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SEISMIC PERFORMANCE OF REINFORCED CONCRETE BUILDINGS DURING BHUJ EARTHQUAKE OF JANUARY 26, 2001

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ABSTRACT

Reinforced concrete multi-storeyed buildings in India, for the first time, have been subjected to a strong ground motion shaking during Bhuj earthquake of January 26, 2001. A number of such buildings have been damaged in this earthquake. A team of Department of Earthquake Engineering, IITR, Roorkee conducted the damage survey of buildings. It has been observed that the principal reasons of failure are due to soft storeys, floating columns, mass irregularities, poor quality of construction material and faulty construction practices, inconsistent earthquake response, soil and foundation effect, and pounding of adjacent structures. This paper presents description of types of construction, types of damage, causes of damage in selected multi-storeyed reinforced concrete buildings, and lessons learnt from the failure. Dynamic analysis of a damaged building on floating columns and an instrumented undamaged building at Ahmedabad has been carried out. Modifications needed in the design practices to minimize earthquake damages have also been proposed.

KEYWORDS: Bhuj Earthquake, Reinforced Concrete Buildings, Damage Survey

INTRODUCTION

A massive earthquake of magnitude (M_L = 6.9 on Richter scale, M_b = 7.0, M_S = 7.6 and M_W = 7.7) occurred on the morning of 51st Republic day of India (January 26, 2001, Friday) at 08:46:42.9 hours (IST) as reported by Indian Meteorological Department (IMD), New Delhi. The epicentre of this earthquake was located near Bhachau (latitude 23.40°N and longitude 70.28°E), focal depth 25 km (Srivastav, 2001) with radius of fault area as 23 km. As per USGS NEIC, the source parameters are latitude 23.41°N and longitude 70.23°E, M_W = 7.7 and focal depth = 16 km.

The earthquake is subsequently referred to as Bhuj earthquake or Kachchh earthquake. The earthquake ranks as one of the most destructive events recorded so far in India in terms of death toll, damage to infrastructure and devastation in the last fifty years. The major cities affected by the earthquake are Bhuj, Anjar, Bhachau, Gandhidham, Kandla Port, Morbi, Ahmedabad, Rajkot, Sundernagar etc., where majority of the casualties and damages occurred (Figure 1). Every earthquake leaves a trail of miseries by way of loss of life and destruction, but it also provides lessons to the human society, particularly to engineers, architects, builders and administrators for improving design and construction practices. Various types of structures reveal weakness in the form of design and planning practices, inadequate analysis, design deficiency and even poor quality of construction. This paper describes various factors which may be responsible for the devastation of reinforced concrete (RC) buildings in Bhuj earthquake. The majority of the RC buildings surveyed by the team are in Ahmedabad, Bhuj, Gandhidham, Anjar and Bhachau. In Ahmedabad, there are approximately 750 high-rise buildings out of which three G+10 (ground + 10 storeys) buildings collapsed and 88 buildings of varying heights up to G+4 were significantly damaged. In Bhuj itself, innumerable structures collapsed and many have led to cave-in and tilting.

REINFORCED CONCRETE BUILDING CONSTRUCTION PRACTICES

Reinforced concrete construction is the most common type of construction in major cities of Gujarat, and most of the damages have occurred to these buildings. The buildings are in the range of G+4 to G+10 storeys high. The building framing system is generally moment resisting, consisting of reinforced concrete slabs cast monolithically with beams and columns on shallow isolated footing. The upper floors are generally constructed with infill wall made of unreinforced bricks, cut stones or cement concrete blocks. In major commercial cities, the ground floor/basement is often used for commercial and parking purposes, where the infill walls are omitted, resulting in soft or weak storeys. Most of the buildings have overhanging covered balconies of about 1.5 m span on higher floors. The architects often erect a heavy beam from the exterior columns of the building to the end of the balcony from the first floor onwards. A peripheral beam is provided at the end of erected girder to create more parking spaces at the ground floor and to allow more space on the upper floors. The upper floor balconies or other constructions are constructed on the peripheral beams. The infill walls, which are present in upper floors and absent in the ground floor, create a floating box type situation (Figure 2).

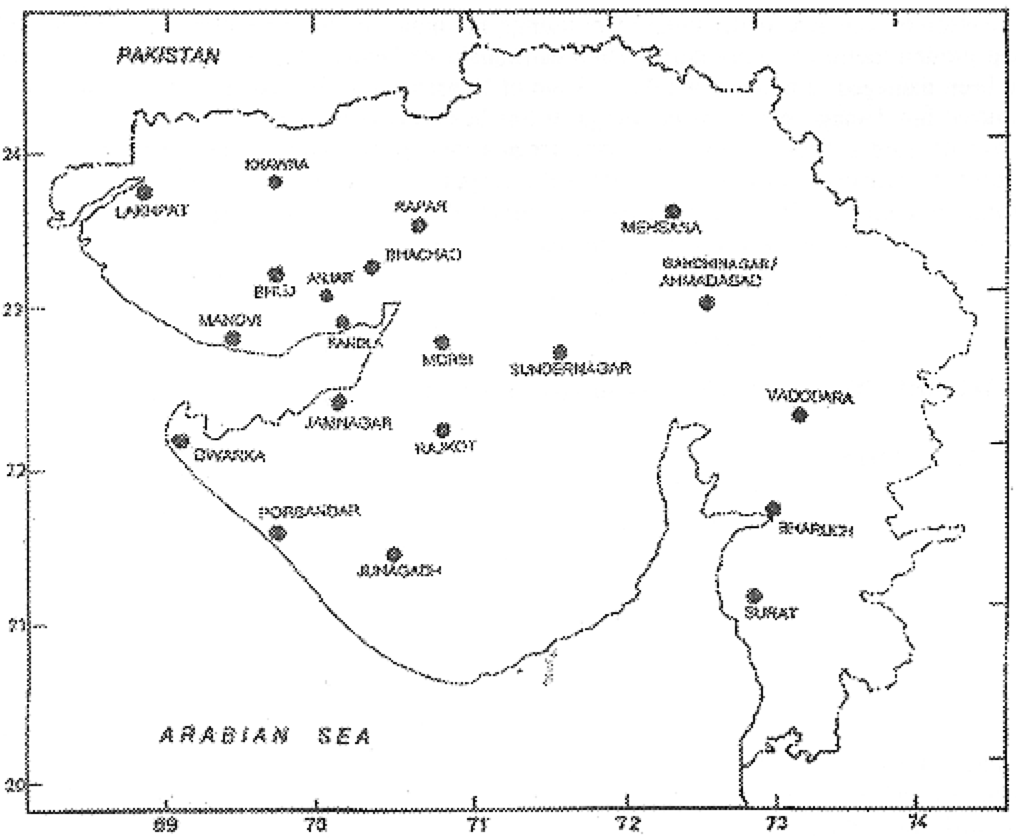


Fig. 1 Map showing the major cities of Gujarat affected by Bhuj earthquake, January 26, 2001

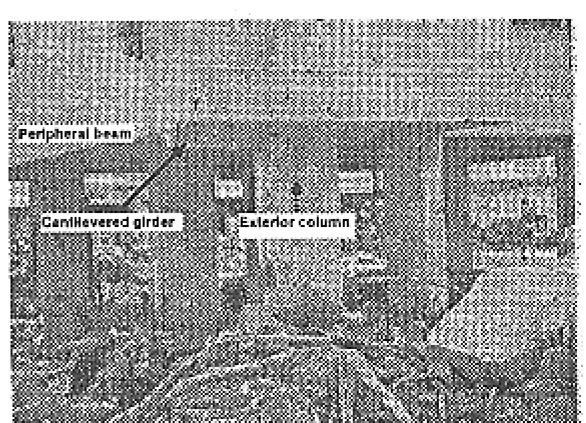


Fig. 2 Floating box construction in residential building in Ahmedabad (Goel, 2001)

The plan dimensions of building vary considerably, ranging from 10 m x 25 m or more in Ahmedabad. Storey heights remain typically between 2.7 m to 3 m, except for the lowest storey, which may be as high as 3.5 m to 4.5 m. The lift cores in the multi-storeyed buildings are generally provided in the central portion of the building.

Beam spans generally vary from 2 to 5 m, owing to irregular column spacing. In many buildings, beam reinforcement consists of three to four longitudinal bars of 12 to 16 mm in diameter. Transverse stirrups are usually 6 to 8 mm in diameter placed at a spacing of 20 to 25 cm, and ends of the stirrups are usually terminated with 90^0 hooks.

Columns in most of the buildings are of uniform sizes along the height of the buildings, with marginal change in the grade of concrete and reinforcement at ground floor. It is apparent that the columns are designed only for axial load, without considering the effect of framing action and lateral loads. The ground floor columns are not cast up to the bottom of the beam, and a gap of 200 mm to 250 mm is left, called as "topi", to accommodate the beam reinforcement, which makes the construction more vulnerable (Figure 3). Due to congestion of reinforcement in this region, the compaction of concrete is not properly done which results in poor quality of concrete and honeycombing.

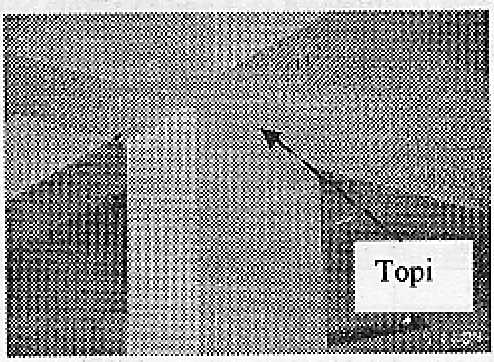


Fig. 3 Typical construction detail of "topi"

Columns often have rectangular cross-sections, with typical dimensions of 25 cm wide and 60 cm to 80 cm long. Longitudinal reinforcements consist of two rows of four to six bars of 12 to 18 mm diameter. The longitudinal reinforcement ratio is generally between 1 to 2% of gross cross-sectional area. Transverse reinforcements consist of a single hoop of 6 mm to 8 mm diameter, having 90^0 hooks, spaced at 20 to 25 cm, and terminated at the joints. The longitudinal reinforcement is often lap-spliced just above the floor slab. The spacing of transverse reinforcement over the lap splice is same as elsewhere in the column rather than being closely spaced. There is no sign of special confinement reinforcement and ductile detailing in the columns. This is a faulty design practice from seismic point of view.

Roofs usually consist of horizontal reinforced concrete slabs, 10 to 12 cm thick and resting on beams, which are 50 to 60 cm deep (including the slab) and 20 to 25 cm wide. In some cases, slab is directly cast on columns. The main reinforcement in slab is of 8 mm diameter, at a spacing of 10 cm c/c, and distribution steel is of 6 mm diameter @ 15 cm to 20 cm c/c.

The foundations in private buildings generally consist of isolated footings with a depth of about 1.5 m for G+4 buildings and 2.7 to 3.5 m for G+10 buildings. In some government buildings, raft foundation has been provided. The isolated foundations have been designed assuming bearing capacity of soil as 250 kN/m^2 (Goyal et al., 2001), though the investigations after the collapse show lower value at the foundation level (200 kN/m^2). The majority of damaged buildings were founded on deep alluvium, where amplification of motion in soil seems to have caused large forces in the buildings.

It has been observed that most buildings are designed only for gravity loads and that only a few buildings are designed considering earthquake forces with ductile detailing practices. The materials used in the construction are M15 grade concrete for G+4 storeyed buildings, M20 grade concrete for G+10 storeyed buildings, and FY 415 reinforcement.

STRONG MOTION RECORDS OF BHUJ EARTHQUAKE

A network of seventeen structural response recorders (SRRs) has been deployed by the Department of Earthquake Engineering, IITR, Roorkee in different cities of Gujarat under the strong motion project sponsored by Department of Science and Technology (DST), Government of India, New Delhi. A ten-storeyed Regional Passport Office staff quarters building at Ahmedabad has also been instrumented in a separate World Bank-aided DST project. The building is instrumented with fourteen channels of recording at different floor levels, along with a tri-axial sensor (Force Balance Accelerometer) at ground floor. The measured peak ground acceleration was 0.106g, 0.08g and 0.07g in two horizontal and vertical directions, respectively. The duration of strong shaking was about 30 seconds. The peak ground acceleration estimated from SSR record at Anjar, closest to the epicentre, was 0.547g. It is expected that the acceleration in epicentral region, near Bhachau, was larger. The estimated peak ground acceleration (PGA) values at other places are tabulated in Table 1 (Chandra et al., 2002). These values are derived from the SRR records of 0.75 s period and 5% damping. A litho-logical map of the region was used to classify the sites of recording stations approximately. Accordingly, six sites are classified as rocky sites, and rest of them are classified as alluvium.

Table 1: Estimated PGA and PSA from SSR Data

S. No.	Location	Site Class	Epicentral Distance (km)	PSA (g)	PGA (g)
1.	Anjar	Rock	43.80	0.705	0.547
2.	Naliya	Rock	147.1	0.216	0.168
3.	Khambhaliya	Rock	150.2	0.065	0.050
4.	Jamjodhpur	Rock	166.0	0.152	0.118
5.	Junagarh	Rock	216.0	0.063	0.049
6.	Amreli	Rock	225.4	0.066	0.051
7.	Kandla	Alluvium	53.20	0.571	0.333
8.	Niruma	Alluvium	97.00	0.650	0.379
9.	Dwarka	Alluvium	187.8	0.058	0.029
10.	Porbandar	Alluvium	206.8	0.248	0.144
11.	Ahmedabad	Alluvium	238.0	0.230	0.134
12.	Cambay	Alluvium	266.0	0.041	0.024
13.	Anand	Alluvium	288.0	0.061	0.036

PSA – Pseudo Spectral acceleration; PGA – Peak Ground Acceleration

STRUCTURAL RESPONSE OF INSTRUMENTED BUILDING

A ten-storeyed Regional Passport Office staff quarters building at Ahmedabad has been instrumented with 14 channels of acceleration sensors. The instrumentation consists of two tri-axial force balance accelerometers placed at the ground floor and roof of the building. At intermediate floors, 3rd, 5th, 7th, and 9th, two uni-axial force balance accelerometers have been placed near the beam-column joint in horizontal and transverse directions of the building. All these transducers (acceleration sensors) have been cabled to a recorder located in control cabin at the ground floor. The recorder is also equipped with a GPS to synchronise the real time clock and with a modem to establish remote connection through telephone line. The entire instrumentation is powered by 12V 65 AH battery, which is charged through mains as well as through solar cells. Fourteen acceleration time histories of 133.53 sec duration at 200 samples per second were recorded at these locations (Kumar et al., 2002).

These time histories were processed and corrected for sensor and other corrections. The acceleration time histories obtained at the ground floor of instrumented building are shown in Figures 4(a), 4(b) and 4(c). Dynamic analysis has been carried out with a view to compare the recorded and analytical responses of the instrumented building. 3D space frame model of the building is shown in Figure 5.

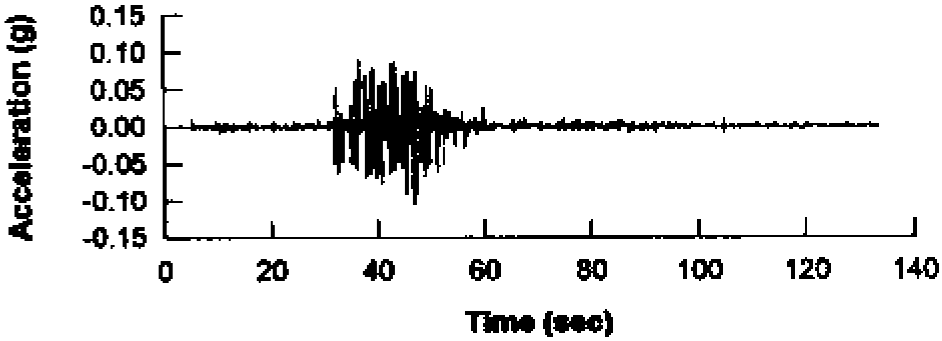


Fig. 4(a) Longitudinal acceleration time history recorded at Ahmedabad, N78^{0}E component

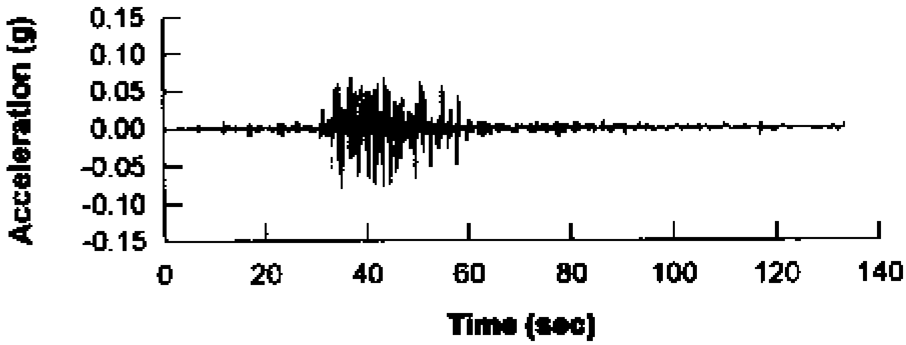


Fig. 4(b) Transverse acceleration time history recorded at Ahmedabad, N12^{0}W component

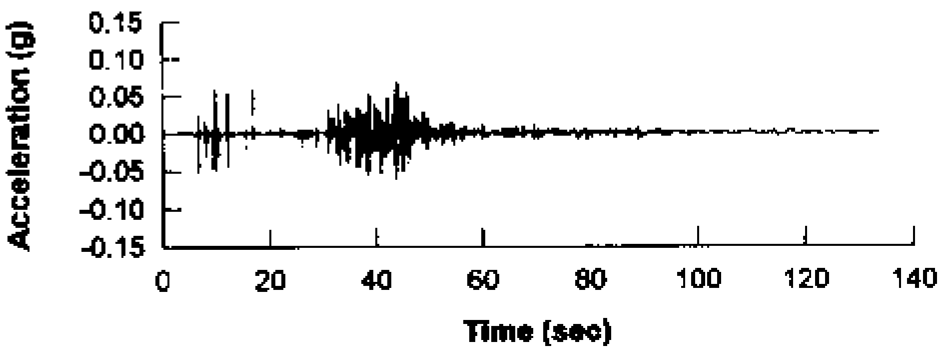


Fig. 4(c) Vertical acceleration time history recorded at Ahmedabad

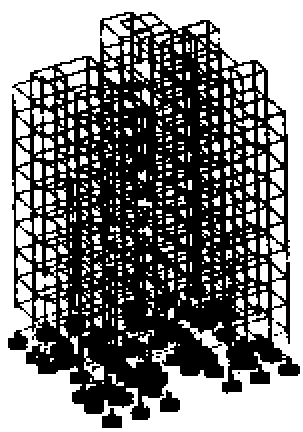
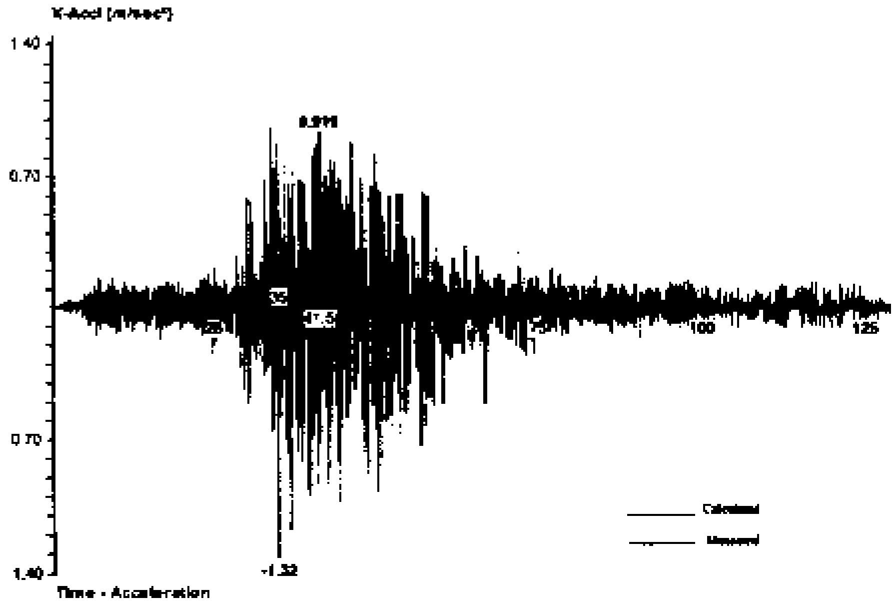
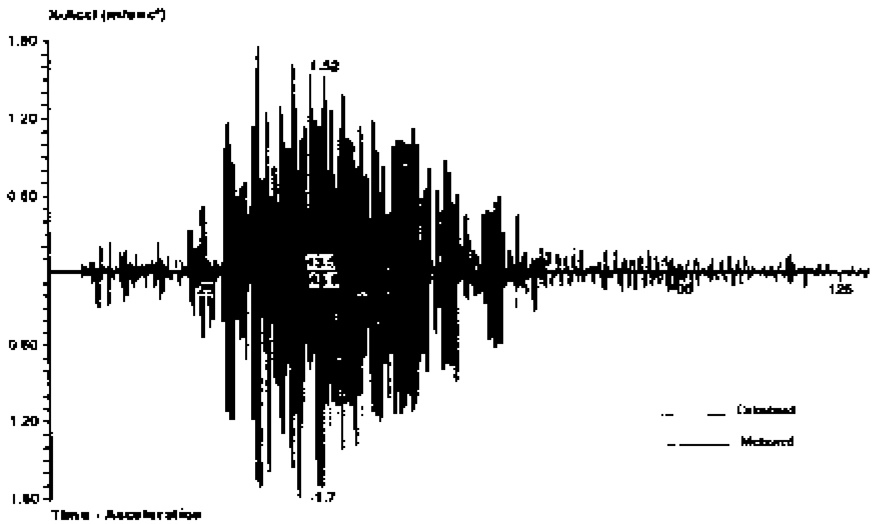


Fig. 5 Isometric view of 3D model of Regional Passport Office staff quarters building

The beam and column are idealized as 3D beam element, while the floor and slab are modelled as a 4-noded plane stress element. The staircase is modelled as truss element, and landing slab is modelled as a 4-noded plane stress element. The dead weight of machine room and water tank at the roof are lumped at appropriate nodes. The stiffness of infill walls is ignored, and only its mass is lumped at the relevant nodes appropriately. The live load on various floors is considered to be 25% of design live load. Free vibration and time history analyses have been carried out for the building. The vibration in the first mode is predominant in the longitudinal direction and has the natural frequency of 0.762 Hz, whereas second mode shape in the transverse direction is having natural frequency of 0.982 Hz. The time history analysis is carried out by applying the main shock of recorded motion at the ground floor of the building. All the three components are simultaneously applied at all the base nodes. Comparisons of analytically determined and measured peak values of accelerations at the nodes where sensors are installed, have been made and are given in Table 2. Figure 6 shows the comparison of measured and calculated acceleration time histories of 3rd and 9th floor of the building in the longitudinal direction. The comparison of peak values of calculated and measured acceleration shows reasonable matching.



(a) Response at 3rd floor in longitudinal direction



(b) Response at 9th floor in longitudinal direction

Fig. 6 Comparison of measured and calculated acceleration time histories

Table 2: Comparison of Analytical and Measured Values of Acceleration at Different Floor

Floor	Direction	Duration (s)	Peak Acceleration (cm/sec^2)	
			Analytical	Measured
9th floor	Z	47.15	361	317
	X		170	177
7th floor	Z	47.15	240	203
	X		126	126
5th floor	Z	47.15	255	157
	X		124	98.75
3rd floor	Z	47.15	132	94.4
	X		202	124

SEISMIC EVALUATION OF A DAMAGED RC BUILDING

Seismic evaluation of a typical 4 storeyed residential building at Ahmedabad has been carried out on the basis of capacity and demand ratios (C/D) of various structural members. The purpose of the study is to demonstrate the inadequacy of reinforcement in the member and size of sections. The capacity of the member has been determined on the basis of IS 456: 2000 and SP-16: 1984. The demand in the member has been worked out by a linear time history analysis of building, for the motion recorded at the base of Regional Passport Office staff quarters building as shown in Figure 4.

1. Description of the Building

A residential 4 storeyed building, which was severely damaged in the earthquake, has been considered for the study. The typical floor plan of the building is shown in Figure 7(a). The plan dimensions are 25 m x 10 m, with staircases in the central portion of the building. It is a moment-resisting framed building with unequal column spacing. The details of beam, column with foundation, slab and staircase are shown in Figure 7(b). The column sections are of uniform size throughout the height of the building. The ground floor columns are in M20 grade concrete, while the upper floor columns are in M15. The longitudinal reinforcement in ground floor and first floor columns consists of 12 bars of 12 mm diameter, enclosed by 6 mm diameter hoops at a spacing of 17.5 cm. The reinforcement in the upper floor columns is reduced to 8 in number, with the same hoop spacing. Isolated footings have been provided without tie beams. The peripheral beam section is 35.50 cm x 11.5 cm in size, with 5 bars @ 12 mm diameter at the bottom and 2 bars @ 8 mm diameter at the top, and with 6 mm diameter hoop at a spacing of 15 cm c/c. The RC slab is 11.5 cm thick, with main steel of 8 mm diameter at spacing 15 cm c/c and distribution steel of 6 mm diameter at a spacing of 20 cm c/c.

2. Estimation of Capacity and Demand of the Members

The capacities of the members (beams and columns) are estimated according to the limit state method of IS 456: 2000 on the basis of available reinforcement details and sectional properties. In beams, moment capacities are computed at the mid-span sections of members while the shear force capacities are computed at the ends only.

In columns, axial and biaxial moment capacities are estimated. Seismic demands of the members are determined from a linear time history analysis of building. The half-portion building, which is symmetric about the staircase, is idealized as a 3D space frame model (Figure 8a). The beams and column are idealised as 3D beam element and the slab is modelled as a 4-noded plane stress element. The stiffness of infill walls is ignored and only its mass is lumped at the relevant nodes appropriately. The live load on various floors is considered to be 25% of design live load, i.e., 1.5 kN/m^2 is lumped at all the nodes of the respective floor. The input motion is the recorded time history at the ground floor of Regional Passport Office staff quarters building at Ahmedabad for the main shock. The first mode of the building (Figure 8b) is torsional mode with fundamental period as 0.548 s, whereas in the second mode, the vibration is predominant in transverse direction with period of 0.492 s. The C/D ratios of the typical columns of building are tabulated in Table 3. It manifests that most of the members have C/D ratios less than 1 and are susceptible to damage.

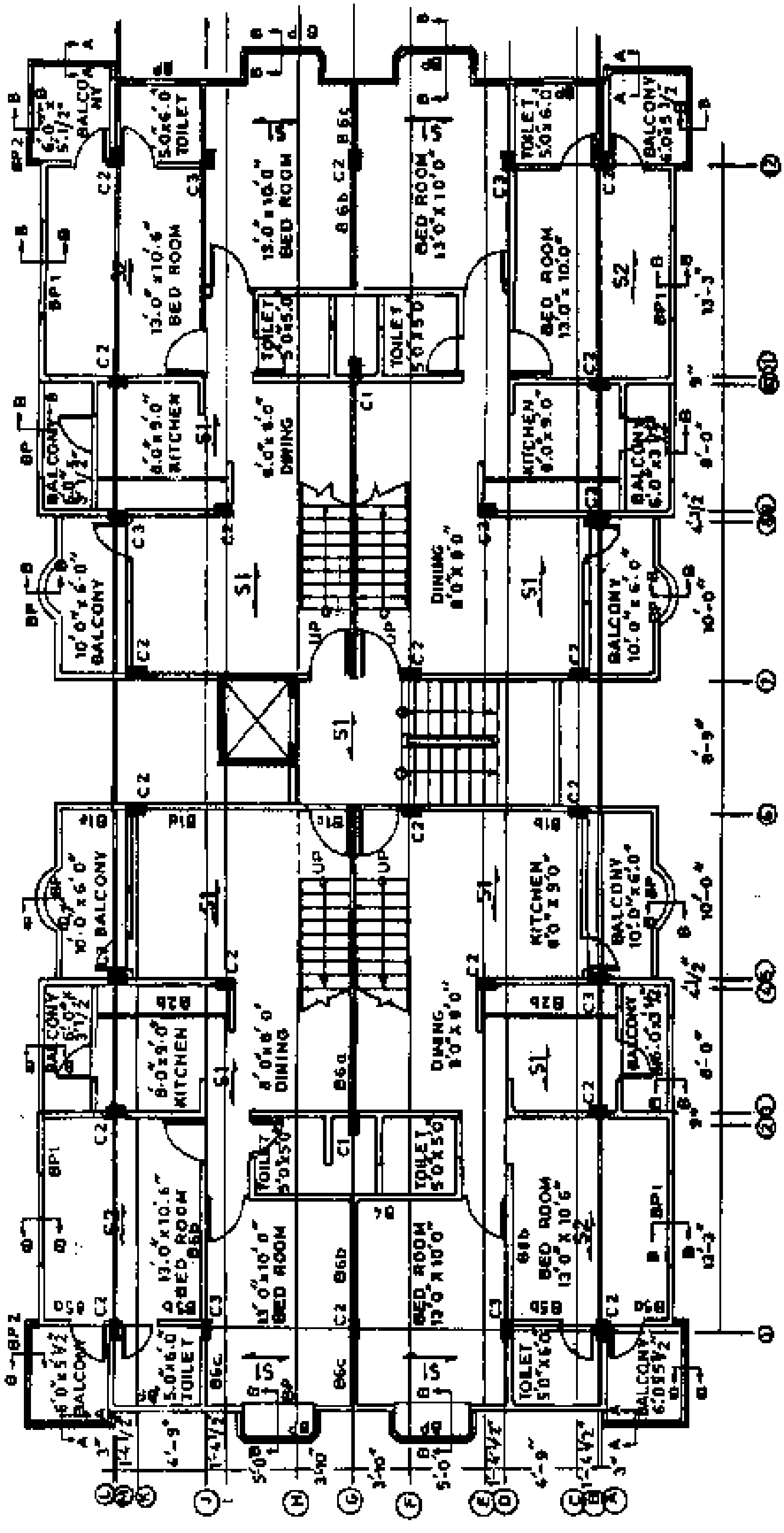


Fig. 7(a) Typical floor plan of a residential building (G+4) in Ahmedabad

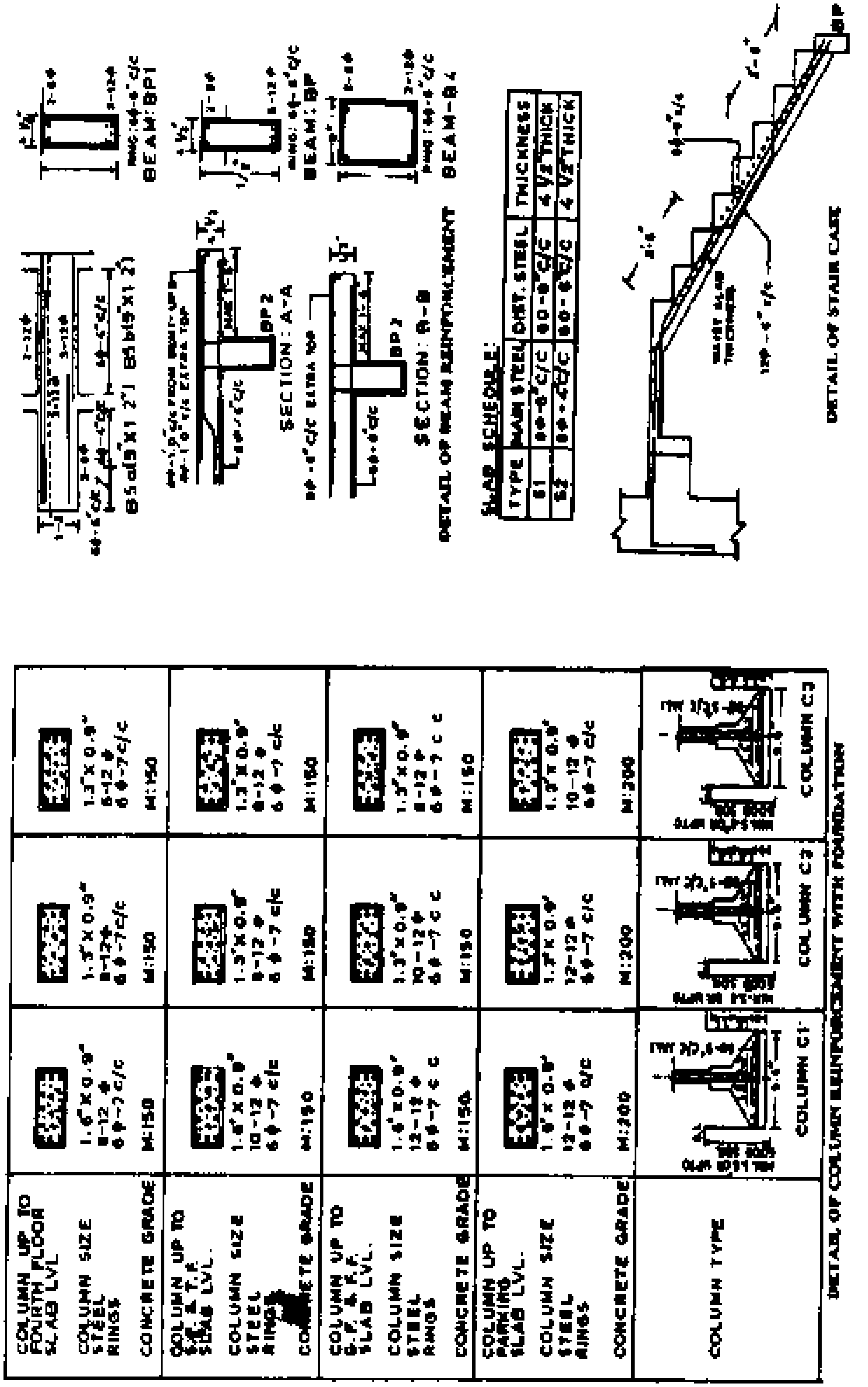
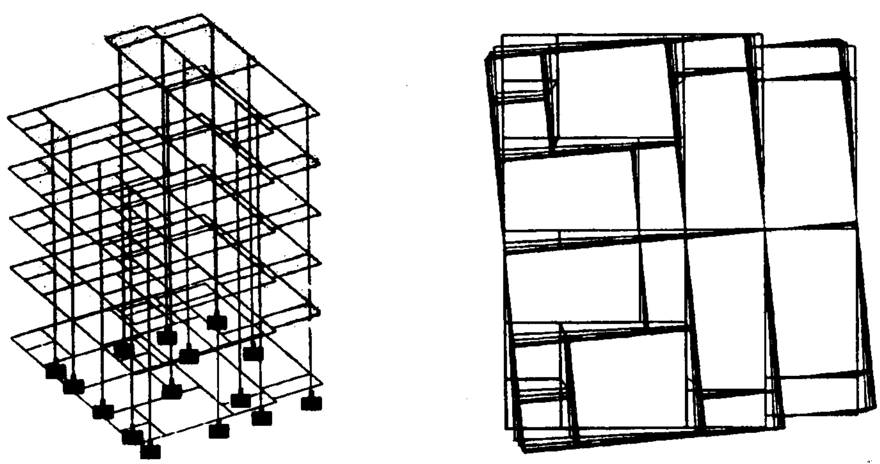


Fig. 7(b) Reinforcement detailing in structural members of a typical residential building (G+4) in Ahmedabad

Some of the design deficiencies which are clearly manifested are: the grade of concrete used in columns is M15, while the recommended value is M20, and isolated foundations with no tie beams. There is no sign of special confining reinforcement in any of the structural members.

Table 3: Capacity and Demand (C/D) Ratio of the Building Columns

	C/D ratio of the columns of the building								
Column designation	C1			C2			C3		
Member forces	P_u	M_{ux}	M_{uy}	P_u	M_{ux}	M_{uy}	P_u	M_{ux}	M_{uy}
Columns below ground floor	0.85	1.58	2.02	1.20	1.01	4.49	1.26	1.41	5.23
Columns of ground floor	0.85	0.44	1.15	1.12	0.56	2.69	1.18	0.61	1.32
Columns of first floor	1.09	0.50	1.22	1.56	1.00	8.68	1.61	1.13	2.27
Columns of second floor	1.48	0.67	1.47	2.36	1.12	13.98	3.25	1.55	2.04
Columns of third floor	2.44	0.85	1.96	4.89	1.34	18.14	4.87	1.70	2.50
Columns of fourth floor	7.86	1.61	2.88	6.05	1.01	3.40	8.41	1.74	5.82



(a) 3-D model of a floating type RC building at Ahmedabad

(b) First mode shape of the building in plan – a torsional mode

Fig. 8 Isometric view of 3D model of building and its first mode shape

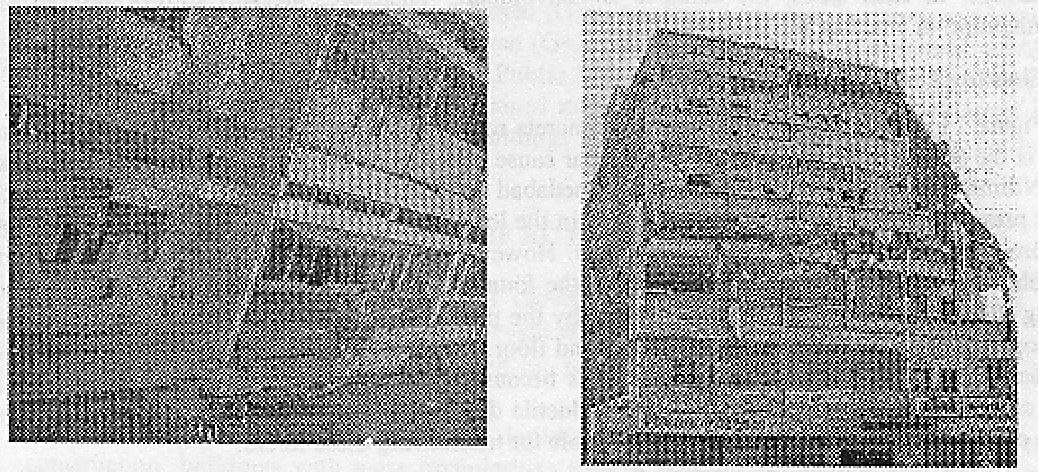
CAUSES OF DAMAGE IN REINFORCED CONCRETE BUILDINGS

Reinforced concrete buildings have been damaged on a very large scale in Bhuj earthquake of January 26, 2001. These buildings have been damaged due to various reasons. Identifying of a single cause of damage to buildings is not possible. There are combinations of reasons, which are responsible for multiple damages. It is difficult to classify the damage, and even more difficult to relate it in quantitative manner. This is because of the dynamic character of the seismic action and the inelastic response of the structures. In spite of all the weaknesses in the structure, either due to code imperfections or error in analysis and design, the configuration system of the structure and proportioning and detailing of structural elements play a vital role in the catastrophe. It has been observed that the causes of damage in Bhuj earthquake are more or less similar to those observed in other past earthquakes (Cassaro and Romero, 1986; EERI, 1990, 1993, 2000). The principal causes of damage to buildings are soft storeys, floating columns, mass irregularities, poor quality of material and faulty construction practices, inconsistent seismic performance, soil and foundation effect, pounding of adjacent structures and inadequate ductile detailing in structural components. These have been described in detail subsequently.

1. Soft Storey Failure

Figure 9 shows some of the examples of soft/flexible storeys and/or weak storey failure in Bhuj earthquake. The Apollo Apartment (Figure 9a) in Ahmedabad, nearly 15-20 years old where ground floor is used for parking purposes, got significantly damaged. The two blocks of this apartment at the entrance have completely collapsed, and the upper floors are resting on ground in significantly tilted condition.

Figure 9b shows the example of intermediate weak storey failure in a G+6 reinforced concrete framed building in Bhuj.



(a) Apollo Apartment at Ahmedabad; ground floor completely collapsed

(b) G+6 RC framed building at Bhuj; intermediate weak-storey failure

Fig. 9 Soft storey failures in reinforced concrete buildings

According to IS: 1893-2002 (BIS, 2002), a soft storey is defined as 'one in which the lateral stiffness is less than 70% of that in the storey immediately above or less than 80% of the combined stiffness of the three storeys above'. The essential characteristics of a "weak" or "soft" storey consist of a discontinuity of strength or stiffness occurring at the junction of first and second storey. Discontinuity thus caused because of lesser strength or increased flexibility of the first storey results in extreme deflections in the first storey, which in turn, lead to concentration of forces at the second storey connections and inelastic action.

The soft storey concept has technical and functional advantages over the conventional construction. Firstly, there is reduction in spectral acceleration and base shear due to increase of natural period of vibration of structure, as in a base-isolated structure. However, the price of this force reduction is in the form of increase in structural displacement and inter-storey drift (Figures 10(a) and 10(b)), thus entailing a significant P-Δ effect, which is a threat to the stability of the structure (Hart and Wong, 2000).

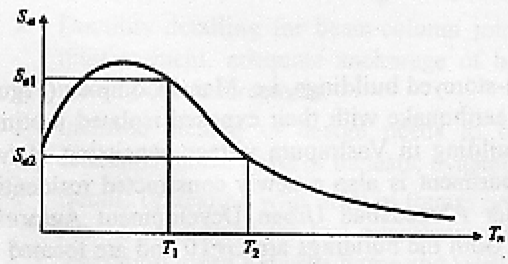


Fig.10(a) Design earthquake spectral acceleration (S_a) versus period (T_n)

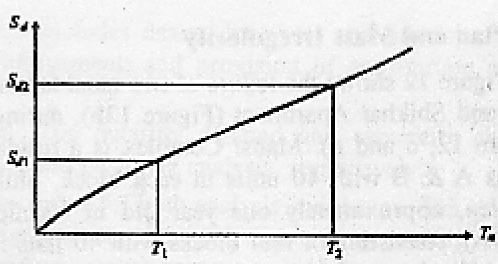


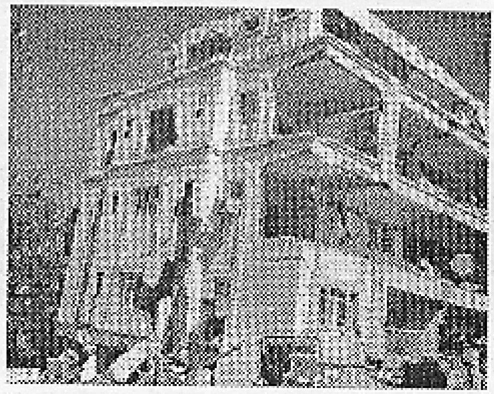
Fig.10(b) Design earthquake spectral displacement (S_d) versus time period (T_n)

Secondly, a taller first storey is sometimes necessitated for parking of vehicles and/or for retail shopping, large space for meeting room or for a banking hall. Due to this functional requirement, the first storey has lesser stiffness of columns as compared to the stiff upper floor frames, which are generally constructed with masonry infill walls. It has been observed that the damage is due to collapse and buckling of columns, especially where parking places are not covered appropriately. On the contrary, the damage is reduced considerably where the parking places are covered adequately. It is recognised that this type of failure results from the combination of several other unfavourable reasons, such as torsion, excessive mass on upper floors, P-Δ effects and lack of ductility in the bottom storey. These factors lead to local stress concentrations accompanied by large plastic deformations. In addition, most of the energy developed during the earthquake is dissipated by the ground floor columns of the soft storeys. In this process, plastic hinges are formed at the ends of columns, which transform the soft storey into a mechanism. In such cases, the collapse is unavoidable. Therefore, the soft storeys deserve a special consideration in analysis and design.

2. Floating Columns

Figure 11 shows damage in reinforced concrete residential buildings (G+4) due to floating columns. This is the second most notable and spectacular cause of failure in buildings. The 15th August Apartment and Nilima Park Apartment buildings at Ahmedabad are the typical examples of failure, in which infill walls present in upper floors are discontinued in the lower floor. This type of construction does not create any problem under vertical loading conditions. However, during an earthquake, a clear load path is not available for transferring the lateral forces to the foundation. Lateral forces accumulated in upper floors during the earthquake have to be transmitted by the projected cantilever beams. Overturning forces thus developed overwhelm the columns of the ground floor. Under this situation, the columns begin to deform and buckle, resulting in total collapse. This is because of primary deficiency in the strength of ground floor columns, projected cantilever beam and ductile detailing of beam-column joints. Ductile connection at the exterior beam-column joint is indispensable for transferring these forces.

(a) 15th August Apartment, Ahmedabad; collapse of building on floating columns

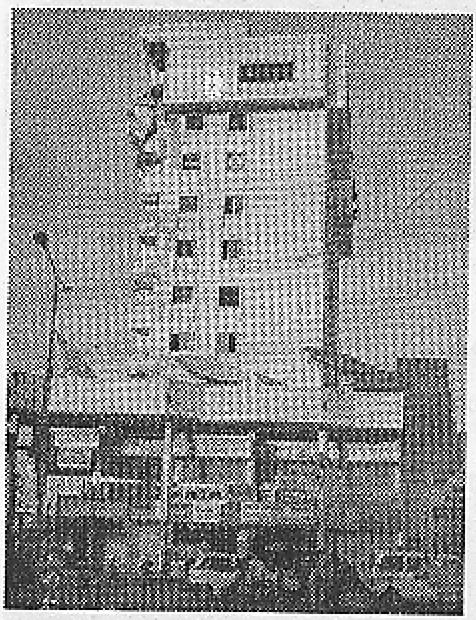


(b) Nilima Park Apartment, Ahmedabad; large scale damage in the upper floors

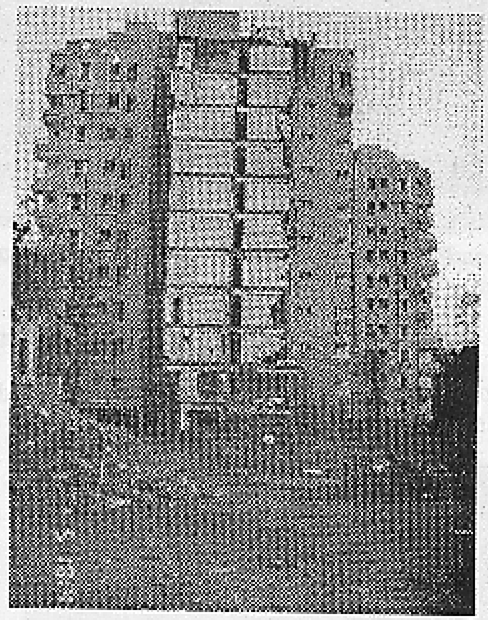
Fig. 11 Failure of reinforced concrete buildings with floating columns

3. Plan and Mass Irregularity

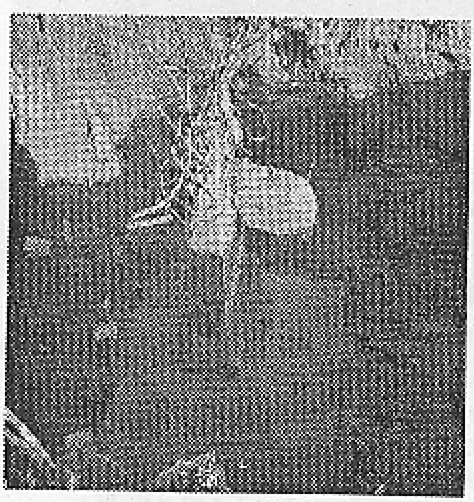
Figure 12 shows the failure of two most famous multi-storeyed buildings, i.e. Mansi Complex (Figure 12a) and Shikhar Apartment (Figure 12b), during Bhuj earthquake with their exposed isolated footings (Figure 12, c and d). Mansi Complex is a residential building in Vastrapura village consisting of two blocks A & B with 40 units in each block. Shikhar Apartment is also a newly constructed residential complex, approximately one year old in Vezalpur under Ahmedabad Urban Development Authority (AUDA), consisting of four blocks with 40 flats in each. Both the buildings are G+10 and are located in the satellite town of Ahmedabad. The plans of both the buildings are irregular. The Mansi Complex has C shaped plan, while the Shikhar Apartment has U-shaped plan with no expansion or separation joint as reported (Sinvhal et al., 2001). In A-Block of Mansi Complex, the staircase was in the central portion of the building, while in the D-Block of Shikhar Apartment, the staircase was located at the extreme end. Irregularities in plan (C and U shape), mass, stiffness and strength result in significant torsional response. These associated torsional effects may be attributed to collapse of buildings. Presence of a massive swimming pool at 10th floor, a fancy penthouse, and some rooms that were not mentioned in original plan, are also believed to be cause of failure of Mansi Complex. Excess mass leads to increase in lateral inertia forces, reduced ductility of vertical load resisting elements and increased propensity toward collapse due to P-Δ effect. Irregularity of mass distribution results in irregular response and complex dynamics. It may be inferred from the characteristic-sway mode of a building that the excessive mass on higher floors produces more unfavourable effects than that at lower floors. The other reasons that contribute to failure are: effect of soft storeys, position of service core between the two wings, wall and staircase separation, inadequate connection with slabs at each floor, and improper framing system. The column dimension in one direction is relatively high as compared to the other direction. The typical size of the column of Mansi Complex in A-block is 80 cm x 27.5 cm and 80 cm x 25 cm in B-Block. The exposed foundation of one column of collapsed portion of the building shows the isolated footing of approximate size of 2 m x 2 m in plan and 60 cm deep with no tie beams. The failure of Shikhar apartment occurred because of column shear failure, poor quality of construction material, and unsymmetrical location of lift, leading to the torsional effect.



(a) Total collapse of half portion of A-Block of Mansi Complex, Ahmedabad



(b) Collapse of D-Block of Shikhar Apartment, Ahmedabad



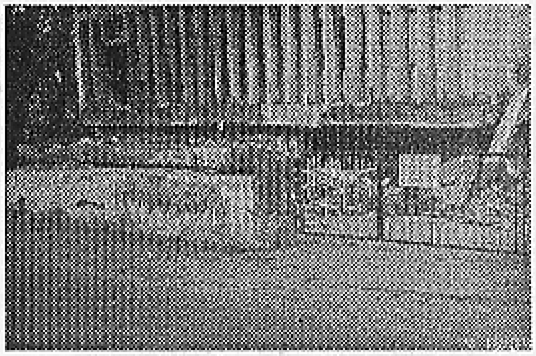
(c) Oblong isolated footing of a column; A-Block of Mansi Complex, Ahmedabad

(d) Exposed isolated footing of a column of Shikhar Apartment, Ahmedabad

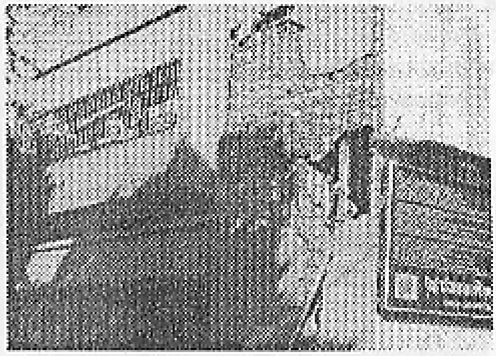
Fig. 12 Failure in reinforced concrete buildings due to structural irregularity

Poor Quality of Construction Material and Corrosion of Reinforcement

Figure 13 shows some typical examples of building failure due to poor quality of construction material. The failure of Mehta Chambers, G+3, housing morning daily newspaper 'Karnvati Express' (Figure 13a) and a RC building (Figure 13b) at Ahmedabad was due to poor quality of material and corrosion of reinforcement.



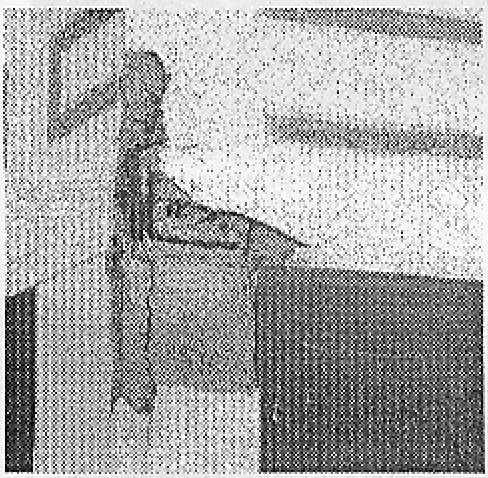
(a) Old construction, corroded reinforcement prior to earthquake, Mehta Chambers, Ahmedabad



(b) Poor quality of material, corrosion of reinforcement

Fig. 13 Damage of RC buildings due to poor quality of construction

Figure 14 shows typical examples of damage due to corrosion of reinforcement at beam column joint, slab of staircase and column face.



(a) Damage due to corrosion of steel at beam column joint

(b) Damage due to corrosion of steel at column face

Fig. 14 Damage due to corrosion of steel in reinforced concrete buildings

There are numerous instances in which faulty construction practices and lack of quality control contributed to the damage. The cement-sand ratio was dangerously high. This was experienced while striking a nail into the wall. It also appeared that recycled steel was used as reinforcement. Himgiri Apartment is now a pile of rubble as a result of poor quality of construction. Many buildings are damaged due to spalling of concrete by the corrosion of embedded reinforcing bars. The corrosion is related to insufficient concrete cover, poor concrete placement and porous concrete. Several buildings constructed about 5 to 10 years ago were damaged due to lack of quality control. It is reported that the water supply in the outer part of the city is through ground water, which is salty in taste, and the same water was used in preparing the concrete mix for construction. The presence of salts may have also affected the quality of concrete (Goel, 2001).

5. Pounding of Buildings

Although the number of buildings damaged by pounding is small, there are few examples in which the primary cause of damage in buildings is hammering of adjacent buildings. Anand building, G+5, (Figure 15) at Bhuj is an example of pounding failure.

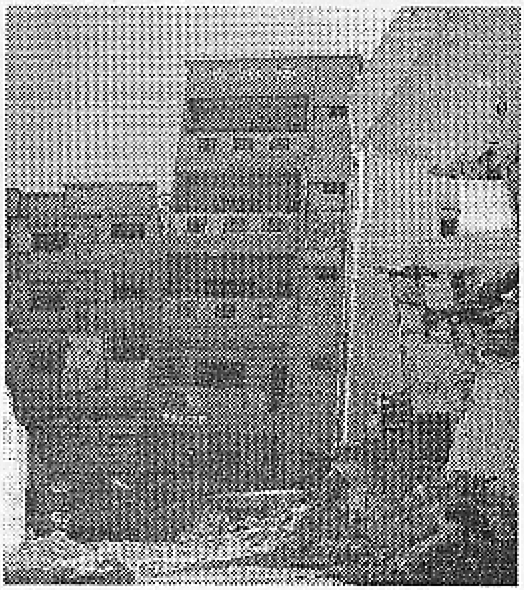
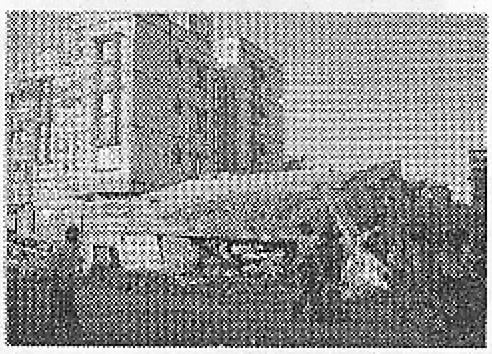
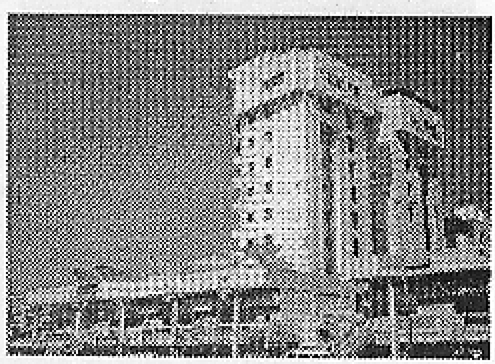


Fig. 15 Anand building, Bhuj; damage resulting from pounding

Pounding is the result of irregular response of adjacent buildings of different heights and of different dynamic characteristics. When the floors of adjacent buildings are at different elevations, the floors of each building act like rams, battering the columns of the other building. When one of the buildings is higher than the other, the building of lower height acts as a base for the upper part of the adjacent taller building. The low-height building receives an unexpected load, while the taller building suffers from a major stiffness discontinuity at the level of the top of the lower building. Pounding may also occur because of non-compliance of codal provisions, particularly for lateral and torsional stiffnesses, and cumulative tilting due to foundation movement. Damage due to pounding can be minimized by drift control, building separation, and by aligning floors in adjacent buildings.



(a) Swaminarayan school building collapsed while the adjacent building suffered minor damages



(b) Collapse of A-Block of Mansi Complex while B-block suffered minor damage

Fig. 16 Failure of reinforced concrete buildings due to different earthquake responses

6. Inconsistent Seismic Performance of Buildings

It is evident that the earthquake did not affect all the structures uniformly. The dynamic characteristics of buildings are one of the predominant factors. The severity of damage varied dramatically, with total collapse of buildings in some cases to minor damage in nearby buildings.

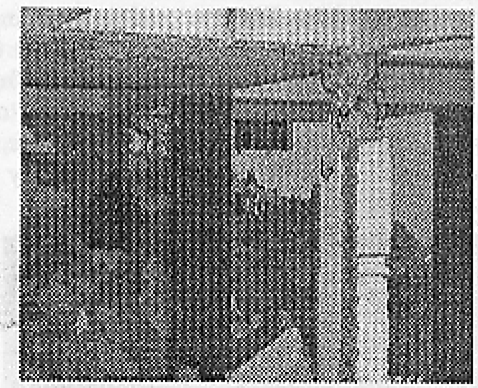
Swaminarayan Higher Secondary School in Mani Nagar at Ahmedabad, a four storeyed RC building, collapsed (Figure 16a), while nearby buildings suffered minor damages. Similarly, B-Block of Mansi Complex in satellite town sustained only minor damage, while the adjacent half-portion of the A-Block completely collapsed (Figure 16b).

A multi-storeyed RC building, under construction, across the road from Shikhar Apartment escaped damage, while D-Block of Shikhar Apartment collapsed. In some cases, the buildings appeared to be identical, but the degree of damage varied significantly. Possible explanations for such behaviour could be workmanship, detailing practices, quality of material, design, etc.

More than two-thirds of reinforced concrete buildings, which collapsed, were recently constructed. Shradha Apartment, a housing society in Ahmedabad's posh Fatehpura area barely six months old, came down. Other buildings in the same area, of less than two years vintage, have also collapsed. It has also been observed that most of buildings that collapsed, lie along the old path of Sabarmati river passing through the city. The south part of the city, especially the Mani Nagar area, where majority of damages were observed, falls between two lakes, thus indicating the presence of either poor soil strata or possible construction on reclaimed land.

DAMAGE TO STRUCTURAL ELEMENTS

Figure 17 shows failures in reinforced concrete columns in reinforced concrete buildings. Oblong cross-section, a space left at the top of column called 'topi' during casting, and relatively slender column sections compared with beam sections are the main structural defects in columns. These columns are neither designed nor detailed for ductility. Lack of confinement due to large tie spacing, insufficient development length, inadequate splicing of all columns bars at the same section, and hook configurations of reinforcement do not comply with ductile detailing practices. Figure 17 shows the failures at the top and bottom of the column due to poor quality of concrete, inadequate spacing of ties in the critical areas, and the presence of strong beams.



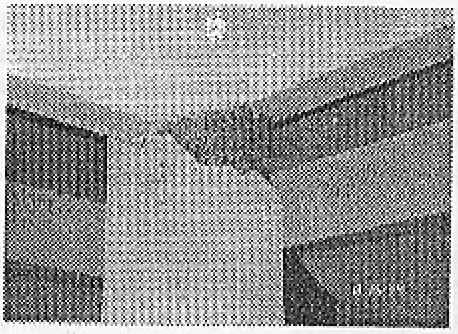
(a) Cracking and spalling of concrete in first storey column

(b) Base of first storey column with widely-spaced ties and spalled concrete

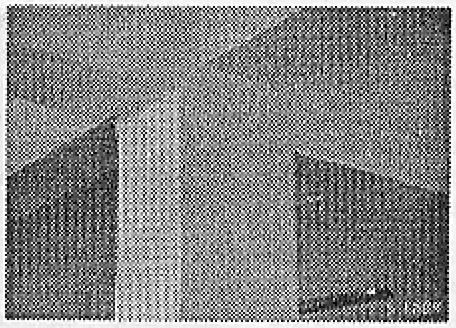
Fig. 17 Typical failure of columns in reinforced concrete buildings

Crushing of the compression zone is manifested first by spalling of the concrete cover to the reinforcement; subsequently, the concrete core expands and crushes. This phenomenon is usually accompanied by buckling of bars in compression and by hoop fracture. The opening of the ties and the disintegration of concrete lead to shortening of the column under the action of axial force. This type of damage is serious, as the column not only loses its stiffness but also loses its ability to carry vertical loads (Penelis and Kappos, 1997).

Buildings which were inspected during the team's visit, have been found with little evidence of failure of beams. There are numerous cases in which the beam-column joints of multi-storeyed buildings have been damaged. Figure 18a shows an example of damage at beam-column joint in a reinforced concrete building. One typical feature of joints constructed in RC buildings is shown in Figure 18b, where beams of different cross sections meet at the column faces at the same floor level. Inadequacy of reinforcement in beam-column joints, absence of confining hoop reinforcement, inappropriate location of bar splices in columns, are common causes of failures of beam-column joints.



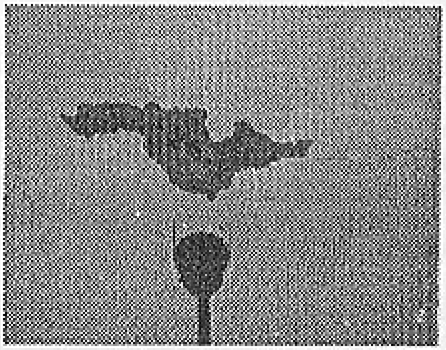
(a) Minor damage at beam-column joint



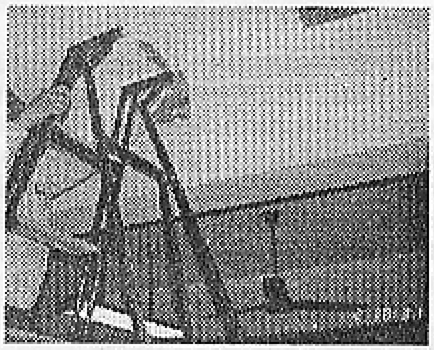
(b) Detail of beam-column joint

Fig. 18 Beam-columns joints in reinforced concrete buildings

Figure 19 shows cracking of reinforced concrete slab and beam-slab joints in buildings. It is mainly due to widening of existing micro cracks, which were formed either because of bending action or temperature/shrinkage. These cracks are further widened and become visible due to strong ground shaking. Damage in slab is generally not considered to be dangerous for the stability of the structure. However, it creates serious functional and aesthetic problems.



(a) Cracks at ceiling - existing micro cracks



(b) Cracks at slab–beam junction

Fig. 19 Damage to RC slab in reinforced concrete buildings

(a) Shear (X) cracking of masonry infill

(b) Shear failure in "captive column"

Fig. 20 Failure of infill wall and panel in reinforced concrete buildings

DAMAGE TO NON-STRUCTURAL PANEL ELEMENTS

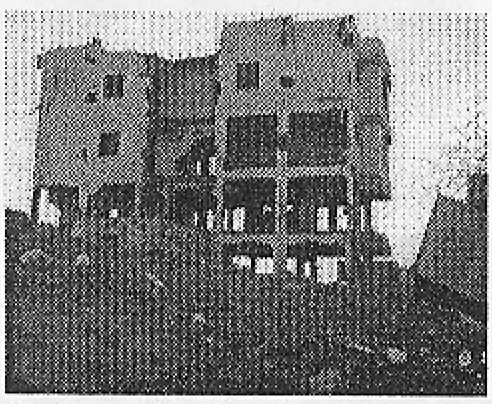
1. Damage to Infill Walls

Masonry infill walls are used as interior partitions, and as exterior walls to form part of the building envelope in multi-storeyed buildings. In general design practices in India, the strength and stiffness of infill walls are ignored with the assumption of conservative design. But this assumption is sometimes not valid, because infill walls stiffen the flexible frame, affecting the distribution of lateral loads significantly. This phenomenon leads to cracking of the infill walls and overstressing of the frames. Figure 20a shows an example of cracking of infill wall in a residential building of Oil & Natural Gas Commission (ONGC) at Ahmedabad.

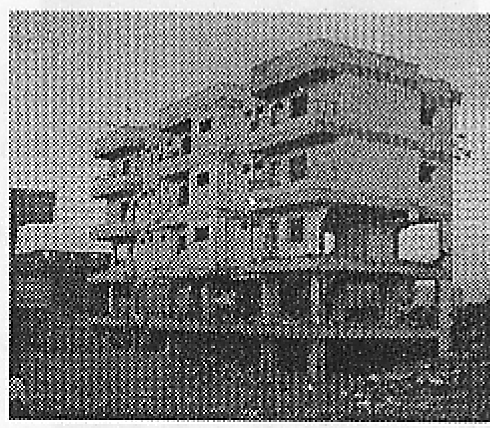
During the excitation of the structure, the reinforced concrete frame begins to deform, and initially the first cracks appear on the plaster along the line of contact of the masonry infill with the frame. As the deformation of the frame becomes larger, the cracks penetrate into the masonry, and are manifested by the detachment of the masonry infill from the frame. Subsequently, diagonal cracks (X-shaped) appear because of the strut action of the infill. To avoid this type of failure, either interaction of infill wall with the frame should be considered in design or a movable joint between infill and frame should be provided. Sometimes, perimeter infill walls are pierced with many closely spaced windows. The piers enclosed between the windows are called as 'captive' columns. The shear required to develop flexural yield in the effectively shortened column is substantially higher than the shear required for developing flexural yield of full-length column. If the designer has not considered this effect of the infill, shear failure may occur before the flexural yield (Figure 20b). The cracking in 'captive' column generally initiates from window headers and sill levels (Moehle and Mahin, 1991).

2. Damage to Exterior Walls

Figure 21 illustrates characteristic examples of damage to exterior walls that are poorly connected with the RC frame. These walls are subjected to out-of-plane vibrations. This form of construction of large exterior walls creates a weak plane around the perimeter. When subjected to intense shaking, these large un-reinforced masonry panels confined by stiff frame members have a tendency to resist large out-of-plane vibrations with little sign of distress. When the flexure strength of these panels becomes insufficient to resist these forces, the entire infill panels fail. The magnitude of damage is found to be dependent on the quality of materials and method of construction.



(a) Collapse of exterior wall due to restricted ductility of concrete frame



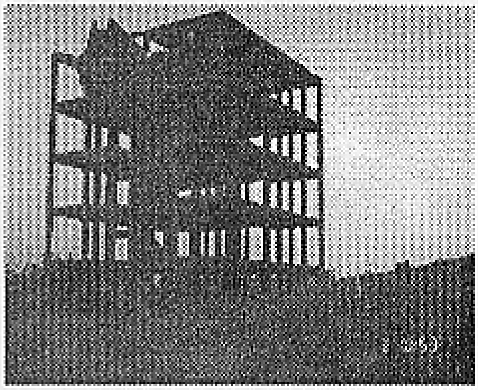
(b) Damage to walls of ground and first floor

Fig. 21 Failure of exterior walls in non-ductile concrete frame

DAMAGE TO WATER TANK AND PARAPETS

Figure 22a shows a reinforced concrete building, Prabhu Kripa at Bhuj, in which failure of water tank at the roof of the building was observed. Figure 22b shows the failure of the top portion of a bare framed reinforced concrete building under construction. Water tanks constructed at the roof level of buildings experience large inertia forces due to amplification of the ground acceleration along the height of the building. Un-reinforced concrete parapets, with large height-to-thickness ratio and not properly anchored to the roof diaphragm, may also constitute a hazard. The hazard posed by a parapet increases in direct proportion to its height above building base. This has been observed at several places.

(a) Failure of water tank in a RC frame building



(b) Top portion of the bare framed RC building under construction collapsed

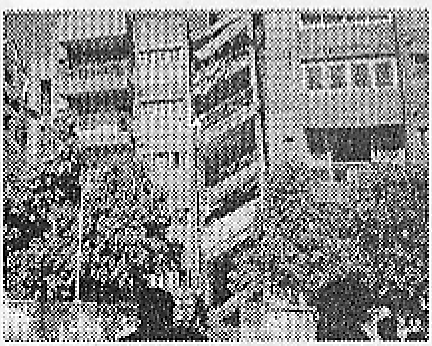
Fig. 22 Amplification effects of acceleration in RC frame buildings

DAMAGE TO VERTICAL CIRCULATION SYSTEM

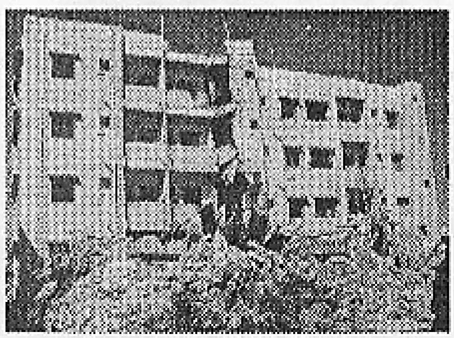
Staircases and lifts are the only means of vertical movement in building, and the staircases also serve as escape routes during an earthquake. Figures 23 and 24 show the failures of staircases and a lift core in reinforced concrete buildings.

1. Damage to Staircase

Figure 23 shows typical examples of failures of staircases in Vishram Flat, G+5, in Navarangpura, Ahmedabad and in a RC building at Bhuj, due to out-of-phase vibrations of two blocks.



(a) Strut action of staircase during out-of-phase vibration of two blocks



(b) Damage to staircase in reinforced concrete frame building

Fig. 23 Failure of staircase in reinforced concrete framed buildings

In quite a few multi-storey complexes, failure of staircase is a major cause of damage. Staircases and corridors are found to have been blocked by the failures of the unreinforced masonry enclosure walls. Many exit doors are found to be jammed due to racking of doorframes. Stairs can start acting as diagonal-bracing elements during earthquake-induced motion, and therefore, should be used with sliding joints in the seismic design of buildings. Isolation of stairs from the primary structural system may also minimise the damage to the stair system.

2. Damage to Elevator

Figure 24 shows the undamaged lift core of a building during the earthquake at Gandhidham. Elevators constitute an integral part of building, and are vulnerable to earthquakes. It is important to prevent damage to the elevators for the following reasons: (i) there is danger to the passengers trapped during the event and there are difficulties in rescue operations, (ii) undetected damage can cause substantial danger in elevators used after the earthquake, (iii) vertical circulation systems (elevators and stairs) are essential in hospitals which deliver crucial health services after an earthquake.

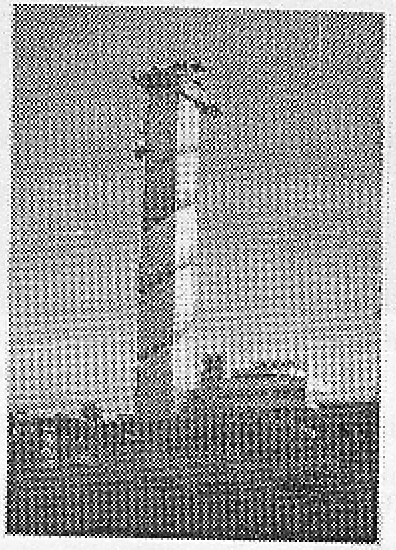
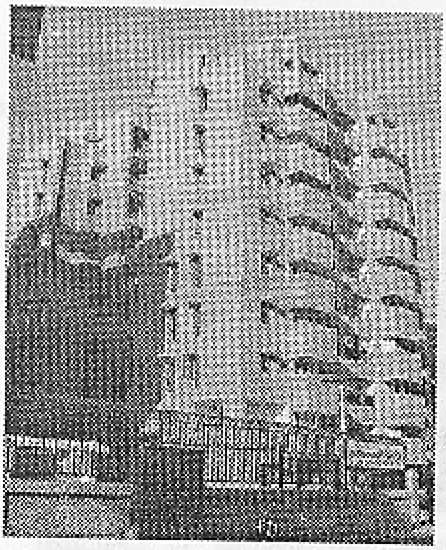
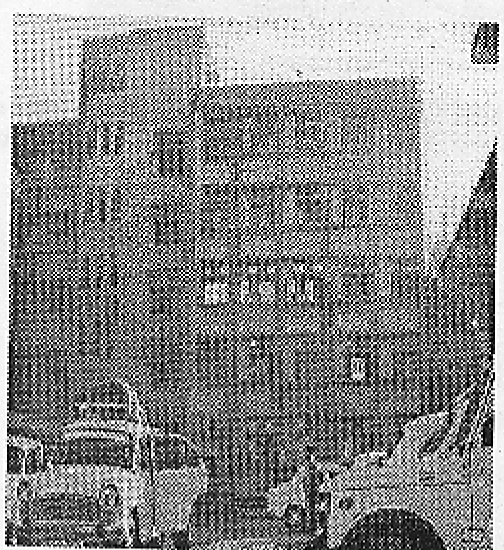


Fig. 24 Undamaged lift core of a reinforced concrete building



(a) Residential quarters for Regional Passport Office (G+9), Ahmedabad; minor cracking in filler walls



(b) Ayakar Bhawan (G+3), a RC building, Ahmedabad; diagonal and junction cracks in filler brick wall

Fig. 25 Damage in government-constructed reinforced concrete buildings

EFFECT OF EARTHQUAKE ON CODE DESIGNED STRUCTURES

The Bureau of Indian Standards (BIS) has published two codes, IS: 1893 (Part 1) (BIS, 2002) and IS: 13920 (BIS, 1993) for the earthquake-resistant design of reinforced concrete buildings. The former code deals with the determination of forces and general considerations for design of buildings, while the latter code deals with the detailing of reinforced concrete structures for ductility. The government buildings follow the design codes as a mandatory requirement. Therefore, the performance of governmental buildings in this earthquake has been relatively better on account of code compliance (Thakkar et al., 2001a, 2001b). Figure 25 shows a multi-storeyed (G+9) reinforced concrete building, residential quarters for regional passport office, and Ayakar Bhawan (G+3) RC building, with part basement, at Ahmedabad. These buildings were constructed by Central Public Works Department (CPWD) in the years 2000 and 1954, respectively. These two buildings sustained minor damage in the form of cracking of infill brick walls and non-functioning of lift. Both the buildings were in working condition after the earthquake, and were not required to be vacated.

LESSONS LEARNT FROM DAMAGES OF RC BUILDINGS

The occurrence of Bhuj earthquake has caused significant damage to multi-storeyed reinforced concrete buildings. The lessons learnt from damages are presented below:

1. The design of buildings should be based on seismic codes, IS: 1893 (Part 1) (BIS, 2002) and IS: 13920 (BIS, 1993).
2. The multi-storeyed reinforced concrete buildings with vertical irregularities like soft storey construction, buildings with mass irregularities such as massive swimming pool on the roof of the building, and buildings with floating columns should be designed on the basis of dynamic analysis and inelastic design. The ductility provisions are most important in such situations.
3. More care is necessary at the time of planning. The torsional effects in a building can be minimised by proper location of vertical resisting elements and mass distribution. Building design with strong column-weak beam can be achieved at the planning stage. The soft storey stiffness can also be controlled by an appropriate design procedure.
4. The infill construction in RC buildings should be duly accounted for in the structural analysis. The staircase connection with buildings should be made while using sliding joints.
5. Shear walls should be employed for increasing stiffness, where necessary, and be uniformly distributed in both principal directions.
6. There should be a greater emphasis on the quality of construction.

MODIFICATIONS REQUIRED IN SEISMIC CODES

Design provisions for the following should be provided in seismic codes:

1. Methods of dealing with plan and vertical irregularities in a building; this should be done on the basis of 3D mathematical modelling of the building.
2. Ductility detailing for beam-column joints, which includes design for shear and provision of shear reinforcement, adequate anchorage of beam reinforcement, and provision of appropriate space for placing of reinforcement.
3. Inelastic method of design needs to be added. The ductility demand and capacities should be estimated on the basis of inelastic dynamic analysis and pushover analysis, respectively.
4. Details of dealing with infill in frames are to be included for both analysis and design. Design of infill can be considered for two situations: either integral connection with frame by providing dowels or separation given between frame and infill by providing moveable joint.
5. Guidelines for making mathematical model of building with foundation need to be included in codes. When soil–foundation is modelled with building frame, soil springs should be provided at the base. The effect of soil amplification should be considered in the analysis.

SUMMARY AND CONCLUSIONS

The earthquake-resistant design of reinforced concrete buildings is a continuing area of research since the days earthquake engineering started not only in India but in developed countries also. In spite of that, reinforced concrete buildings are damaged for various reasons. The Bhuj earthquake of January 26, 2001 in India is a recent example, in which the reinforced concrete buildings were damaged on a large scale.

The buildings falling in the range of G+4 to G+10 storeys sustained damage. The maximum acceleration in the epicentral region at Anjar was recorded as 0.547g by a Structural Response Recorder (SRR). It has been observed that the acceleration recorded at the ground floor of the Regional Passport Office staff quarters building (G+9) of 30 m height at Ahmedabad was 0.106g, which got amplified to nearly 3 times on the roof. The measured and calculated time histories at 3rd and 9th floors of the building are closely matching. This verifies the mathematical modelling of the building. The dynamic analysis of a building on floating columns reveals that the vibration in fundamental mode is predominant in torsion.

The causes of failure which are identified after field survey are (i) soft storey failure: vertical irregularity in stiffness/strength, (ii) floating column failure: complex load path to transfer of forces, (iii) mass irregularities: eccentric loading and P-Δ effect, (iv) poor and old construction: corrosion of reinforcement, (v) pounding: hammering of adjacent buildings, (vi) design deficiency: lack of ductility and ductile detailing, and (vii) construction consideration: lack of sliding and moveable joints. These failures could be minimised by the technical awareness of earthquake-resistant design practices among engineers, architects, planners and builders.

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