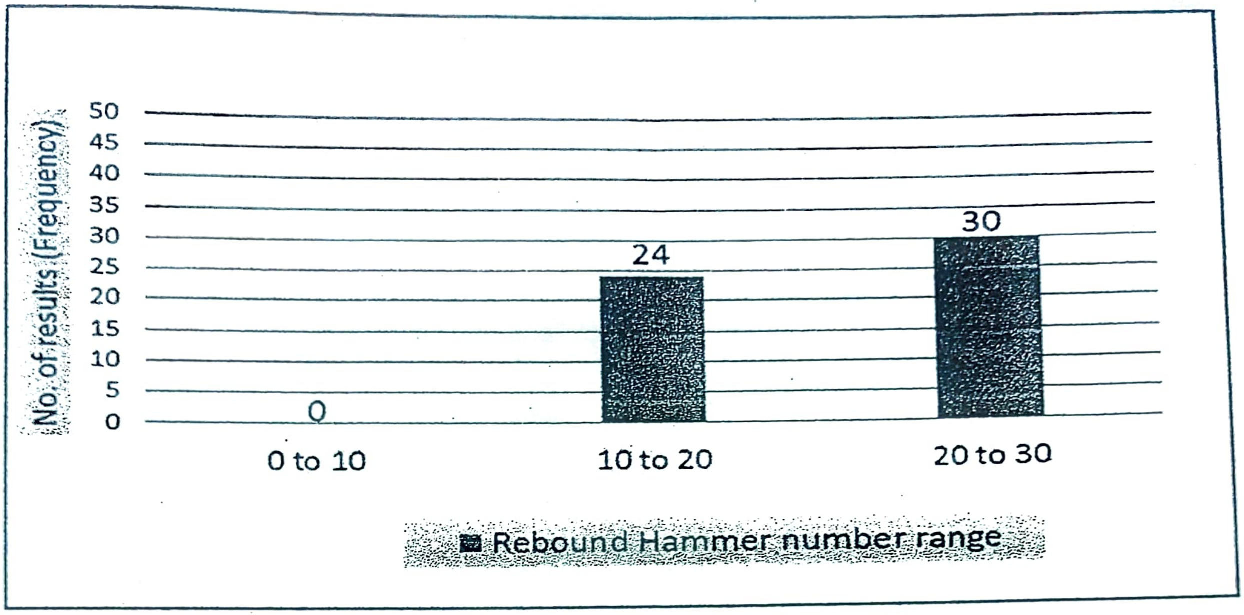
TABLE-2 (Ghalib Apartment)

Sr. No.	Test Location		Hammer Alignment	Avg. Rebound No.	Equiv. Comp. Strength (N/mm^2)	Design Strength
	Location	Mark ID				
1	Tower A Stilt	AC/2	Horizontal	17	10.0	Not Available
2	Tower A Stilt	AC/5	Horizontal	24	14.8	
3	Tower A Stilt	AC/7	Horizontal	21	11.2	
4	Tower A Stilt	AC/8	Horizontal	25	16.0	
5	Tower A Stilt	AC/9	Horizontal	15	10.0	
6	Tower A Stilt	AC/12	Horizontal	20	10.0	
7	Tower A Stilt	AB/1	Horizontal	19	10.0	
8	Tower A Stilt	AB/3	Horizontal	19	10.0	
9	Tower A Stilt	AB/4	Horizontal	18	10.0	
10	Tower A Stilt	AB/13	Horizontal	19	10.0	
11	Tower A Stilt	AS/6	Vertical	23	10.0	
12	Tower A Stilt	AS/10	Vertical	24	10.0	
13	Tower A Stilt	AS/11	Vertical	22	10.0	
14	Tower B Stilt	BC/4	Horizontal	14	10.0	
15	Tower B Stilt	BC/7	Horizontal	14	10.0	
16	Tower B Stilt	BC/11	Horizontal	25	16.0	
17	Tower B Stilt	BC/12	Horizontal	22	12.4	
18	Tower B Stilt	BC/13	Horizontal	25	16.0	
19	Tower B Stilt	BB/1	Horizontal	21	11.2	
20	Tower B Stilt	BB/3	Horizontal	22	12.4	

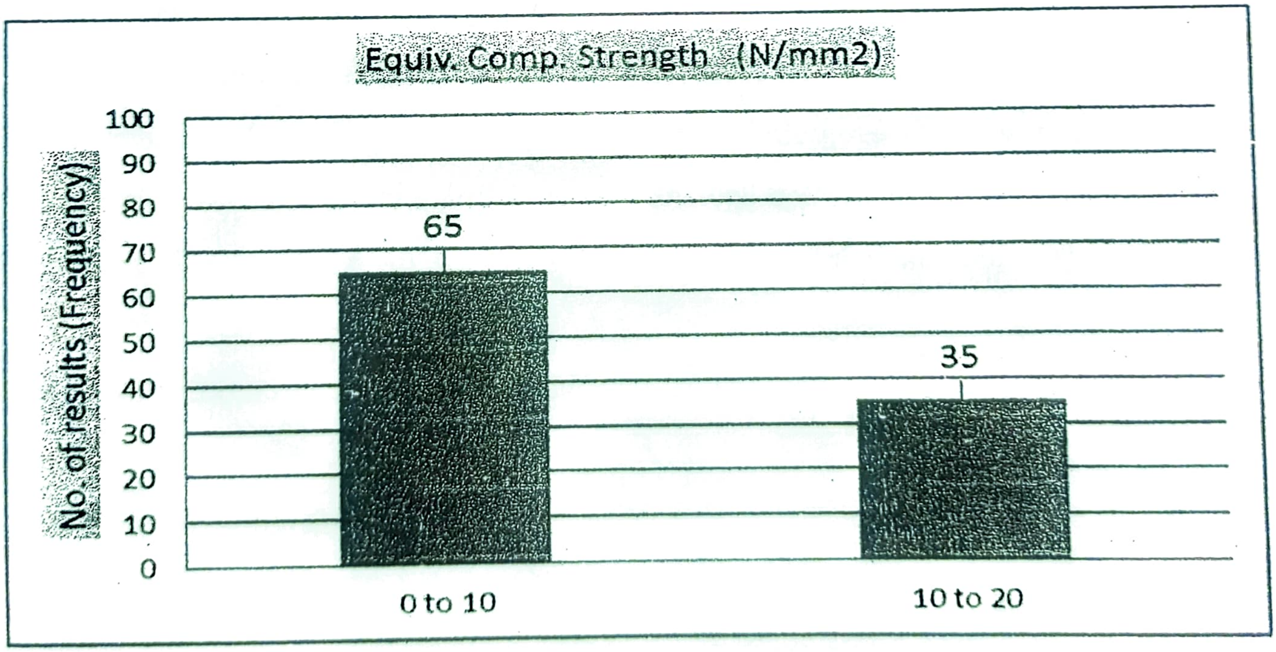
Sr. No.	Test Location		Hammer Alignment	Avg. Rebound No.	Equiv. Comp. Strength (N/mm²)	Design Strength
	Location	Mark ID				
21	Tower B Stilt	BB/8	Horizontal	26	17.5	Not Available
22	Tower B Stilt	BB/10	Horizontal	18	10.0	
23	Tower B Stilt	BS/2	Vertical	23	10.0	
24	Tower B Stilt	BS/5	Vertical	17	10.0	
25	Tower B Stilt	BS/6	Vertical	23	10.0	
26	Tower B Stilt	BS/9	Vertical	22	10.0	
27	Tower C Stilt	CC/1	Horizontal	22	12.4	
28	Tower C Stilt	CC/4	Horizontal	22	12.4	
29	Tower C Stilt	CC/6	Horizontal	24	14.8	
30	Tower C Stilt	CB/3	Horizontal	19	10.0	
31	Tower C Stilt	CB/5	Horizontal	17	10.0	
32	Tower C Stilt	CS/2	Vertical	28	13.6	
33	Tower C Stilt	CS/7	Vertical	25	10.0	
34	Tower D Stilt	DC/2	Horizontal	19	10.0	
35	Tower D Stilt	DC/6	Horizontal	13	10.0	
36	Tower D Stilt	DC/8	Horizontal	12	10.0	
37	Tower D Stilt	DC/9	Horizontal	17	10.0	
38	Tower D Stilt	DB/1	Horizontal	14	10.0	
39	Tower D Stilt	DB/4	Horizontal	20	10.0	
40	Tower D Stilt	DB/7	Horizontal	22	12.4	

Sr. No.	Test Location		Hammer Alignment	Avg. Rebound No.	Equiv. Comp. Strength (N/mm^2)	Design Strength
	Location	Mark ID				
41	Tower D Stilt	DS/3	Vertical	22	10.0	Not Available
42	Tower D Stilt	DS/5	Vertical	21	10.0	
43	Tower E Stilt	EC/7	Horizontal	20	10.0	
44	Tower E Stilt	EC/9	Horizontal	16	10.0	
45	Tower E Stilt	EB/1	Horizontal	20	10.0	
46	Tower E Stilt	EB/2	Horizontal	17	10.0	
47	Tower E Stilt	EB/3	Horizontal	25	16.0	
48	Tower E Stilt	EB/5	Horizontal	22	12.4	
49	Tower E Stilt	EB/6	Horizontal	23	13.6	
50	Tower E Stilt	EB/8	Horizontal	27	19.0	
51	Tower E Stilt	EB/11	Horizontal	22	12.4	
52	Tower E Stilt	ES/4	Vertical	21	10.0	
53	Tower E Stilt	ES/10	Vertical	26	10.0	
54	Tower E Stilt	ES/12	Vertical	19	10.0	

HISTOGRAM PLOT OF REBOUND NO FOR GHALIB APARTMENTS



HISTOGRAM PLOT OF EQUIV. COMP. STRENGTH FROM RBH TEST FOR GHALIB APARTMENTS



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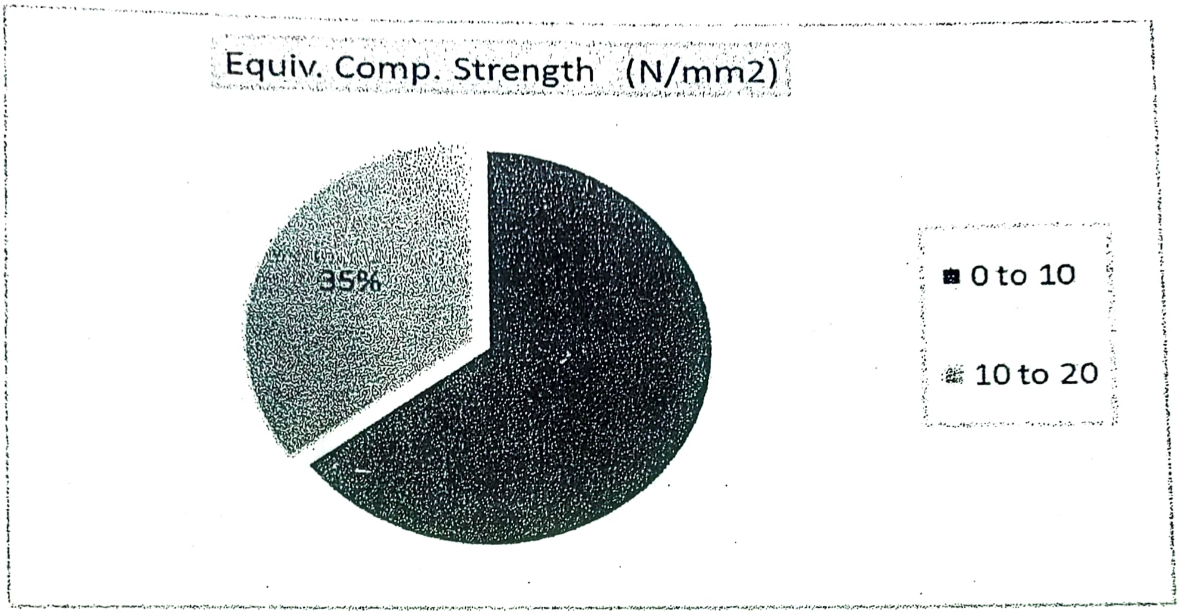
PIE CHART RESULT DISTRIBUTION OF EQUIV. COMP. STRENGTH FROM RBH TEST FOR GHALIB APARTMENTS



A.3 HALF-CELL POTENTIAL TESTS

Corrosion being an electrochemical phenomenon, the electrode potential of steel rebar with reference to a standard electrode undergoes changes depending on corrosion activity. A systematic survey on well-defined grid points gives useful information on the presence of probability of corrosion activity. The common standard electrodes used are

Copper – Copper Sulphate Electrode (CSE- Cu/Cuso4)

Silver – Silver Chloride Electrode (SSE- Ag/Agcl)

Standard Calomel Electrode (SCE- Hg/Hg2Cl2)

The measurement consists of giving an electrical connection to the rebar and observing the voltage difference between the rebar and a reference electrode in contact with concrete surface. Generally the potential values become more and more negative as the corrosion becomes more and more active. However, less negative potential values may also indicate the presence of corrosion activity, if the pH value of concrete is less. This method covers the estimation of electrical half-cell potential of reinforcing steel in concrete, for the purpose of determining the corrosion activity of the reinforcing steel.

A copper – copper sulphate half-cell is used in this test. It consists of a rigid tube of a dielectric material that is non-reactive with copper or copper sulphate, a porous wooden plug that remains wet by capillary reaction, and a copper rod that is immersed within the tube in a saturated solution of copper sulphate. The solution shall be prepared with reagent grade copper sulphate crystals dissolved in distilled water. The solution may be considered saturated when an excess of crystals (un-dissolved) lies at the bottom of the solution.

Half-Cell potential testing is to be done on columns / beams / slabs / any concrete element by marking locations randomly wherever there may be a possibility of corrosion. The location needs to be pre-wetted before taking the half-cell potential readings. A direct electrical connection needs to be induced to the reinforcing steel by means of a compression type clamp. To ensure a low electrical resistance the rebar needs to be cleaned by wire brush.

Then the positive terminal of the voltmeter to be connected to the rebar whereas the other terminal is connected to the half-cell probe filled with copper sulphate solution. The half-cell shall be placed on the concrete surface by keeping wet sponge for electrical continuity. The voltmeter readings shall be observed till not fluctuations are observed. The half-cell potential voltage readings are noted when it is stable and the test then similarly repeated at other locations of structure.

A.3.1INTERPRETATION OF RESULTS

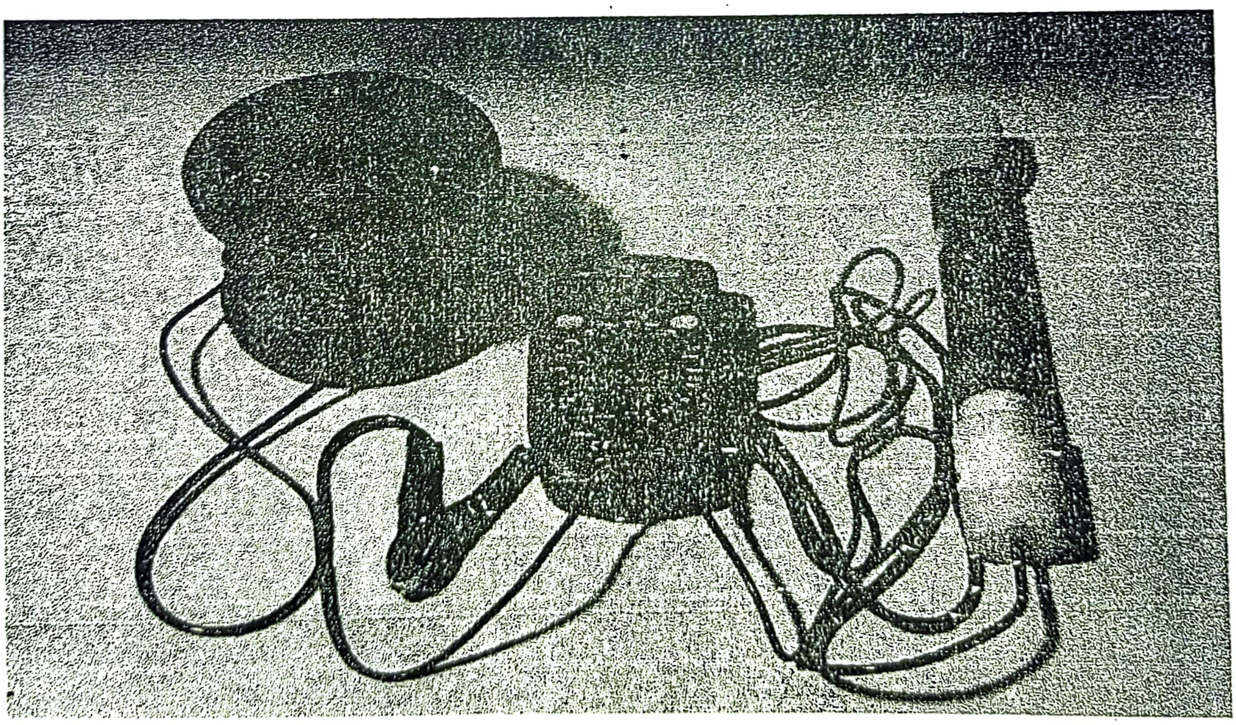
The general guideline of ASTM-C-876-15 has been published on this method, which indicates the potential ranges in relation to the probability of corrosion as given below. The method is widely used and being recognized to be useful with the availability of more and more authoritative reports and data.

% of Probability of Corrosion	**E_{corr} (Vs Cu/CuSo4)**
Greater than 90%	More negative than - 350 mV
Corrosion in this area is uncertain	between – 200 and - 350 mV
Less than 10%	More positive than - 200 mV

Generally, the probability or presence of corrosion is assessed if the potential values are more negative than "-250mV". This guideline is more applicable only when the corrosion is due to chloride contamination.

A.3.2 TEST LIMITATION:

This method cannot indicate the actual corrosion rate & quantum of rebar corroded. It requires a small hole to be drilled to concrete structure to enable electrical contact to be made with the rebar in the concrete structure which is to be tested.The interpretations of the results are considering other limitations of the method such as the effect of protective or decorative coatings applied to the concrete.



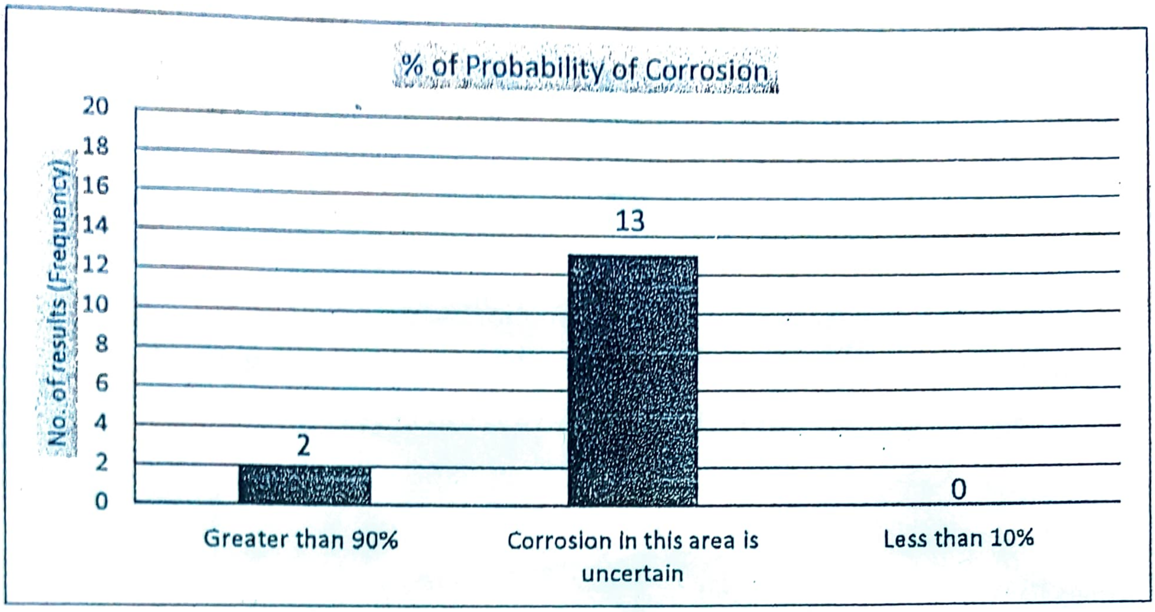
HALF CELL POTENTIOMETER APPARATUS

TABLE-3 (Ghalib Apartment)

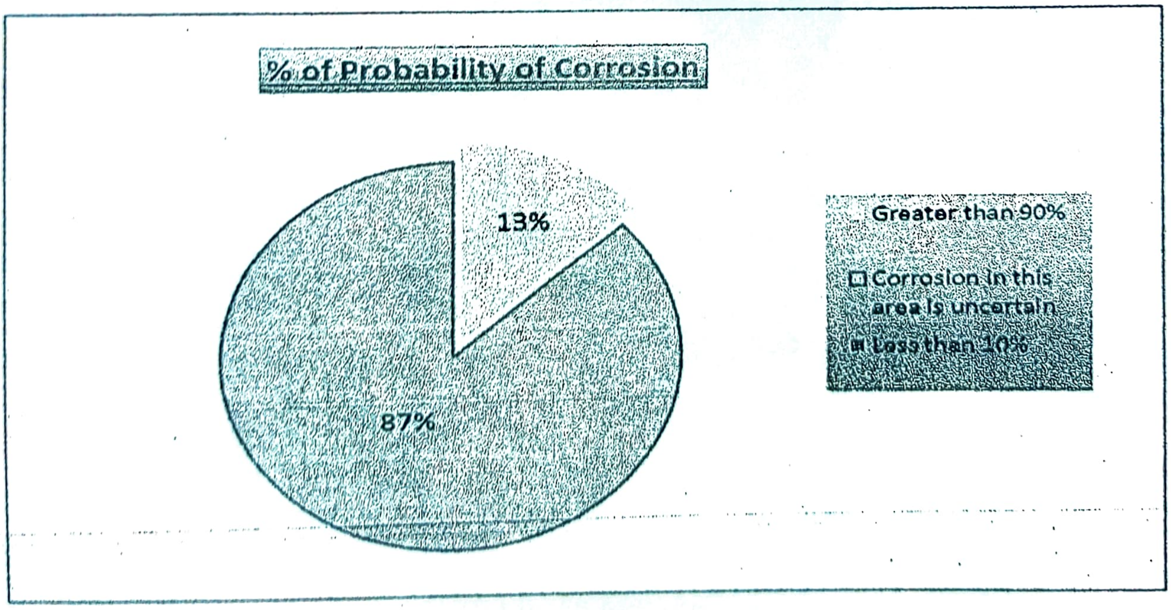
S. N.	Test Location		Rdg. 1	Rdg. 2	Rdg. 3	Rdg. 4	Rdg. 5	Rdg. 6	Rdg. 7	Rdg. 8	Rdg. 9	Rdg. 10	Avg. of HCP value	% of Probability of Corrosion as per ASTM-C-876
	Location	Mark ID												
1	Tower A Stilt	AC/2	-280	-285	-270	-310	-290	-283	-295	-300	-280	-290	-288.3	Probability of Corrosion to rebars in these area is uncertain
2	Tower A Stilt	AC/5	-303	-400	-345	-126	-238	-159	-177	-275	-198	-275	-249.6	Probability of Corrosion to rebars in these area is uncertain
3	Tower A Stilt	AC/7	-358	-364	-364	-328	-315	-282	-299	-345	-350	-342	-334.7	Probability of Corrosion to rebars in these area is uncertain
4	Tower B Stilt	BC/11	-335	-350	-326	-332	-360	-342	-350	-369	-370	-345	-347.9	Probability of Corrosion to rebars in these area is uncertain
5	Tower B Stilt	BC/12	-950	-210	-325	-351	-345	-291	-265	-345	-365	-310	-375.7	Probability of Corrosion to rebars in these area is Greater than 90 %
6	Tower B Stilt	BS/6	-313	-245	-265	-300	-310	-315	-319	-295	-287	-298	-294.7	Probability of Corrosion to rebars in these area is uncertain

S. N.	Test Location		Rdg. 1	Rdg. 2	Rdg. 3	Rdg. 4	Rdg. 5	Rdg. 6	Rdg. 7	Rdg. 8	Rdg. 9	Rdg. 10	Avg. of HCP value	% of Probability of Corrosion as per ASTM-C-876
	Location	Mark ID												
7	Tower C Stilt	CC/1	-280	-285	-270	-265	-245	-283	-295	-270	-280	-275	-274.8	Probability of Corrosion to rebars in these area is uncertain
8	Tower C Stilt	CC/6	-299	-250	-280	-270	-275	-280	-310	-295	-250	-270	-277.9	Probability of Corrosion to rebars in these area is uncertain
9	Tower C Stilt	CS/7	-290	-275	-290	-315	-305	-290	-310	-365	-310	-320	-307.0	Probability of Corrosion to rebars in these area is uncertain
10	Tower D Stilt	DC/6	-350	-380	-264	-310	-375	-277	-305	-392	-364	-370	-338.7	Probability of Corrosion to rebars in these area is uncertain
11	Tower D Stilt	DC/8	-375	-300	-374	-354	-310	-320	-380	-348	-357	-369	-348.7	Probability of Corrosion to rebars in these area is uncertain
12	Tower D Stilt	DB/4	-241	-250	-404	-419	-425	-230	-157	-355	-413	-412	-330.6	Probability of Corrosion to rebars in these area is uncertain
13	Tower E Stilt	EC/7	-392	-404	-390	-337	-335	-335	-390	-375	-377	-351	-368.6	Probability of Corrosion to rebars in these area is Greater than 90 %
14	Tower E Stilt	EB/1	-350	-324	-112	-360	-380	-465	-414	-371	-250	-238	-326.4	Probability of Corrosion to rebars in these area is uncertain
15	Tower E Stilt	EB/11	-390	-160	-272	-420	-458	-170	-425	-485	-360	-322	-346.2	Probability of Corrosion to rebars in these area is uncertain

HISTOGRAM PLOT FROM HALF CELL POTENTIAL FOR GHALIB APARTMENTS



PIE CHART RESULT DISTRIBUTION FROMHALF-CELL POTENTIAL FOR GHALIB APARTMENTS



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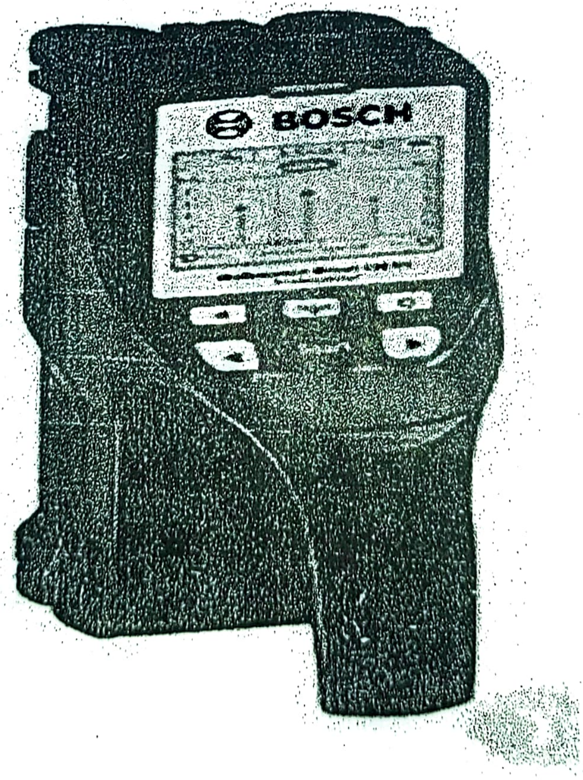
A.3 Cover Meter Test and Bar Locator

A cover meter was used to determine the cover to steel reinforcement embedded in concrete. The basic principal of measurement is that the presence of steel affects the field of an electromagnet and the intensity of this field can be correlated with the concrete cover to reinforcement. Provision of required cover made of good quality concrete significantly reduces chances of durability-related problems. The actual cover to the reinforcement bars was estimated using a hand-held cover meter. A set of three cover meter readings were taken at each test location and the minimum values of the measured cover have been reported in the observation sheets. Location of reinforcement bars are also identified so that the UPV test and Rebound hammer test can be avoided at that particular location since at the location of reinforcement the above two tests do not represent the actual readings.



COVER METER APPARATUS

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TABLE-4 (GHALIB APARTMENT)

S. N.	Test Location		Concrete Cover (mm)	Average Cover (mm)	Observation
	Location	Mark ID			
1	Tower A Stilt	AC/6	35,30,32,35,32	32	The diameter & cover of Rebars observed at scanned area of test location only.
2	Tower A Stilt	AC/7	30,25,32,35,40	32	
3	Tower A Stilt	AC/8	20,25,32,35,30	28	
4	Tower A Stilt	AC/12	35,30,28,30,35	31	
5	Tower B Stilt	BC/4	35,38,32,35,30	35	
6	Tower B Stilt	BB/3	38,40,32,35,35	36	
7	Tower B Stilt	BS/6	25,30,32,25,28	28	
8	Tower B Stilt	BS/12	30,25,28,30,30	28	
9	Tower C Stilt	CC/1	35,32,30,35,38	34	
10	Tower C Stilt	CC/4	30,35,35,32,35	33	
11	Tower C Stilt	CC/6	35,30,32,28,30	31	
12	Tower C Stilt	CS/2	25,28,30,25,32	28	
13	Tower D Stilt	DC/2	50,45,30,35,32	38	
14	Tower D Stilt	DC/6	45,30,35,32,35	35	
15	Tower D Stilt	DC/8	35,40,32,35,32	35	
16	Tower D Stilt	DB/4	35,32,30,35,32	32	
17	Tower E Stilt	EC/7	35,40,38,32,35	36.0	
18	Tower E Stilt	EC/9	40,32,35,38,32	35.0	
19	Tower E Stilt	EB/1	35,30,32,30,35	32.0	
20	Tower E Stilt	EB/2	30,28,32,30,35	31.0	

A.4 Carbonation Test

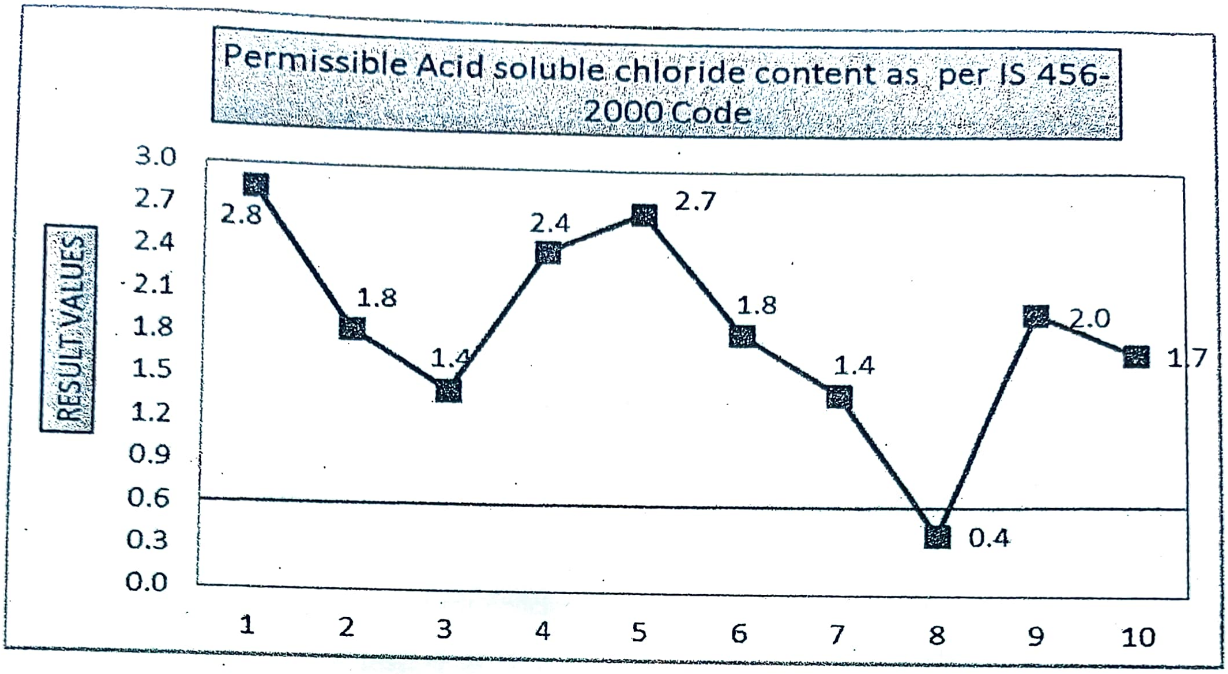
Carbonation of concrete because of attack from atmospheric carbon dioxide results in a reduction in alkalinity of the concrete, and increases the risk of reinforcement corrosion. Carbonation is usually restricted to a surface layer of only a few millimeters in thickness in good quality concrete but can be much deeper in poor quality concrete. The extent of carbonation is assessed by treating with phenolphthalein indicator the freshly exposed surface of concrete. For measurement of carbonation depth a selected location in column of the tower a hole was drilled and the powdered concreted which was obtained from this hole was treated with phenolphthalein. The location of the carbonation front was taken as the drill depth up to which the phenolphthalein indicator when applied to the powdered concrete changed from colorless to pink. Corrosion has been started.

A.5 ACID SOLUBLE CHLORIDE IN CONCRETE AND pH TEST

In addition to carbonation of concrete, corrosion is also caused due to presence of acid soluble chlorides in concrete. Chloride content in the column concrete of the towers is evaluated as per IS 14959 (Part 2): 2001 reaffirmed in 2011, Indian Standard Determination of Water Soluble and Acid Soluble Chlorides in Mortar and Concrete — Method of Test Part 2 Hardened Mortar and Concrete. pH value of concrete was also evaluated to estimate the rate of corrosion in steel reinforcement due to corrosion of concrete.

TABLE-5 (Ghalib Apartment)

S. N.	Test Location		Weight. of Sample (gm)	Vo% of distilled water	Vo% AgNo3 Wed in ml (X)	Y	% of Chloride Content	kg/m3	Permissible Acid soluble chloride content as per IS 456-2000 Code	ph value
	Location	Mark ID								
1	Tower A Stilt	AC/8	5gm	45ml	2.50ml	0.5	0.567	2.8	0.6kg/m3	8.7
2	Tower A Stilt	AC/12	5gm	45ml	2.50ml	1.2	0.369	1.8		9.4
3	Tower B Stilt	BC/4	5gm	45ml	2.50ml	1.5	0.284	1.4		10.2
4	Tower B Stilt	BB/3	5gm	45ml	2.50ml	0.8	0.482	2.4		11.6
5	Tower C Stilt	CC/4	5gm	45ml	2.50ml	0.6	0.539	2.7		9.8
6	Tower C Stilt	CC/6	5gm	45ml	2.50ml	1.2	0.369	1.8		8.3
7	Tower D Stilt	DC/6	5gm	45ml	2.50ml	1.5	0.284	1.4		9.1
8	Tower D Stilt	DC/8	5gm	45ml	2.50ml	2.2	0.085	0.4		6.5
9	Tower E Stilt	EB/1	5gm	45ml	2.50ml	1.1	0.397	2.0		8.9
10	Tower E Stilt	EB/2	5gm	45ml	2.50ml	1.3	0.340	1.7		8.0



A.6 COMPRESSIVE STRENGTH TEST ON CONCRETE CORE

A.6.1 Equipment

Equipment Core Drill Machine would be used for this purpose. It is a powerful yet portable core drilling system, which can be carried easily to the field. This Machine can be used to take core samples up a depth of about 350 mm.

It creates minimal vibrations and caused almost no disturbance to the in situ concrete. The diamond core bit to be used is of 50mm or 80 mm inner diameter X 350mm length.

Compressive Strength Test on Drilled Concrete Cores is required to determine the strength of hardened concrete in structure. Following are the specification for drilled concrete cores to be suitable for compressive strength test.

A.6.2 Procedure

Various components of the core drill machine are assembled together and fixed on the location earmarked for taking out core sample. There is provision for using water while cutting the concrete using diamond core bit to avoid any vibrations and keep the heat nominal. The diamond core bit can cut the concrete to a depth equivalent to its ends are cut and leveled to meet length/diameter ration as 2:1. The sample is placed in the Concrete Test machine and tested for compressive strength by following the procedure as perIS: 516-1959.

Diameter of concrete core:

The diameter of the core specimen for the determination of compressive strength in load bearing structural members shall be at least 3.70 inch[94 mm]. For concrete with nominal maximum size of aggregate greater than or equal to 1.5 inch[37.5 mm], the preferred minimum core diameter shall be ,three times the nominal maximum size of coarse aggregate but it should be at least two times the nominal maximum size of the coarse aggregates.

Length of Concrete Core:

The preferred length of capped specimen is between 1.9 and 2.1 times the diameter. High lengths can be trimmed and for specimens having low length, correction factor has to be applied in compressive test.

Capping of Concrete Core:

- If the ends of cores do not confirm to the perpendicularity and plainness requirement, they shall be sawed or ground or capped.
- If cores are capped, the capping device shall accommodate actual core diameters and produce caps that are concentric with the core ends.
- The material used for capping shall be such that it's compressive strength is greater than that of the concrete in the core.
- Caps shall be made as thin as practicable and shall not flow or fracture before the concrete fails when specimen is tested.
- Capped surface shall be right angles to the axis of the specimen and shall not detach depart from a plane by more than 0.05 mm.
- Measure core lengths to the nearest 0.1 inch [2 mm] before capping.

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Calculation of Compressive Strength:

Calculate the compressive test of the specimen using the computed cross sectional area based on average diameter of the specimen. If the L/D ratio is 1.75 or less, correct the result obtained by multiplying with Correction factors as given below:

L/D Ratio	Correction Factor
1.75	0.98
1.5	0.96
1.25	0.93
1.0	0.87

The value obtained after multiplying with correction factor is called corrected compressive strength, this being equivalent strength of a cylinder having L/D ratio of 2. The equivalent cube strength can be calculated by multiplying the corrected cylinder strength by 5/4.

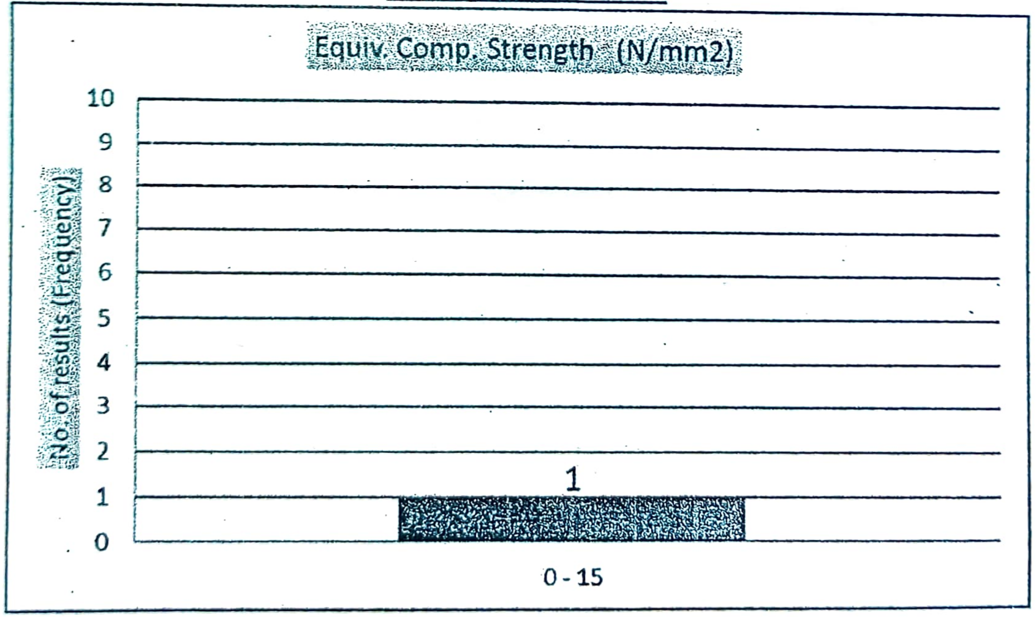
- **Core samples had to be taken at 3 locations but due to fragile nature of concrete, core sample have been taken only from 1 location, as condition of concrete is very weak.**

Sadanand Ojha

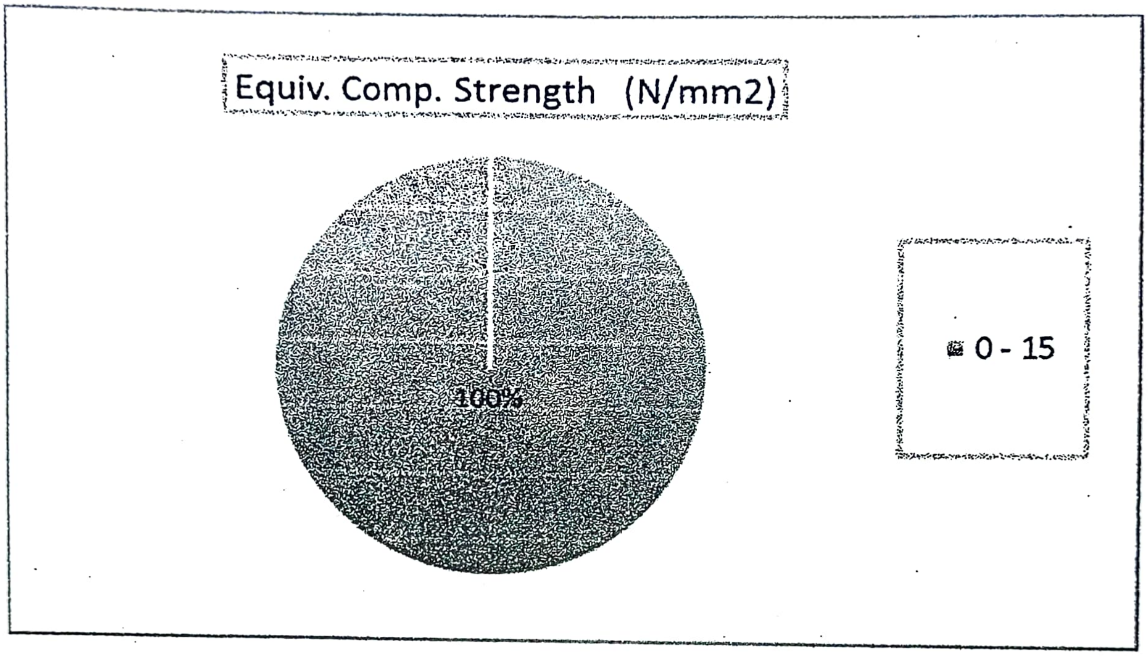
TABLE-6 (Ghalib Apartments)

Lab I.D. Mark	Location of Core	Final Length (mm)	Core Diameter (mm)	Mass (Natural) (kg)	Mass (Saturated) (kg)	L/D Ratio	Correction Factor for L/D Ratio	Core Failure Load (kN) (a)	Cylindrical Comp. Strength (N/mm2) (b)	Corrected Cyl. Comp. Strength for Cores<100 mm dia. (c)	Corrected Cyl. Comp. Strength for L/D ratio (N/mm2) (d)	Equivalent Cube Comp. Strength (N/mm2) (e)	Natural Density (Kg/m3)	Saturated Density (Kg/m3)
Core-1	CC/6	100	45	0.681	0.920	2.22	1.024	**12**	7.55	8.15	8.35	**10.4**	4284	5788
Core-2	AC/8	Core samples could not be taken at these 2 locations due to fragile nature of concrete, core sample have been taken only from 1 location , as condition of concrete is very weak												
Core-3	DC/9													

HISTOGRAM PLOT FOR EQUIVALENT COMP. STRENGTH OF GHALIB APARTMENTS



PIE CHART OF RESULT DISTRIBUTION OF COMP. STRENGTH OF GHALIB APARTMENTS



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ANNEXURE-B

STRUCTURE ANALYSIS REPORT

B.0 ANALYSIS OF THE STRUCTURE

B.1 INTRODUCTION

The proposed structures consists of Staircases, lifts, toilets, rooms and kitchens etc. Framing system shall consist of columns, beams & slabs to resist both vertical & lateral loads. The structures has been analyzed using Etabs software. The analysis report of the structures have been presented in this report.

B.2 SIZES OF COLUMN & BEAMS

Various sizes of beams and columns have been assigned in the model in accordance with the sizes mentioned in the drawing.

a) Columns = 230X230

b) Beams = 230X400 & 200X450

B.3 LIST OF CODES

List of primary Indian standards codes and publications used for analysis and design given in Table below:

IS 875:1987 Design Loads (Other Than Earthquake) for Buildings and Structures		**Code of Practice for Design Loads for Buildings and Structures**
	Part I	Dead Loads
	Part II	Imposed Loads
	Part V	Special Loads and Combinations
IS 875:2015 Part III		Wind loads
NBC: 2016		National Building Code of India 2016
IS 1893:2016 (Part 1 General Provisions and Building)		Criteria for Earthquake Resistant Design of Structures
IS 13920:2016		Code of Practice for Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces
IS 456:2000/2013(amendment)		Plain and Reinforced Concrete - Code of Practice

B.4 SOFTWARE USED

ETABS, a Product of Computers and Structures Inc. have been used for the analysis of the structure.

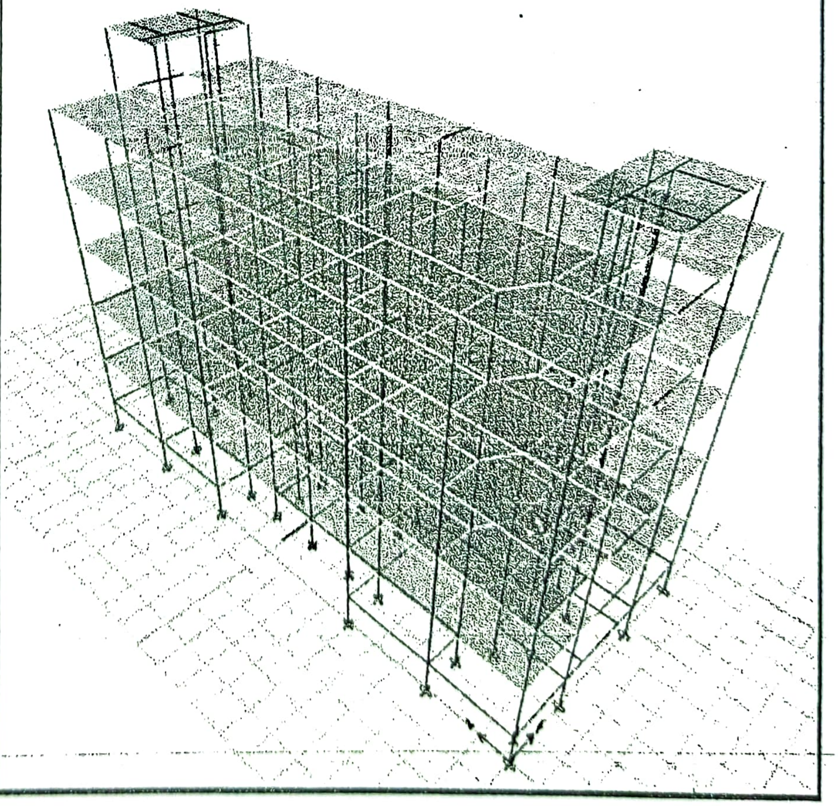
B.5 SEISMIC AND WIND DESIGN CRITERIA

SEISMIC DESIGN CRITERIA:

Seismic loads are calculated as per provisions of IS: 1893-2016. The parameters selected are given below

Parameter	Value
Z - i.e. Zone Factor	0.24 (For zone IV)
I - Importance Factor	1.2
Analysis type	Dynamic analysis

(SIMPLIFIED VULNERABILITY ASSESSMENT)

	Name of the District	Delhi	Building block	1
	Block	SINGLE BUILDING	Building Code	GA_01
	Building Name	Ghalib Apartment Co-operative Group Housing Societies Pitampura, Delhi-110088	Building Code(CES)	CES-0001
S.No.	Parameters	Description		Comments
1	Brief Structural Description	Ghalib Apartment is basement + Ground + 4 storey RCC building with a concrete slab. The building measures 30.4m x 12.2m in plan. Wall Thickness are of 250mm and 110mm.		
2	Visual Structural Damage	Cracks & are observed at almost every location in the structure. There is visible Blistering, Delamination, Seepage, Efflorescence & Dampness in the structure. Rusting of reinforcement has been observed in some exposed reinforcement.		
3	Model			

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4	Category	Building Category as per Table 2 IS 4326-2016	Moment Resisting R.C.C Framed Structure		
5	EQ Zone	IV			
6	Zone Factor	0.24			
7	Importance factor	1.2			
8	R factor	5			
9	Building height from Base	19.12			
10	length of building in X- direction	15.189			
11	length of building in Y- direction	9.22			
12	Time Periods as per IS1893	X direction	0.44	sec.	
		Y direction	0.57	sec.	
13	Time Periods (from ETABs)	X direction	0.072	sec.	
		Y direction	0.07	sec.	
14	Sa/g	As per figure 2(a) IS1893:2016	2.5		
15	A^h factor	X direction	0.072		
		Y direction	0.072		
16	Siesmic Weight	The DL lumped at the roof of the building	14628	KN	
17	Total Weight	Total Dead Load + Live Load of the building 1. Density of Masonary : 20KN/m^3 2. Density of Reinforced concrete: 25KN/m^3 3. Roof finish load on roof: 3 KN/m^2	18134	KN	
18	Static Base Shear (from ETABs model)	X direction	1174	KN	
		Y direction	1174	KN	
19	Calculated Static Base Shear	X direction	1053.21024	KN	
		Y direction	1053.21024	KN	
20	Base Shear %age	X direction	8.03	%	
		Y direction	8.03	%	
21	Knowledge Factor	0.5			

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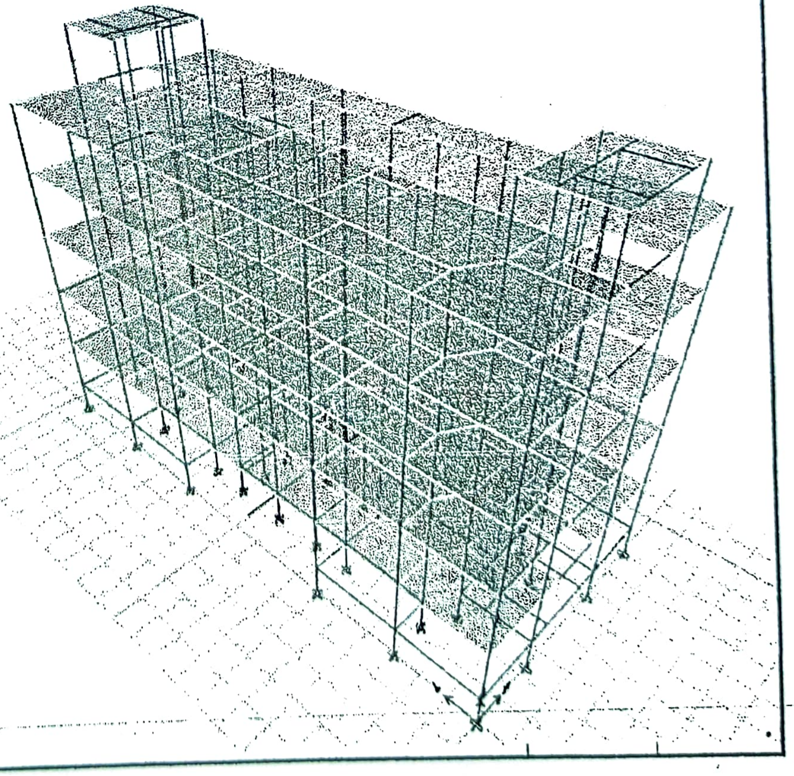
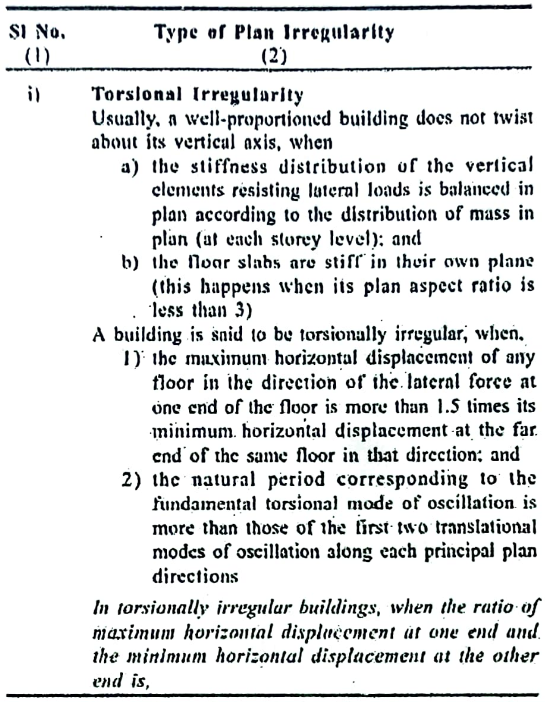
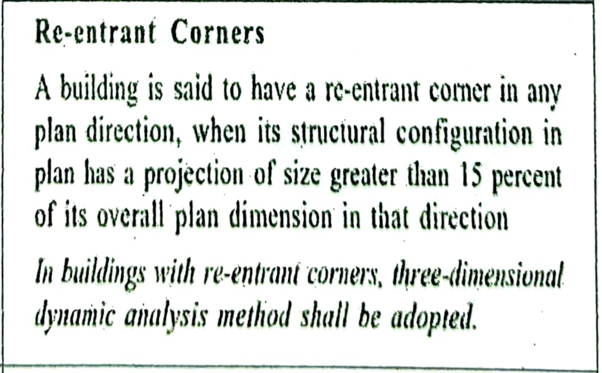
22	**Material Properties**				
	Reinforced Concrete				
	Weight per Unit Volume (RCC)		25	KN/m^2	
	Compressive Strength (RCC)	(assumed)	15	N/mm^2	
	Modulus of Elasticity		19364.92	mpa	
	Masonary Brick				
	compressive Strength (brick wall)	(assumed)	7.5	N/mm^2	
	Modulus of Elasticity		4125	mpa	
23	Ductile detailing as per IS13920 -2016		N		
24	**Mathematical Model**	A linear elastic model of the structure has been created in ETABs. RCC columns and beams are modelled as line element of respective dimensions and properties			
	Assumptions of Mathematical Model				
	Lateral system has been assumed to be Special Moment Resisting Frame(SMRF). Therefore R factor of 5 has been used.				
	For slabs, beams and columns material shall be concrete of grade M15(assumed)				
	Half of the total mass of each infill walls has been lumped as linear load on the beam above the repective wall at terrace level				
	Slabs shall be defined as Membrane				
	Partition walls of height less than storey height shall not be modelled.				
	Height of walls shall be equal to storey height				
	Diaphragm shall be defined Rigid for slabs				
	Parapet walls shall not be modelled. Zero density beams will be used for transfering parapet wall load.				
	RCC beams and columns have been modelled as line elements				

25	**Irregularity Checks**				
25.1	Irregularity Check (Plan)	Type of Irregularity	Presence (Y/N)		
		Torsional	N		
		 Sl No. (1) Type of Plan Irregularity (2) i) **Torsional Irregularity** Usually, a well-proportioned building does not twist about its vertical axis, when a) the stiffness distribution of the vertical elements resisting lateral loads is balanced in plan according to the distribution of mass in plan (at each storey level); and b) the floor slabs are stiff in their own plane (this happens when its plan aspect ratio is less than 3) A building is said to be torsionally irregular, when, 1) the maximum horizontal displacement of any floor in the direction of the lateral force at one end of the floor is more than 1.5 times its minimum horizontal displacement at the far end of the same floor in that direction; and 2) the natural period corresponding to the fundamental torsional mode of oscillation is more than those of the first two translational modes of oscillation along each principal plan directions *In torsionally irregular buildings, when the ratio of maximum horizontal displacement at one end and the minimum horizontal displacement at the other end is,*			
	Comments/Calculations	Comparing the maximum displacement for siesmic load at each end of the floor, obtained from ETABs model. X dim : Horizontal Displacements at two ends = 40.52 mm and 42.90 mm Ratio = 0.9< 1.5 . Therefore, OK. Y dim : Horizontal Displacements at two ends = 48.95 mm and 52.33 mm. Ratio = 0.9< 1.5 . Therefore, torsional irregularity doesn't exists.			
25.2	Irregularity Check(plan)	Re-Enterant Corner	N		
		 **Re-entrant Corners** A building is said to have a re-entrant corner in any plan direction, when its structural configuration in plan has a projection of size greater than 15 percent of its overall plan dimension in that direction *In buildings with re-entrant corners, three-dimensional dynamic analysis method shall be adopted.*			
	Comments/Calculations	There is no plan projection in the structure. Therefore, OK.			

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25.3		Cutout in floor slabs	N		
	Irregularity Check(plan)	**Floor Slabs having Excessive Cut-Outs or Openings** Openings in slabs result in flexible diaphragm behaviour, and hence the lateral shear force is not shared by the frames and/or vertical members in proportion to their lateral translational stiffness. The problem is particularly accentuated when the opening is close to the edge of the slab. A building is said to have discontinuity in their in-plane stiffness, when floor slabs have cut-outs or openings of area more than 50 percent of the full area of the floor slab *In buildings with discontinuity in their in-plane stiffness, if the area of the geometric cut-out is,* a) *less than or equal to 50 percent, the floor slab shall be taken as rigid or flexible depending on the location of and size of openings; and* b) *more than 50 percent, the floor slab shall be taken as flexible.*			
	Comments/Calculations	There are no significant cut-outs or openings in the slab. Therefore, OK.			
25.4		Out of plane offset in vert. elements	N		
	Irregularity Check(plan)	**Out-of-Plane Offsets In Vertical Elements** Out-of-plane offsets in vertical elements resisting lateral loads cause discontinuities and detours in the load path, which is known to be detrimental to the earthquake safety of the building. A building is said to have out-of-plane offset in vertical elements, when structural walls or frames are moved out of plane in any storey along the height of the building *In a building with out-of-plane offsets in vertical elements,* a) *specialist literature shall be referred for design of such a building, if the building is located in Seismic Zone II; and* b) *the following two conditions shall be satisfied, if the building is located in Seismic Zones III, IV and V:* 1) *Lateral drift shall be less than 0.2 percent in the storey having the offset and in the storeys below; and* 2) *Specialist literature shall be referred for removing the irregularity arising due to out-of-plane offsets in vertical elements.*			
	Comments/Calculations	There are no out-of-plane in vertical elements. Therefore, OK.			

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25.5	Irregularity Check(plan)	Non-parallel LFRS	N		
		Non-Parallel Lateral Force System Buildings undergo complex earthquake behaviour and hence damage, when they do not have lateral force resisting systems oriented along two plan directions that are orthogonal to each other. A building is said to have non-parallel system when the vertically oriented structural systems resisting lateral forces are not oriented along the two principal orthogonal axes in plan *Buildings with non-parallel lateral force resisting system shall be analyzed for load combinations mentioned in* **6.3.2.2** or **6.3.4.1**.			
	Comments/Calculations	All walls are directed along two principal direction. Therefore, OK.			
25.6	Irregularity Check (vertical)	Stiffness Irregulariy	N		
		Stiffness Irregularity (Soft Storey) A soft storey is a storey whose lateral stiffness is less than that of the storey above. *The structural plan density (SPD) shall be estimated when unreinforced masonry infills are used. When SPD of masonry infills exceeds 20 percent, the effect of URM infills shall be considered by explicitly modelling the same in structural analysis (as per 7.9). The design forces for RC members shall be larger of that obtained from analysis of:* *a) Bare frame, and* *b) Frames with URM infills,using 3D modelling of the structure. In buildings designed considering URM infills, the inter-storey drift shall be limited to 0.2 percent in the storey with stiffening and also in all storeys below.*			
	Comments/Calculations	Not applicable			

25.7	Irregularity Check (vertical)	Mass irregularity **Mass Irregularity** Mass irregularity shall be considered to exist, when the seismic weight *(as per 7.7)* of any floor is more than 150 percent of that of the floors below. *In buildings with mass irregularity and located in Seismic Zones III, IV and V, the earthquake effects shall be estimated by Dynamic Analysis Method (as per 7.7).*	Y		
	Comments/Calculations	Not applicable			
25.8	Irregularity Check (vertical)	Vertical geometric irregularity **Vertical Geometric Irregularity** Vertical geometric irregularity shall be considered to exist, when the horizontal dimension of the lateral force resisting system in any storey is more than 125 percent of the storey below. *In buildings with vertical geometric irregularity and located in Seismic Zones III, IV and V, the earthquake effects shall be estimated by Dynamic Analysis Method (as per 7.7).*	N		
	Comments/Calculations	Not applicable			
25.9	Irregularity Check (vertical)	In-plane discontinuity in vertical element **In-Plane Discontinuity in Vertical Elements Resisting Lateral Force** In-plane discontinuity in vertical elements which are resisting lateral force shall be considered to exist, when in-plane offset of the lateral force resisting elements is greater than 20 percent of the plan length of those elements. *In buildings with in-plane discontinuity and located in Seismic Zones II, the lateral drift of the building under the design lateral force shall be limited to 0.2 percent of the building height; in Seismic Zones III, IV and V, buildings with in-plane discontinuity shall not be permitted.*	N		
	Comments/Calculations	Not applicable			
25.10	Irregularity Check (vertical)	Strength Irregularity **Strength Irregularity (Weak Storey)** A weak storey is a storey whose lateral strength is less than that of the storey above. *In such a case, buildings in Seismic Zones III, IV and V shall be designed such that safety of the building is not jeopardized; also, provisions of 7.10 shall be followed.*	N		
	Comments/Calculations	Not applicable			

25.11	Irregularity Check (vertical)	Stub Columns N **Floating or Stub Columns** Such columns are likely to cause concentrated damage in the structure. *This feature is undesirable, and hence should be prohibited, if it is part of or supporting the primary lateral load resisting system.*		
	Comments/Calculations	Not stub columns are present		
25.12	Irregularity Check (vertical)	Irregular Modes N **Irregular Modes of Oscillation in Two Principal Plan Directions** Stiffnesses of beams, columns, braces and structural walls determine the lateral stiffness of a building in each principal plan direction. A building is said to have lateral storey irregularity in a principal plan direction, if a) the first three modes contribute less than 65 percent mass participation factor in each principal plan direction, and b) the fundamental lateral natural periods of the building in the two principal plan directions are closer to each other by 10 percent of the larger value. *In buildings located in Seismic Zones II and III, it shall be ensured that the first three modes together contribute at least 65 percent mass participation factor in each principal plan direction. And, in buildings located in Seismic Zones IV and V, it shall be ensured that,* 1) *the first three modes together contribute at least 65 percent mass participation factor in each principal plan direction, and* 2) *the fundamental lateral natural periods of the building in the two principal plan directions are away from each other by at least 10 percent of the larger value.*		
	Comments/Calculations	Fundamanetal natural periods obtained from dynamic analysis done in ETABs are 1.224 sec.,1.173 sec. and 1.06 sec. Therefore, fundamental periods are away from each other by more than 10%. Hence, Irregular modes does not exist.		

26	Other Checks				
26.1	Column Width Check (as per IS13920-2016)	Column Width Deficiency	Y		
		7.1.1 The minimum dimension of a column shall not be less than, a) 20 d_b, where d_b is diameter of the largest diameter longitudinal reinforcement bar in the beam passing through or anchoring into the column at the joint, or b) 300 mm (*see* Fig. 7).			
	Comments/Calculations	Width of the comuns is less than 300mm, amd 20 d_b clause is not satisfied. Hence,This check is not satisfied			
26.2	Beam Width Check (as per IS13920-2016)	Beam Width Deficiency	N		
		6.1.2 Beams shall not have width less than 200 mm.			
	Comments/Calculations	Since width of all beams is more than 200mm, this check is satisfied.			
26.3	Approximate Column Shear Stress Check (as per IS15988-2013)	Column Shear Strength Deficiency	Y		
		6.5.1 *Shear Stress in Reinforced Concrete Frame Columns* The average shear stress in concrete columns, τ_{col}, computed in accordance with the following equation shall be lesser of, a) 0.4 MPa; and b) $0.10\sqrt{f_{ck}}$, f_{ck} is characteristic cube strength of concrete: $\tau_{col} = \left(\frac{n_c}{n_c - n_f}\right)\left(\frac{V_j}{A_c}\right)$			
		Total no. of Columns	12		
		Total no. of frames in X dim	5		
		Total no. of frames in Y dim	3		
		Total cross-sectional area of columns (in mm^2)	52900		
		Average Shear Stress (X dim.)	38.045		
		Average Shear Stress (Y dim.)	29.590		
		Maximum Allowed Shear Stress	0.20	mpa	
	Comments/Calculations	Since, average shear stress in both X and Y direction exceeds the maximum allowable shear stress. So, the size of the columns shall be increased.			

20.4	Axial Stress In Moment Frames (as per IS15988-2013)	Axial Stress Deficiency	N		
		6.5.4 Axial Stress in Moment Frames The maximum compressive axial stress in the columns of moment frames at base due to overturning forces alone (F_0) as calculated using the following equation shall be less than $0.25 f_{ck}$. $F_0 = \frac{2}{3}\left(\frac{V_B}{n_f}\right)\left(\frac{H}{L}\right)$ where n_f = total number of frames in the direction of loading, V_B = base shear, H = total height, and L = length of the building.			
		Total Height of Building	19.12	m	
		Total Dimension of Building (X dim)	15.189	m	
		Total Dimension of Building (Y dim)	9.22	m	
		Overturning Force (X dim)	197.05	KN	
		Overturning Force (Y dim)	328.41	KN	
		Axial Stress X dim	3.725	mpa	
		Axial Stress X dim	6.208	mpa	
		Maximum Allowable axial stress	1.875	mpa	
	Comments/Calculations	Since axial stress due to overturning in much less than maximum allowable axial stress, axial stress deficiency doesn't exist.			
26.5		Shear Stress Check for URM Walls	N		
	Shear Stress Check for Unreinforced masonry shear walls (as per IS15988:2013)	*6.5.3 Shear Stress Check for Reinforced Concrete Masonry Infill Walls* The shear stress in the reinforced masonry shear walls shall be less than 0.30 MPa and the shear stress in the unreinforced masonry shear walls shall be less than 0.10 MPa.			
	Comments/Calculations	Not applicable			
26.6		Aspect Ratio Check for RCC Columns/Walls	N		
	Aspect Ratio Check for RCC Columns (as per IS13920:2016)	**7.1.2** The cross-section aspect ratio (that is, ratio of smaller dimension to larger dimension of the cross-section of a column or inclined member) shall not be less than 0.45. Vertical members of RC buildings whose cross-section aspect ratio is less than 0.4 shall be designed as per requirements of **9**.			
	Comments/Calculations	Since, aspect ratio for all coulmns is less than 0.45 this condition is met.			

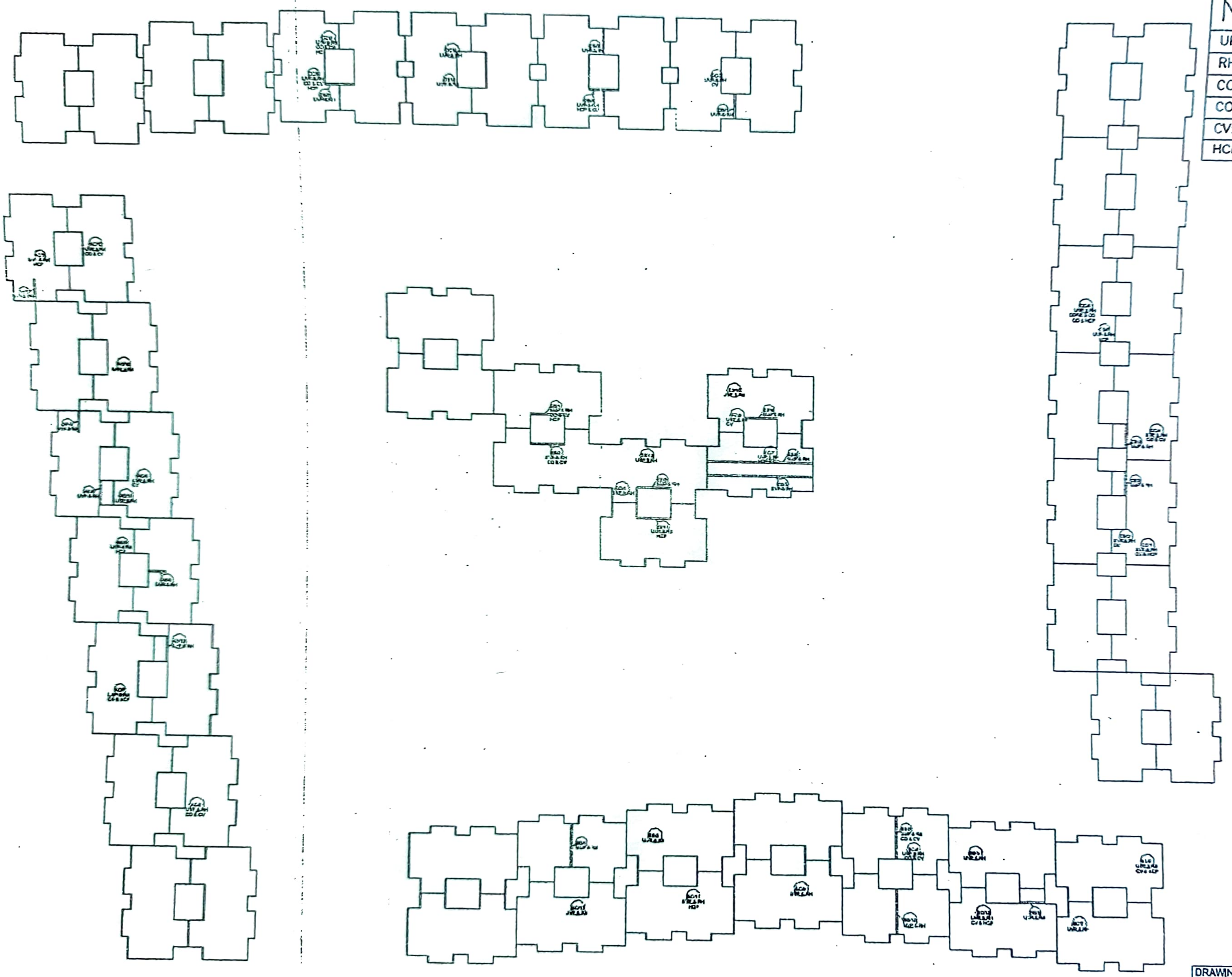
<table>
<tr><td>26.7</td><td>Story Drift Limit
(as per IS1893-2016)</td><td>Story Drift Limit Exceeded</td><td>N</td><td></td><td></td></tr>
<tr><td></td><td></td><td colspan="3">7.11.1 <i>Storey Drift Limitation</i>
7.11.1.1 Storey drift in any storey shall not exceed 0.004 times the storey height, under the action of design base of shear V_B with no load factors mentioned in 6.3, that is, with partial safety factor for all loads taken as 1.0.
7.11.1.2 Displacement estimates obtained from dynamic analysis methods shall not be scaled as given in 7.7.3.</td><td></td></tr>
<tr><td></td><td></td><td>Story Height</td><td>3.09</td><td>m</td><td></td></tr>
<tr><td></td><td></td><td>Displacement (X dim)</td><td>5</td><td>mm</td><td></td></tr>
<tr><td></td><td></td><td>Displacement (Y dim)</td><td>4.24</td><td>mm</td><td></td></tr>
<tr><td></td><td></td><td>Allowable Displacement</td><td>12.36</td><td>mm</td><td></td></tr>
<tr><td></td><td>Comments/Calculations</td><td colspan="3">Since, story displacement in each direction is much less than maximum allowable story displacement story drift limit is not exceeded</td><td></td></tr>
<tr><td>26.8</td><td>Strong Column/ Weak Beam
(as per IS15988:2013)</td><td>Strong Column-Weak Beam deficiency</td><td>N</td><td></td><td></td></tr>
<tr><td></td><td></td><td colspan="2">c) <i>Strong column/weak beam</i> — The sum of the moment of resistance of the columns shall be at least 1.1 times the sum of the moment of resistance of the beams at each frame joint.</td><td></td><td></td></tr>
<tr><td></td><td>Comments/Calculations</td><td colspan="2">Strong Column-Weak Beam Assumption is satisfied</td><td></td><td></td></tr>
<tr><td>26.9</td><td>Parapet Wall Check
(as per IS1893)</td><td>Parapet Wall Aspect Ratio Deficiency</td><td>N</td><td></td><td></td></tr>
<tr><td></td><td></td><td colspan="2">c) <i>Unreinforced masonry parapets</i> — The maximum height of an unsupported unreinforced masonry parapet shall not exceed the height-to-thickness ratio as shown in Table 4. If the required parapet height exceeds this maximum height, a bracing system designed for the forces determined as per non-structural elements specified in 8.5.2.2, shall support the top of the parapet. The minimum height of a parapet above any wall anchor shall be 300 mm. If a reinforced concrete beam is provided at the top of the wall, the minimum height above the wall anchor may be 150 mm.

Table 4 Maximum Allowable <i>h/t</i> Ratio for Parapets
<table>
<tr><th>Unreinforced Masonry Parapets</th><th>Zone V</th><th>All Other Zones</th></tr>
<tr><td>Maximum allowable height-to-thickness ratio</td><td>1.5</td><td>2.5</td></tr>
</table></td><td></td><td></td></tr>
<tr><td></td><td>Comments/Calculations</td><td colspan="2">Not Applicable</td><td></td><td></td></tr>
</table>

26.10	Partition wall check (as per -IS 1905(Appedix D))	**D-3. PARTITION WALLS** D-3.1 These are internal walls usually subjected to much smaller lateral forces. Behaviour of such wall is similar to that of panel wall and these could, therefore, be designed on similar lines. However, in view of smaller lateral loads, ordinarily these could be apportioned empirically as follows: a) Walls with adequate lateral restraint at both ends but not at the top: 1) The panel may be of any height, provided the length does not exceed 40 times the thickness; or 2) The panel may be of any length, provided the height does not exceed 15 times the thickness (that is, it may be considered as a free-standing wall); or 3) Where the length of the panel is over 40 times and less than 60 times the thickness, the height plus twice the length may not exceed 135 times the thickness; b) walls with adequate lateral restraint at both ends and at the top: 1) The panel may be of any height, provided the length does not exceed 40 times the thickness; or 2) The panel may be of any length, provided the height does not exceed 30 times the thickness; or 3) Where the length of the panel is over 40 times and less than 110 times the thickness, the length plus three times the height should not exceed 200 times the thickness; and c) When walls have adequate lateral restraint at the top but not at the ends, the panel may be of any length, provided the height does not exceed 30 times the thickness.	Height of Wall(mm)	3090	
			Length of Wall(mm)	4800	
			Thickness of Wall(mm)	225	
			Height to Thickness Ratio	13.73	
			Length to Thickness Ratio	21.33	
			(Height + 2 X Length)/ Thickness ratio	56.40	
			a)	N	
			a1)	NA	
			a2)	NA	
			a3)	NA	
			b)	Y	
			b1)	Since for the longest and the thinest wall in the structure length does not exceed 40 times the thickness, So OK.	
			b2)	N	
			b3)	NA	
			c)	N	
			c1)	NA	
		D-3.2 Strength of bricks used in partition walls should not be less than 3·5 N/mm² or the strength of masonry units used in adjoining masonry, whichever is less. Grade of mortar should not be leaner than M2.	Strength of Bricks(N/mm2)	3.5 MPA assumed. Hence OK	
			Grade of Mortar	M2 grade assumed. Hence OK	
27	Stress in Structural Members (from ETABs)	During preliminary ETABs design structural members(beams and columns) were found to be overstressed.			
29	Recommendations	Since, During preliminary ETABs design structural members(beams and columns) were found to be overstressed. The structure is structrually unstabled.			

ANNEXURE-C

DRAWINGS SHOWING TEST LOCATIONS



NOTES:-

UPV:- ULTRASONIC PULSE VELOCITY
RH:- REBOUND HAMMER TEST
CO:- CARBONATION TEST
CORE:- CONCRETE CORE
CV:- COVER METER TEST
HCP:- Half-cell Potential

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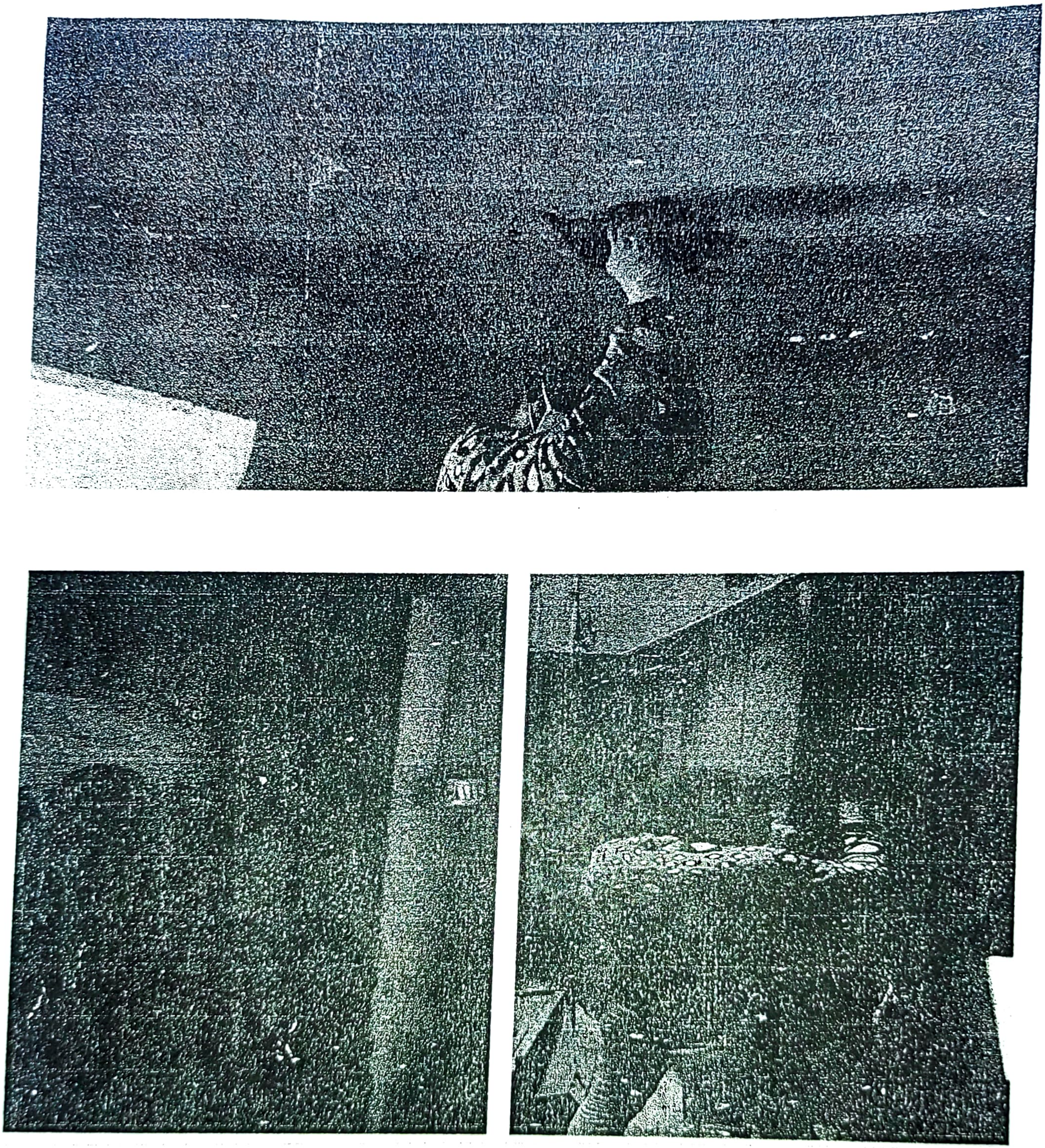
DRAWING TITLE:-

TEST LOCATION PLAN GHALIB APARTMENT

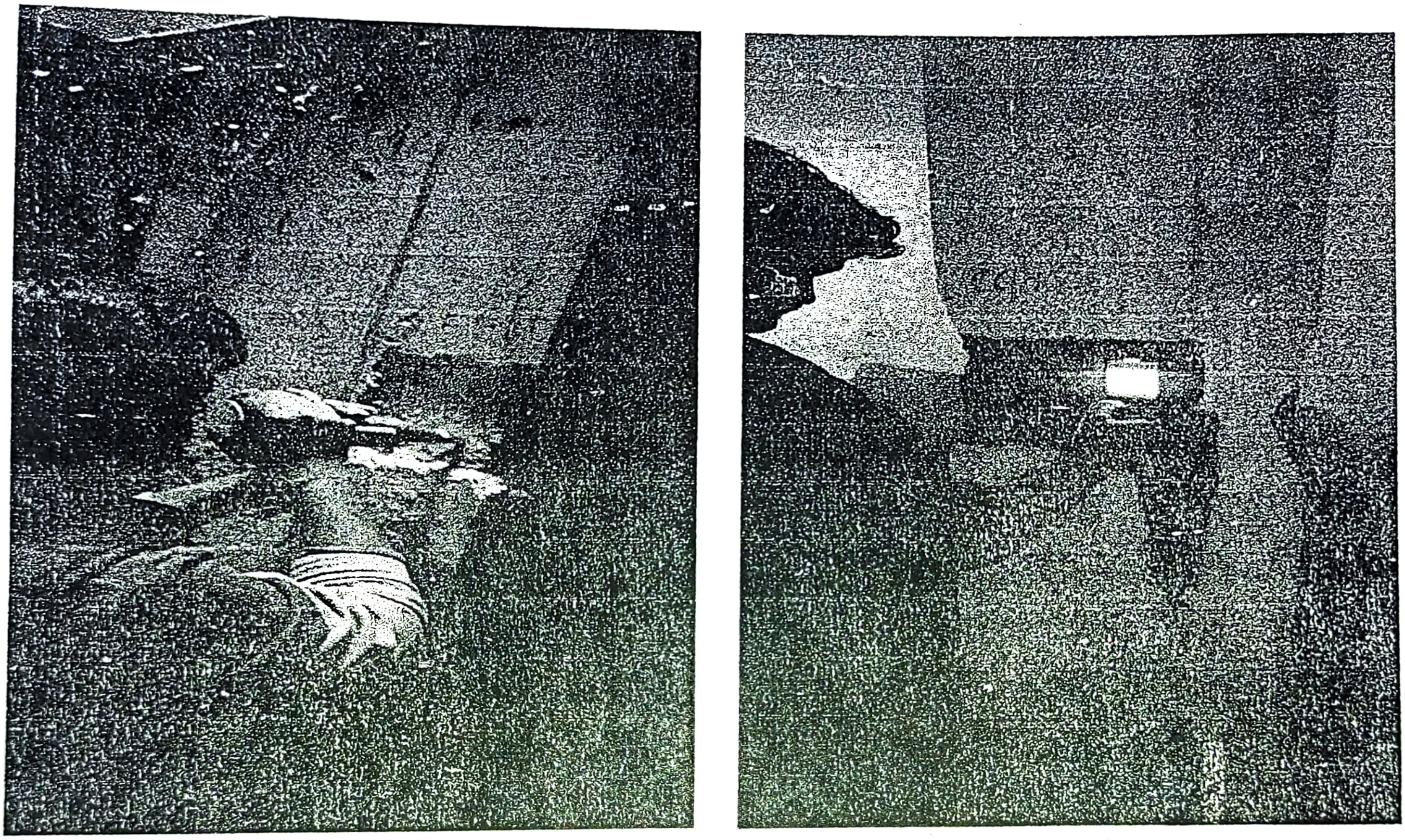
DEALT BY:-	RAJ KISHOR	SHEET NO:-56
CHECKED BY:-	SADANAND OJHA	

ANNEXURE-D

DISTRESS MAPPING

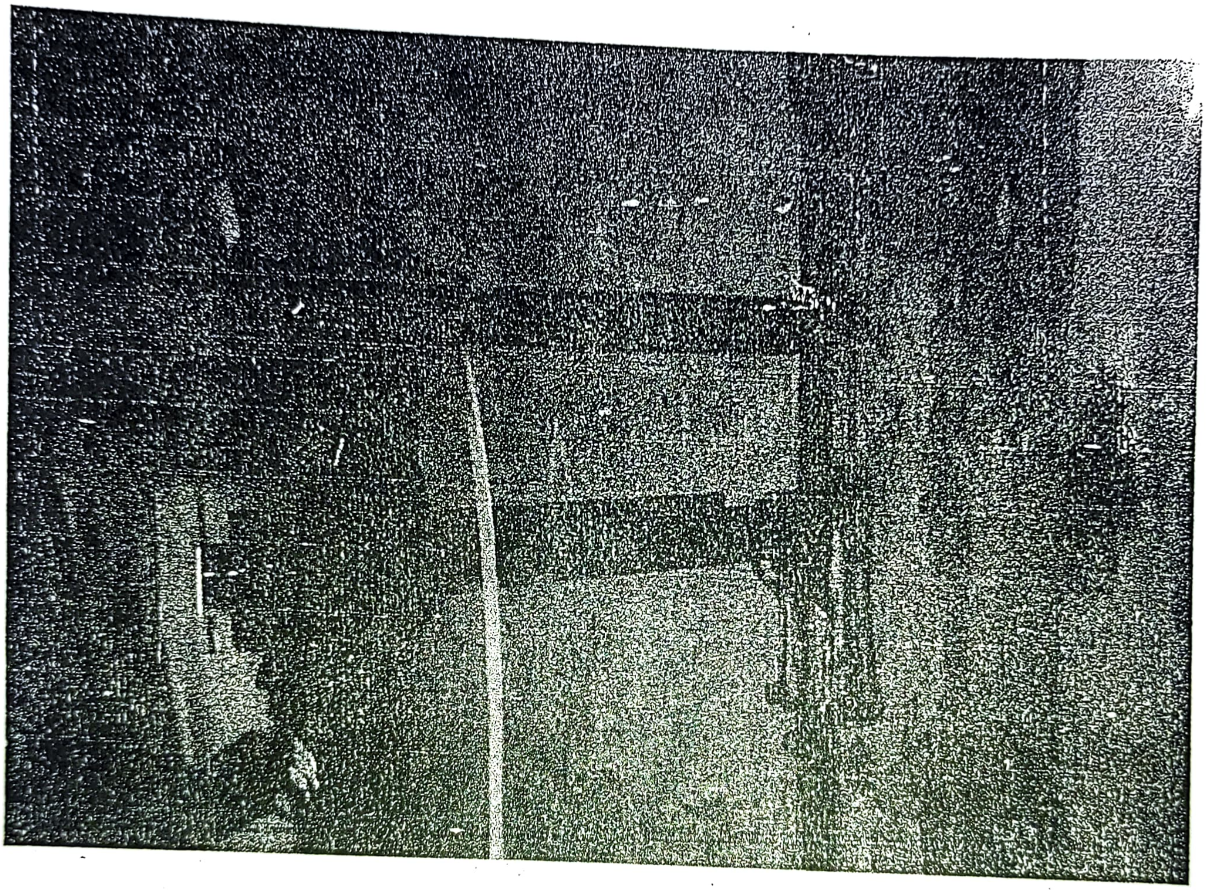


UPV TEST IN PROGRESS

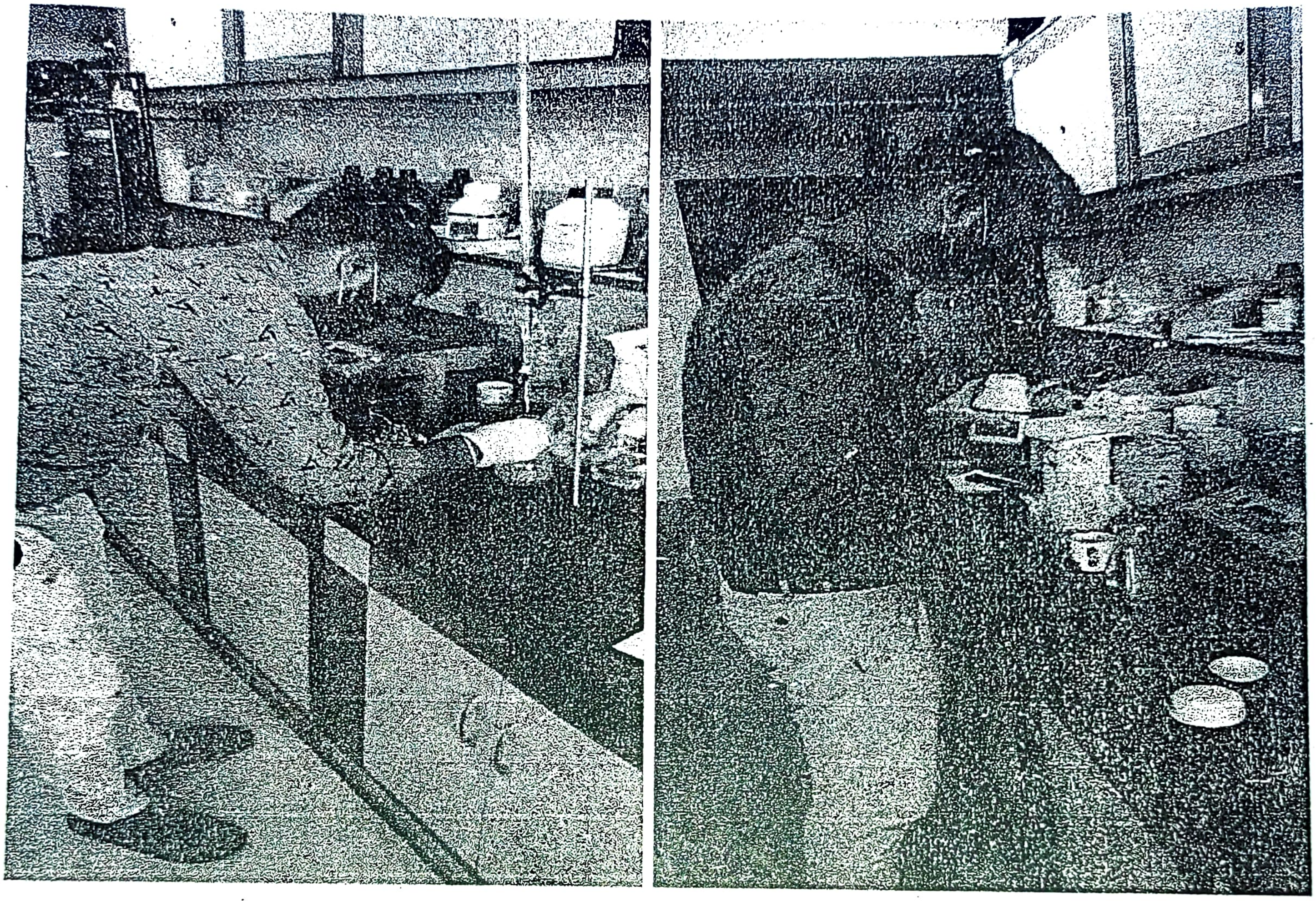


RH AND COVER METER TEST IN PROGRESS

HALF CELL POTENTIO METER



CORE CUTTING TEST IN PROGRESS



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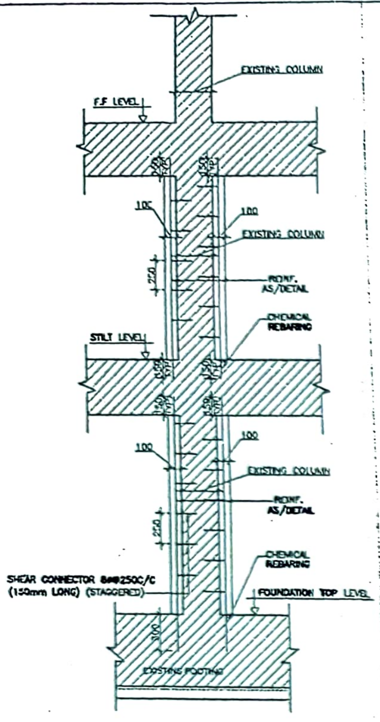
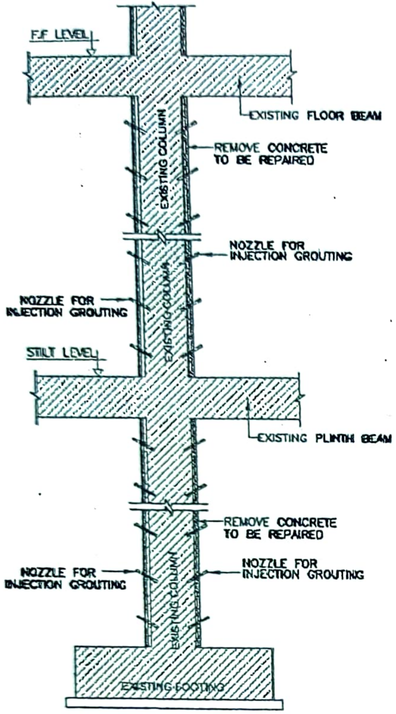
CHLORIDE TEST IN PROGRESS

ANNEXURE-E

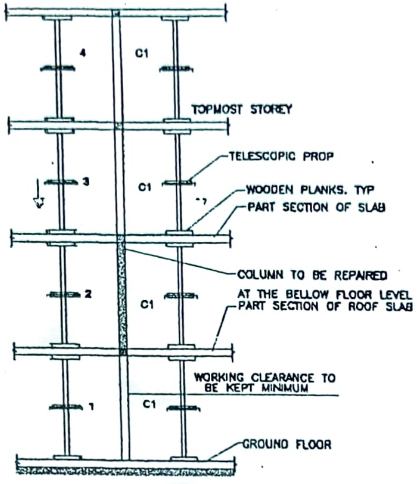
DRAWINGS



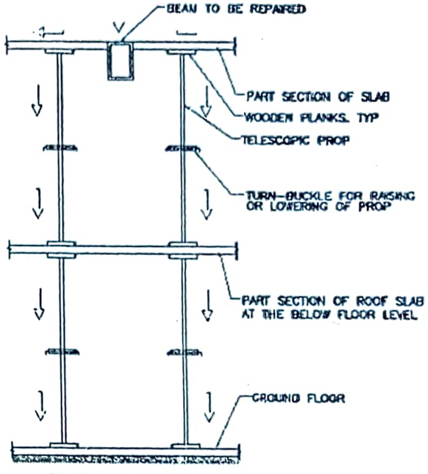
TYPICAL SECTION OF COLUMN JACKETING
FROM EXISTING FOOTING
(TO INCREASE COLUMN CAPACITY)



TYPICAL DETAIL OF INJECTION GROUTING IN COLUMNS



TYPICAL ARRANGEMENT OF PROPPING
FOR RELIEVING A COLUMN OF ITS AXIAL LOAD



TYPICAL ARRANGEMENT OF
PROPPING FOR REPAIR OF BEAM

NOTE:

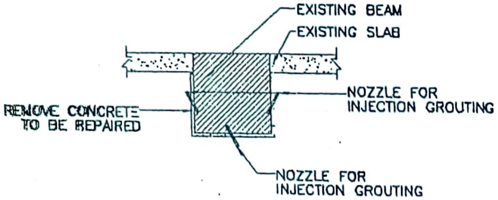
1. FOR PROPPING/SUPPORTING ANY COLUMN AT TOP, LOWEST OR INTERMEDIATESTOREY'S, PROPS SHALL BE PROVIDED AT ALL STOREY'S BELOW ALL BEAMS MEETING AT THE COLUMN

2. ALL PROPS SHALL BE CONCENTRIC I.e AT THE SAME LOCATION IN ALL THE STORES

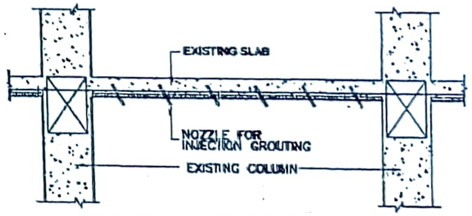
3. PROPS/SUPPORTS SHALL BE MADE UP OF BUILT UP SECTIONS OR TUBULAR STEEL OF ADEQUATE CAPACITY TO BE ABLE TO RESIST TOTAL OF BEAM REACTIONS FROM ALL UPPER STOREY'S USE OF BALLIES, BAMBOOS ETC ARE PROHIBITED.

4. SEQUENCE OF PROP FIXING WOULD BE 1-2-3-4, I.e FROM LOWEST TO UPPER MOST STOREY.

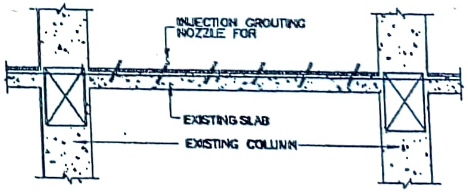
5. SEQUENCE OF PROP REMOVAL SHOULD BE 4-3-2-1.



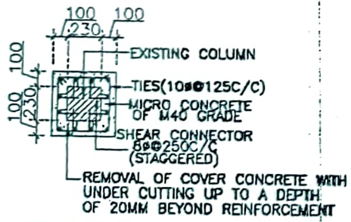
TYPICAL DETAIL OF
INJECTION GROUTING IN BEAM



TYPICAL DETAIL OF INJECTION GROUTING IN SLAB FROM BOTTOM

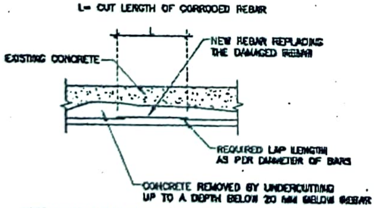


TYPICAL DETAIL OF INJECTION GROUTING IN SLAB FROM TOP



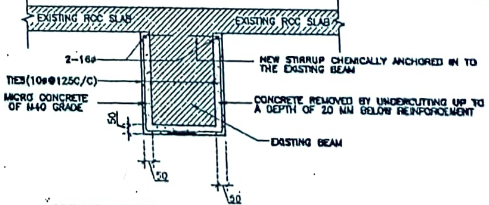
REINF. 12–16ø

COLUMN JACKETING



NOTE: STIRRUPS ARE NOT SHOWN FOR THE SAKE OF CLARITY

TYPICAL DETAILING FOR REPLACEMENT OF LOCALLY
CORRODED LENGTH OF LONGITUDINAL STEEL IN BEAM



TYPICAL DETAILING OF BEAM JACKETING

PROJECT CONSULTANT:
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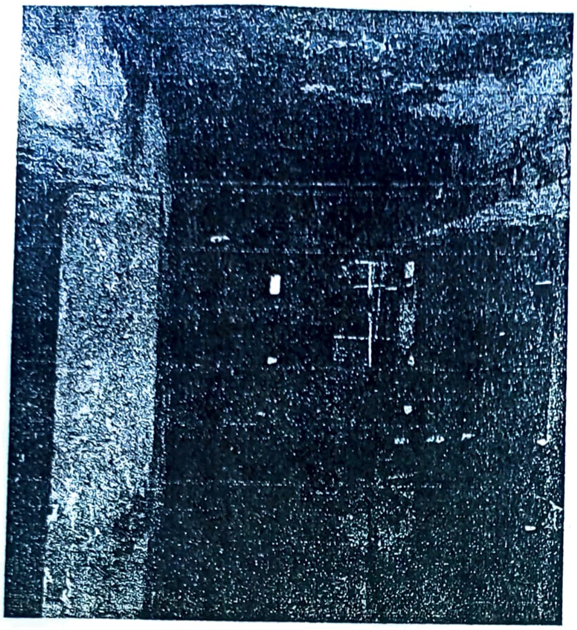
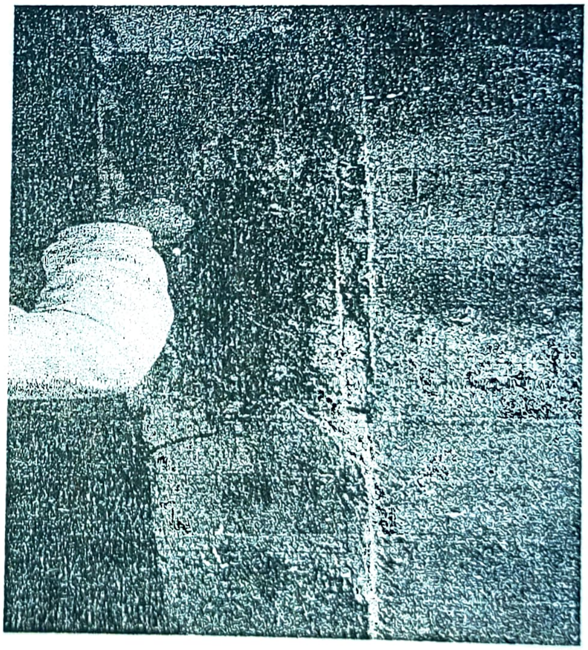
DRAWING TITLE:-
BEAM & COLUMN STRENGTHENING
MISSLIENESS DETAIL & DRAWING

[illegible]	RAJ KISHOR	SHEET NO. 64

ANNEXURE-F

SITE IMAGES





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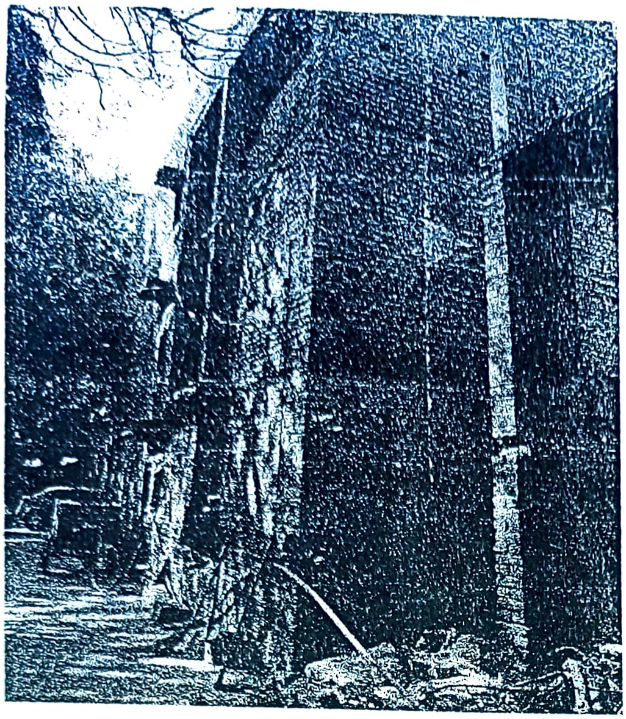

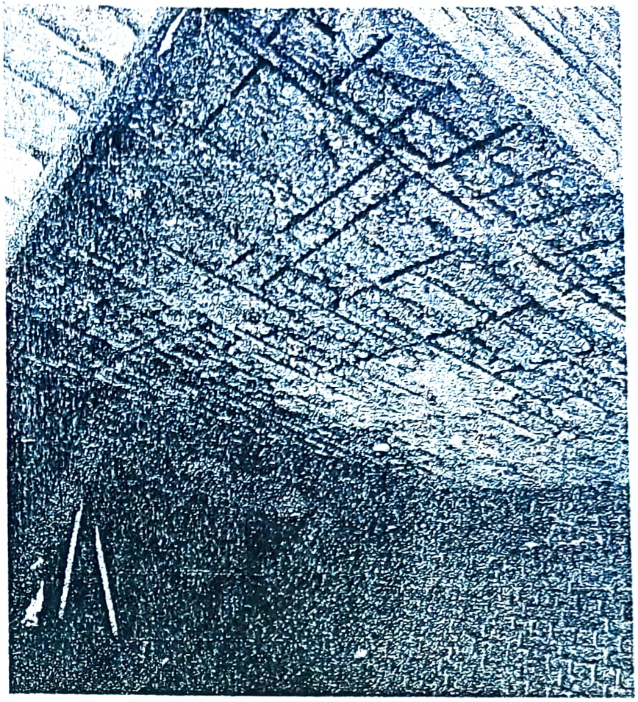

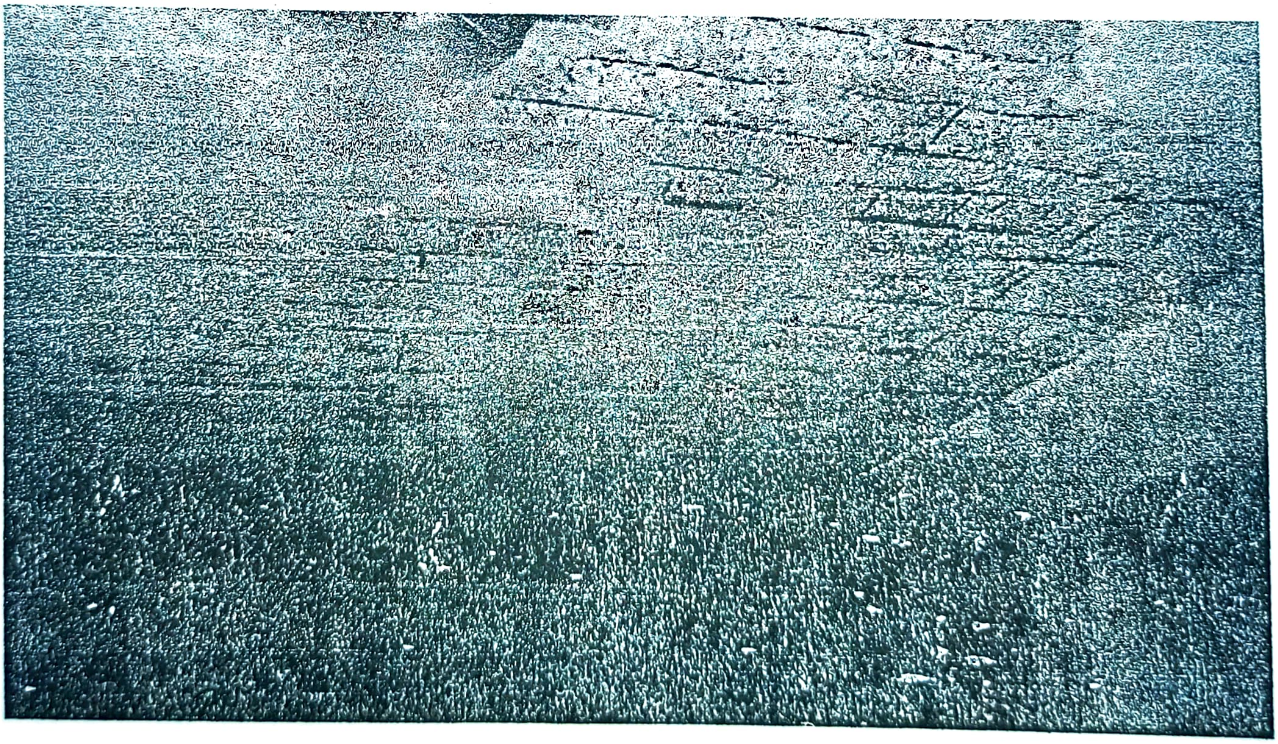

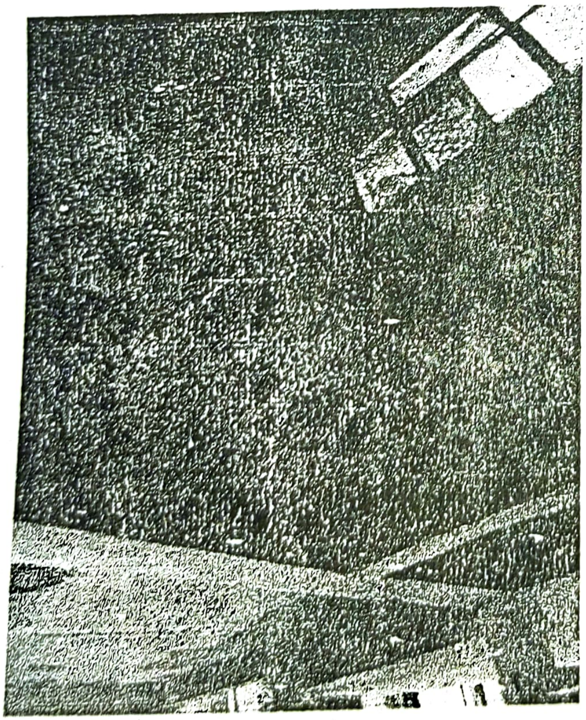

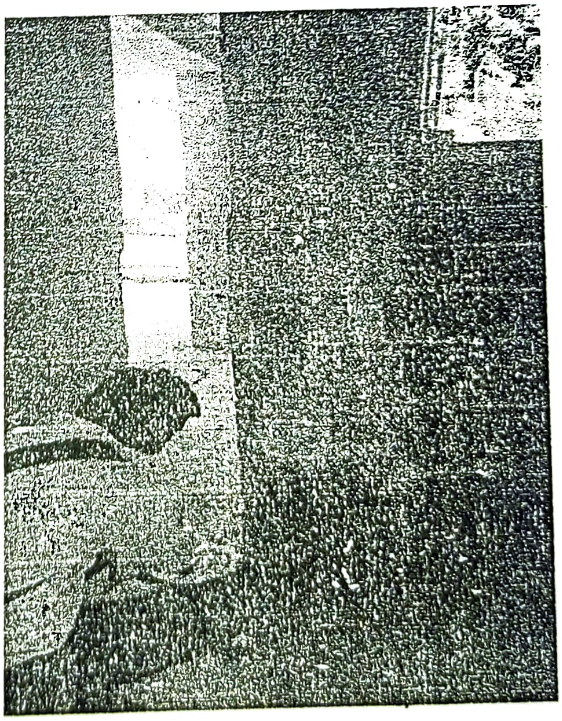

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