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# **The Bhuj, India Earthquake of 26<sup>th</sup> January 2001**

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## **A FIELD REPORT BY EEFIT**

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January 2005

ISBN 0 901297 37 2

EEFIT gratefully acknowledges the support of its corporate members:

Arup Group  
British Geological Survey  
BNFL  
CREA Consultants  
EQE International Ltd  
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Halcrow



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# 1 Overview of the Bhuj Earthquake

Dr S P G Madabhushi, Team Leader  
University of Cambridge

## 1.1 Background

A powerful earthquake rocked the western state of Gujarat in India at 08:46 hours (Indian Standard Time) on the 26<sup>th</sup> January 2001. The earthquake was estimated by USGS to have a Moment Magnitude of 7.9. The epicentre was reported to be near a major town called Bhuj in the Kachchh region of Gujarat. Fig.1 shows the location of the epicentre. In Fig.2 the location of the state of Gujarat is highlighted on the political map of India. Initial reports in the media indicated extensive damage to the civil engineering structures followed by reports on loss of life on a massive scale. This region is known to be seismically active and has seen a major earthquake in 1819 which had a similar magnitude to the current earthquake. That earthquake occurred along the Allah Bund fault and resulted in a major uplift of the ground surface. Ground movements of up to 9m vertically and 3.5m horizontally were reported by Bilham (1998) to have occurred due to that earthquake. More recently a major earthquake of Moment Magnitude 7 occurred with its epicentre close to Anjar in 1956.

Following the news of the current earthquake, there were a large number of enquires to EEFIT from its members and practising engineers regarding a possible mission to Gujarat. Accordingly the EEFIT management committee decided to mount a mission to the earthquake affected region. A team was selected based on the expertise in various aspects of earthquake engineering of individual team members. Table 1 lists the team members, their affiliations and area of expertise. Also effort was made to include younger engineers in the team to enable them to witness earthquake damage in a major event such as this.

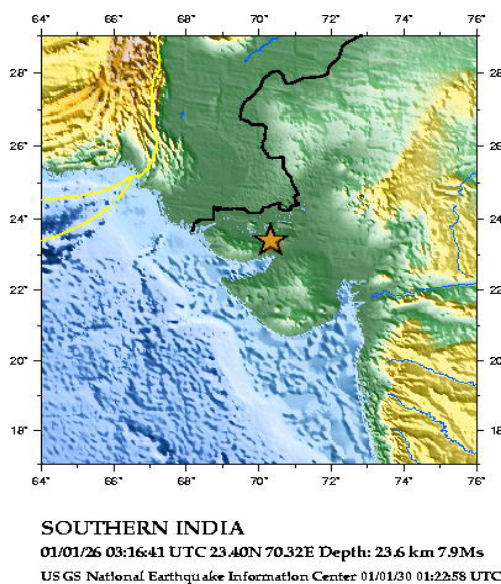


Fig.1 Location of the epicentre as reported by USGS

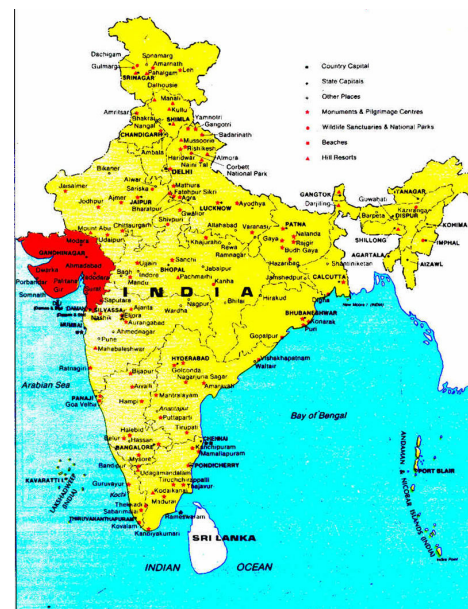


Fig.2 Location of the western Indian State of Gujarat

**Table 1.1: EEFIT Team Members and their areas of expertise**

| Team Member                              | Affiliation                                 | Area of Expertise                             |
|--|---|---|
| Dr S P Gopal Madabhushi –<br>Team leader | University of Cambridge                     | Geotechnical & Soil-Structure<br>Interaction  |
| Dr Paul Greening                         | University of Bristol                       | Structural Engineering                        |
| Mr Stuart Haigh                          | University of Cambridge                     | Geotechnical Engineering                      |
| Mr Domenico Del Re                       | Cambridge Architectural<br>Research Limited | Architectural & Structural<br>Engineering     |
| Mr Joe Barr                              | High Point Rendel                           | Bridge Engineering & Industrial<br>Structures |
| Mr Alan Stewart                          | Babtie (Glasgow)                            | Structural Engineering                        |
| Mr Andy Kirby                            | Halcrow (Glasgow)                           | Structural Engineering                        |
| Mr Andy Thompson                         | Arup (London)                               | Structural Engineering                        |
| Mr Dinesh Patel                          | Arup (London)                               | Geotechnical Engineering and<br>Geology       |

## 1.2 Extent of earthquake damage

The earthquake occurred in the early hours of Friday, 26<sup>th</sup> January 2001 at 08:46 hours Indian Standard Time (IST). This was the Republic day in India, which is a national holiday. However, schools were open due to the Republic day celebrations and several children were injured due to the collapse of school buildings.

The extent of earthquake damage was immense. The large magnitude of the earthquake combined with the poor construction quality contributed to large scale damage to the building stock and a high number of casualties. In Table 1 the outline information that was available on the extent of damage is collated.

**Table 1.2: Extent of Damage\***

| Type of damage                     | Extent                   |
|------------------------------------|--------------------------|
| Number of casualties               | 20,000+                  |
| Number of injured                  | 167,000                  |
| Estimated cost                     | 5 billion US\$           |
| Number of buildings destroyed      | ~ 300,000                |
| Number of buildings damaged        | ~ 700,000                |
| Number of earth dams damaged       | 14                       |
| Area involved in landslides        | ~ 10,000 km <sup>2</sup> |
| Area involved in soil liquefaction | ~ 10,000 km <sup>2</sup> |

\*Information collated from local newspapers

## 1.3 Objectives of the mission

The general aims of EEFIT missions are listed in the mission guide (ref). The specific aims for the mission to Gujarat, India were established prior to the mission as listed below:

- To investigate the damage to different type of buildings (multi-storied and short storied residential buildings) in Ahmedabad, in Bhuj and surrounding villages, and in Jamnagar
- To study the performance of single storied buildings constructed with local materials
- To collect seismological data of the main event of 26<sup>th</sup> January and subsequent aftershocks
- To collect geological data of the region
- To establish the geotechnical aspects of the earthquake namely liquefaction induced damage, lateral spreading etc

- To investigate the damage to infrastructure and lifelines, in particular the damage to road and rail networks serving the region
- To study the performance of industrial structures given the concentration of Indian industries of various types in the region.

The mission was accordingly organised with support from the Indian High Commission in London, State Government of Gujarat, India and Bactie Group offices in Ahmedabad. The dates for the mission were 9<sup>th</sup> February 2001 to 17<sup>th</sup> February 2001. However different members of the mission have spent time after these dates collecting additional information as was necessary. This was found to be very beneficial with experts from the team revisiting some of the damage areas and communicating with teams from other countries including the two teams from the USA.

### 1.4 Mission Route

The route followed by the mission is presented in Fig.3. The mission began with a briefing on the earthquake at Indian Institute of Technology, Bombay by the experts in Department of Geology. Following this the team travelled North to Ahmedabad where the team spent a day looking at the damage. Groups were organised from within the team so that damage in various parts of the city could be covered. Also some groups visited the local government offices to collect information on the post earthquake surveys that were carried out by local engineers on the building stock. A meeting was also organised with local government officials to discuss the post- disaster measures that were undertaken and possible course of action for repairs and remediation works.

Following this the team travelled west to Bhuj which was the major town closest to the epicentre. The team spent four days at Bhuj and travelled in groups to surrounding regions including Gandhidham and Kandla port and villages that saw extensive damage such as Lodai, Bhachau and Anjar.

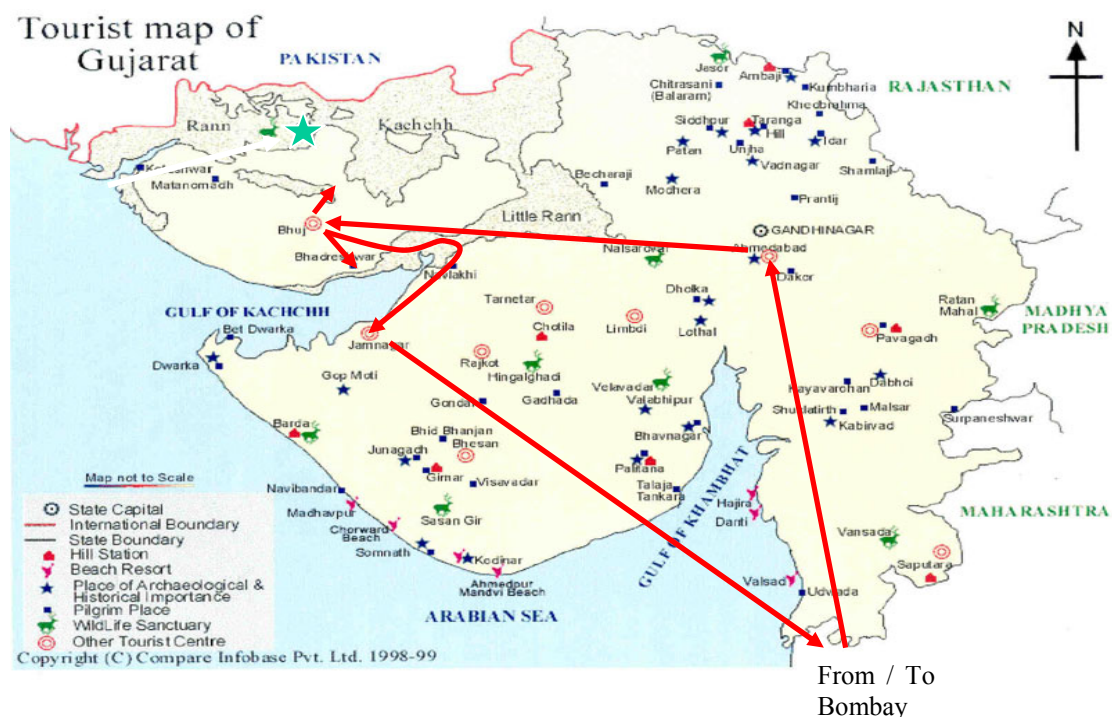


Fig.3 Mission Route

The final leg of the mission was to travel by road from Bhuj to Jamnagar looking at the damage to bridges enroute such as at the Bhachau-Wondh bridge site and Surajbari bridge site. A group also travelled to Navalakhi port which has seen immense liquefaction induced damage. The team spent the

final day of the mission visiting damaged buildings of historical importance and industrial buildings in and around Jamnagar.

## **1.5 Organisation of the Report**

There were many interesting aspects to this earthquake. This report is organised by giving the details of the Geology of the region and Seismological data of the earthquake in Chapter 2 and then presenting the Geotechnical Aspects in Chapter 3. The damage suffered by bridges, ports and industrial facilities are outlined in Chapter 4. The damage to multi-storied buildings are presented in Chapter 5 while the damage to single or two-storied buildings constructed with locally available materials is presented separately in Chapter 6. Finally some observations on the type of damage along with recommendations to minimise such damage in future earthquakes are presented in Chapter 7.

## **1.6 References**

Bilham, R. (1998), Slip parameters for the Rann of Kachchh, India, 16 June 1819 earthquake quantified from contemporary accounts, in Stewart, I.S. and Vita-Finzi, C. (eds.) Coastal Tectonics, Geological Society London, Vol. 146, pp 295-318.

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# Chapter 2 Geological and Seismological data of the Bhuj Earthquake

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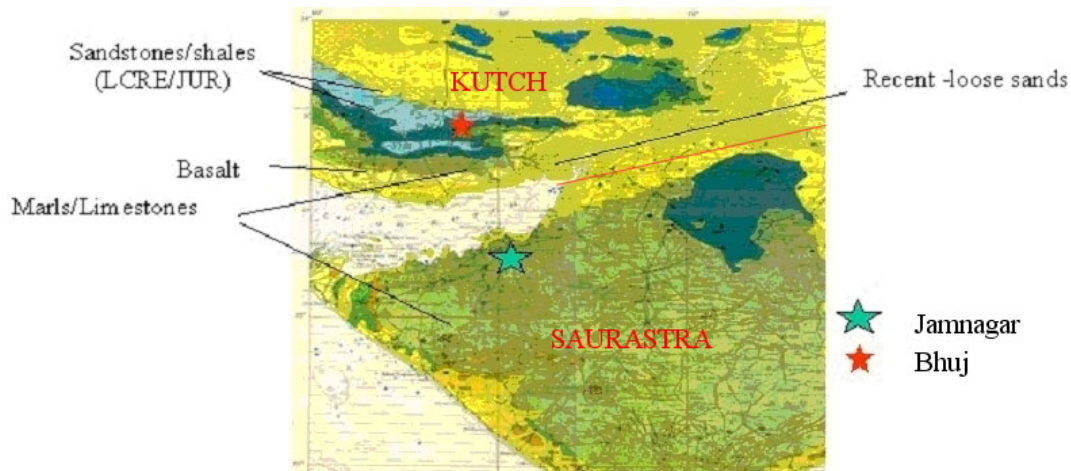
## 2.1 Introduction

In the previous chapter we have seen that the Kachchh region of Gujarat has been seismically active in the recent past, with major earthquakes in 1819 near Allah bund and in 1953 near Anjar. In this chapter we will present the geology of the region and the known fault systems present in this region. We shall look at the Geological data to see if there are any factors that may have contributed to the extensive damage that has resulted from this earthquake. Following this, we will present the seismological characteristics of the current earthquake including the strong motion data.

## 2.2 Geology of the region

### 2.2.1 Kachchh Region

The geology of the Kachchh and Saurashtra regions is presented in Figure 1. The locations of the towns of Bhuj in Kachchh region and Jamnagar in Saurashtra are marked in this Figure. Please note that old spelling of 'Kachchh' was used in this and some of the subsequent figures.



**Figure 1 Geological map of Kachchh and Saurashtra regions**

From Figure 1 we can see that near the town of Bhuj (closest town to the epicentre) there are Sandstones and Shales formed in the Late Cretaceous to Jurassic periods. The Shales and Sandstones are present at shallow depths in this area (typically 2m below ground surface). The sandstones are widely used in the construction of low-rise buildings. This aspect will be discussed further in Chapter 6. In Figure 2 the presence of the Sandstone formations is confirmed in a highway cutting. South of Bhuj there are deposits of Marl and Limestones. As we move further south towards the Gulf of Kachchh we find Basalts. In Figure 2 the presence of the Igneous Basalt is confirmed at the site of an excavation. However in the coastal regions such as Kandla port, there are recently deposited loose



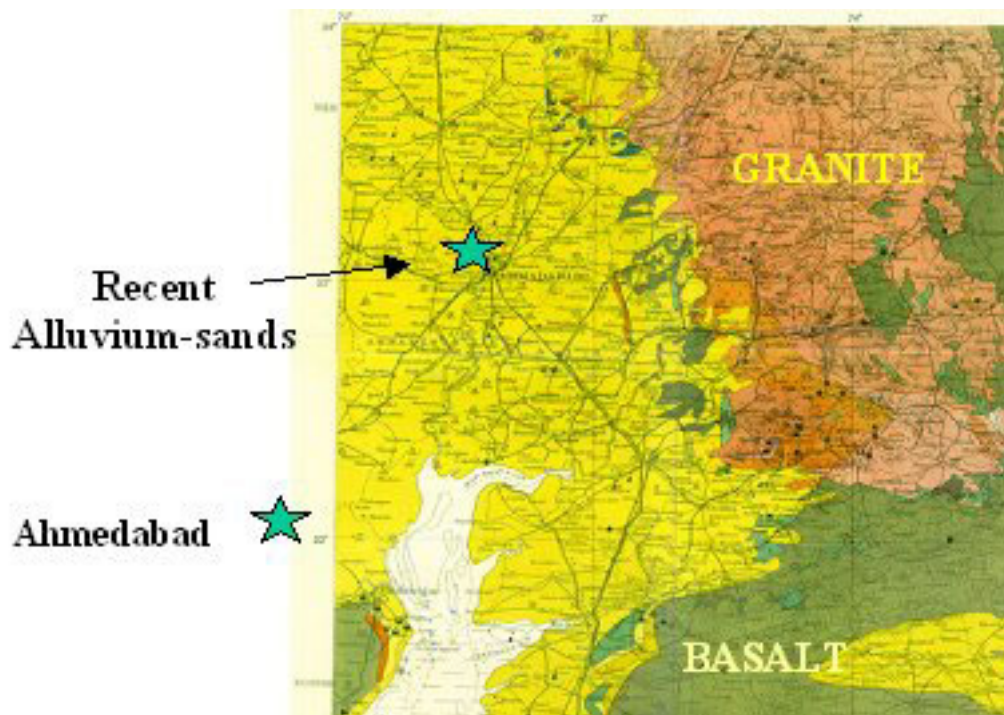
sands. The Kachchh region is known to have folded with the presence of anticline formations along the Katrol hills about 10km south of Bhuj city.

In Figure 1 we see the great extent of Marls and Limestones in the Saurashtra region with the coastal regions showing some Basalt formations. The geology of this region suggests that this region is less susceptible to earthquake risks.



**Figure 2** Excavation and highway cuttings confirm the presence of Igneous Basalt and Sandstone formations in Kachchh

### 2.2.2 Geology of Ahmedabad



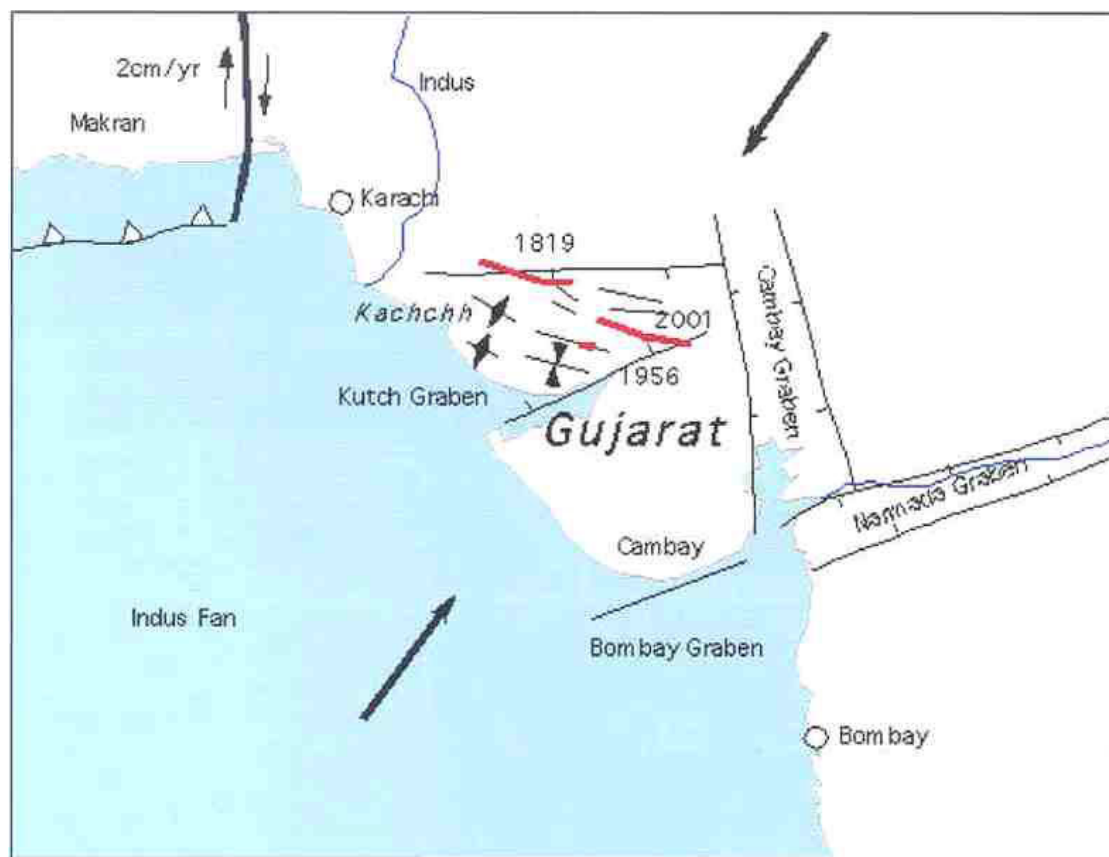
**Figure 3** Geological map of the area surrounding Ahmedabad

The epicentre of the earthquake was about 160 miles west of Ahmedabad, the capital of Gujarat State. Despite the large distance from the epicentre, there was extensive damage to the buildings in Ahmedabad. In Figure 3 the geological map of the area surrounding Ahmedabad is presented, showing the presence of alluvium sands (Quaternary and Tertiary sediments). This region forms the flood plains of the River Sabarmati and hence the presence of alluvial sands here is to be expected. These sediments are known to extend to a depth of several kilometres. This point is further discussed in Chapter 3 in the context of the necessity of soil testing at the site of collapsed buildings in this city. There are also

granite formations to the east and basalt formations to the south-east of Ahmedabad as seen in Figure 3. In general the soil conditions within the city of Ahmedabad did not appear to be problematic under earthquake loading, being neither susceptible to liquefaction or seismic amplification.

### 2.3 Tectonic Setting

The Indian subcontinent is moving northward at a rate of 53 to 63 mm/year colliding with the Asian plate which is also moving northwards but at only half the rate. This relative plate velocity gives rise to an intercontinental collision forming the Himalayan mountains and creating eastward and westward movement of large crustal blocks away from the Himalayan region. The rate of contraction across the Himalayan Frontal Fault System and the western boundary of the plate is about 20 to 25 mm/year. The compression resulted in heavy faulting and folding in the Kachchh region as seen in Figure 4. The 2001 earthquake happened less than 400 km from the Owens fracture zone in the Arabian sea, Makran subduction zone in Pakistan and the Chaman fault running parallel to the Indo-Pakistan border.



**Figure 4 Contraction in the Kachchh region**

### 2.4 Faults in the Kachchh region

There are three main thrust faults in the Kachchh region namely the Allah Bund fault, Kachchh Mainland Fault (KMF) and the Katrol Hill Fault. These are presented in Figure 5. The 2001 earthquake is known to have occurred on the Kachchh Mainland Fault.

The location of the epicentre of the current earthquake is marked on the Kachchh Mainland fault. This fault becomes discontinuous to the east. There are other minor faults between the KMF and Katrol Hill faults. The 1819 earthquake with a similar magnitude of 7.9 to the current earthquake occurred on the Allah Bund Fault. The magnitude 7 earthquake of 1956 near Anjar occurred on the Katrol Hill Fault.

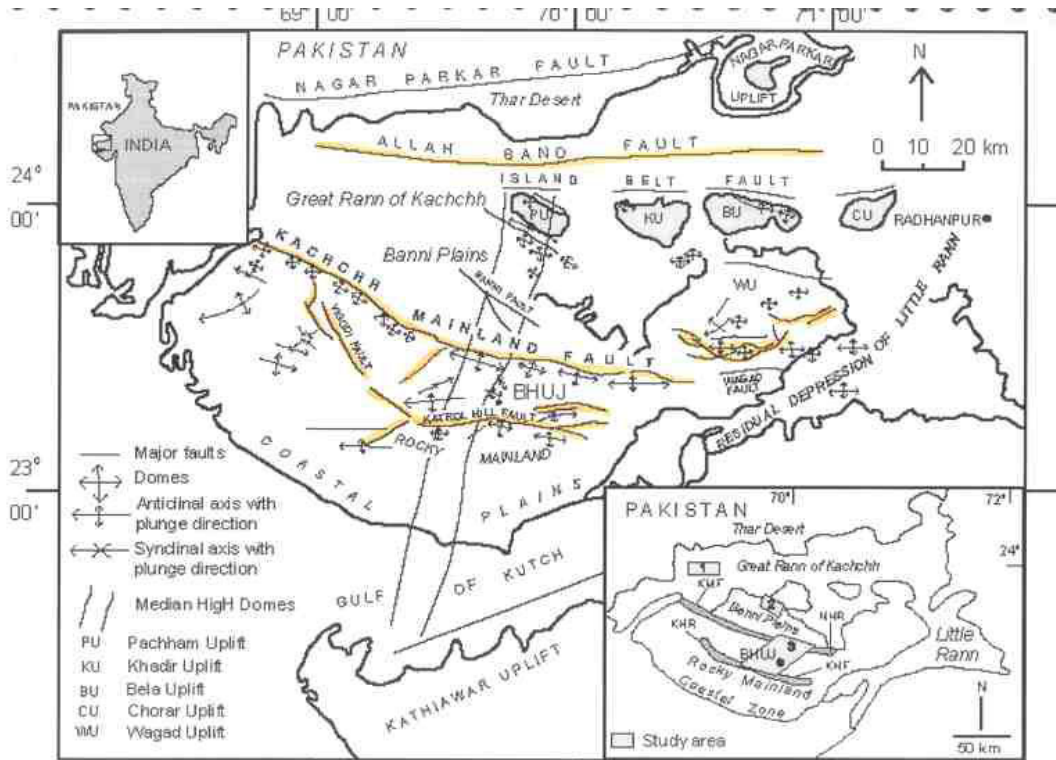


Figure 1. Structural framework of Kachchh (after Biswas and Deshpande, 1970). Inset show generalized geomorphic zones of Kachchh. Shaded boxes in the lower inset shows area of study [ ] and [ ] detailed trench studies in Great Rann-Banni Plain, [ ] study area in rocky mainland. NHR- Northern Hill Range, KHR- Katrol Hill Range, KMF- Kachchh Mainland Fault

Figure 5 The faults in the Kachchh Region

## 2.5 Seismic Zonation

The region of Kachchh is identified as a high seismic risk area in the IS1893 building code for India. The seismic zonation map of India is shown in Figure 6 and the Kachchh region is marked as Zone V, the highest risk on a scale of 1 to V. Ahmedabad on the other hand is classed as Zone III in this Figure.

In Table 1 the Seismic coefficient values suggested in the Indian Building code IS 1893 are presented. For the Kachchh region an appropriate seismic coefficient would be 0.08 i.e. the design horizontal acceleration magnitude may be taken as 0.08g. For Ahmedabad this value would be 0.04g. From the near field records presented in Section 2.6.1 it is seen that the magnitude of ground accelerations in Ahmedabad was 0.11 g which is very much higher than the value suggested by the Indian Building code. It must be pointed out that the factors given in the Indian Building code include a reduction factor as they are applicable to quasi-static design.



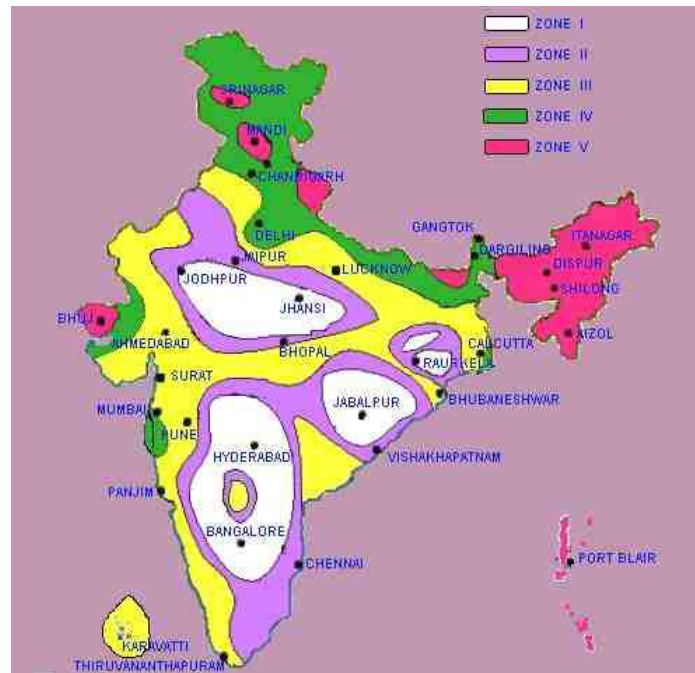


Figure 6 Seismic Zonation map of India

Table 1: Seismic coefficient values for India

| Serial No. | Zone No. | Basic Horizontal Seismic Coefficient $a_0$ | Seismic Zone Factor $F_0$ |
|------------|----------|--|---------------------------|
| 1          | V        | 0.08                                       | 0.4                       |
| 2          | IV       | 0.05                                       | 0.25                      |
| 3          | III      | 0.04                                       | 0.2                       |
| 4          | II       | 0.02                                       | 0.1                       |
| 5          | I        | 0.01                                       | 0.05                      |

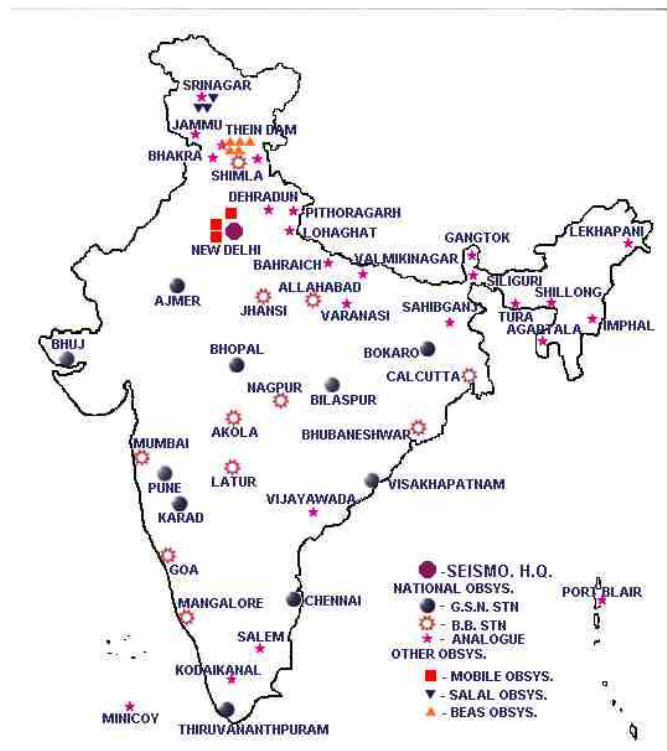
## 2.6 Strong Motion Data

### 2.6.1 Near Field Records

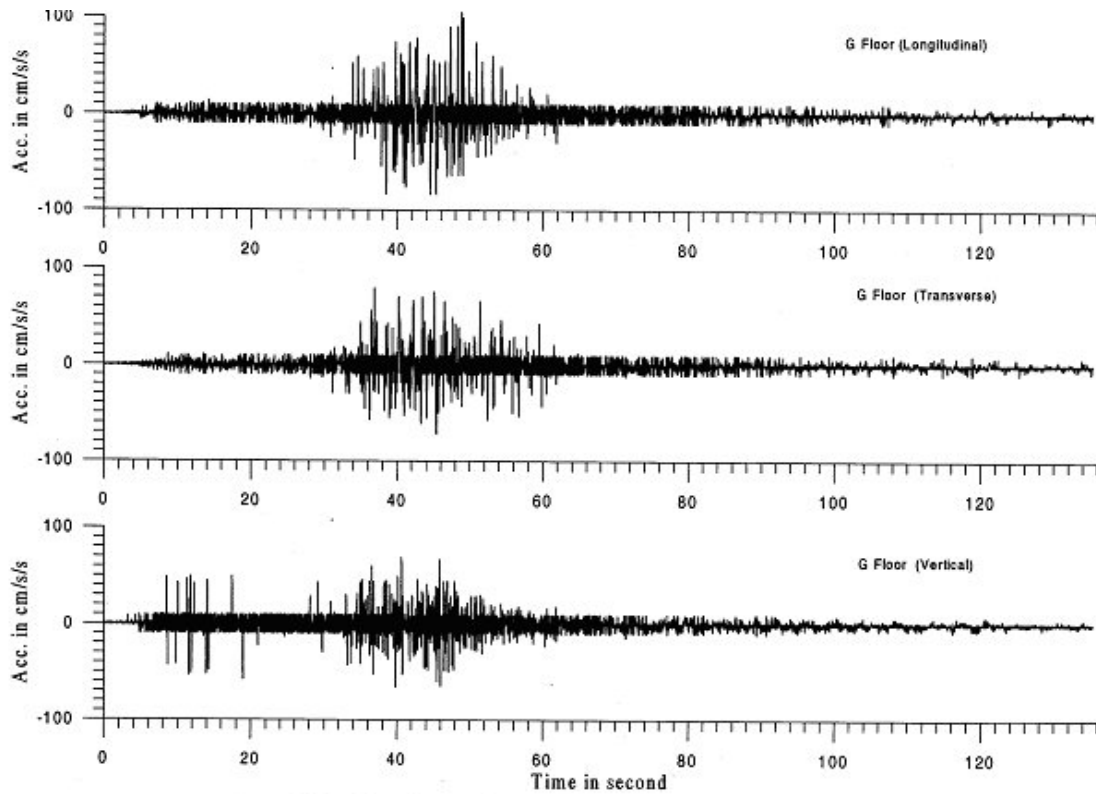
The Bhuj earthquake was a big earthquake and is comparable in terms of magnitude and energy released to the great 1906 San Francisco earthquake. It was an opportunity to collect valuable strong motion data.

Despite the fact that the Kachchh region is known to be seismically active as seen in Seismic Zonation map in Figure 6, very little instrumentation was located in this region. There are various Indian organisations including higher education institutes that establish and maintain the strong motion arrays. This activity could be better co-ordinated so that strong earthquakes such as the 2001 event can be recorded at multiple sites. In Figure 7 the location of the strong motion recording stations in India is presented. There was one strong motion recording station near Bhuj located near the Katrol Hills which did not function well during the earthquake. The only other strong motion recording was in the Passport Office building in Ahmedabad set up by the University of Roorkee. The recordings from this

site were published on the web ([www.rurkiu.ernet.in](http://www.rurkiu.ernet.in)) and are presenting in Figure 8. The acceleration traces seen in this figure are not clear and some amount of interpretation is needed. The peak ground acceleration was estimated to be 0.11 g in Ahmedabad. This is quite large considering the distance of Ahmedabad from the epicentre (nearly 160 miles away) and is 2.75 times bigger than the design values presented in Table 1 for this region. The reason for the large acceleration amplitude may be due to site amplification effects in the thick alluvial sand deposits underlying Ahmedabad. There is no record of the peak ground accelerations in the Bhuj area itself but given the strength of the earthquake and proximity to the epicentre the peak ground accelerations would have been well in excess of the 0.08 g suggested in Table 1. Based on a number of interviews with the survivors in Bhuj the duration of the earthquake was estimated to be about 85 seconds. Various survivors also reported consistently the presence of two distinct pulses which produced predominant shaking in two different directions. It is unclear whether this was due to the difference in arrival times of the P and S waves or whether they were two distinct rupture events in the earthquake.



**Figure 7 Strong motion recording stations in India**

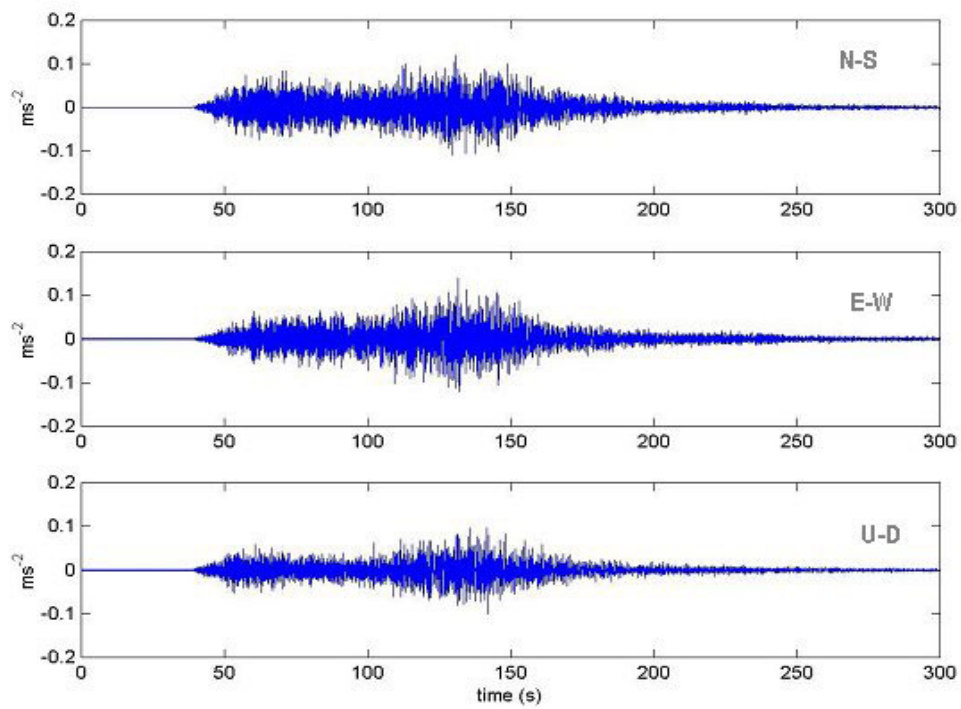


**Figure 8 Accelerations measured in the Ahmedabad passport office (courtesy University of Roorkee, India)**

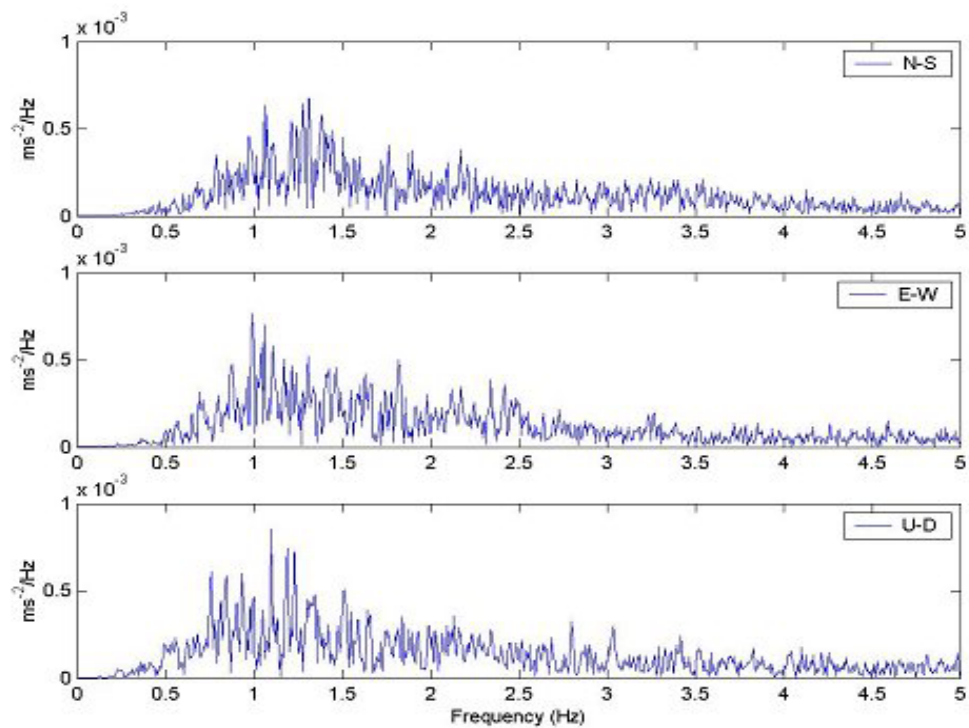
## 2.6.2 Far Field Records

### 2.6.2.1 Acceleration time histories

While the near field records have been scarce, many sources reported the far field data from this earthquake. The USGS published this data soon after the earthquake and has reported the co-ordinates of the epicentre from the teleseismic data to be  $23.36^{\circ}$  N and  $70.34^{\circ}$  E. The Indian Institute of Technology, Bombay had an array near Bombay which recorded acceleration-time histories in the N-S, E-W and U-D directions. This data was provided by Dr Mohan of IIT, Bombay and is reproduced in Figure 9a. No post filtering was carried out on this data. It must be pointed out that this recording station is nearly 400 km from the epicentre and consequently the peak magnitude of acceleration of about  $0.1 \text{ m/s}^2$  recorded here is relatively small. However, in Figs.9a and b we can see that the frequency content of the N-S component is very different from that of the E-W component. The U-D component also has smaller magnitude relative to N-S and E-W components, as would be expected. From Figure 9a we can see that the duration of the earthquake for which significant shaking was taking place is about 120 seconds. In Figure 9b the FFT's for the three components of accelerations shown in Figure 9a are presented.



**Figure 9a** Strong motion data recorded near Bombay, 400 km south of Ahmedabad (Data courtesy of Dr Mohan, IIT Bombay)



**Figure 9b** Fast Fourier Transforms of the ground accelerations recorded near Bombay

### 2.6.2.2 Wavelet Analysis

Fast Fourier Transforms (FFT) are used to transform a time history into the frequency domain. Traditional FFT analysis allows us to look at the frequency content of ground acceleration traces recorded by the seismographs and, using this, we can see the frequency components in the earthquake where the energy is concentrated. However, using traditional FFT analysis we would lose the time information, i.e. if during an earthquake energy has been released at a particular frequency but at multiple separate time instants, we will not be able to distinguish this from an event in which all the energy was released at a single instant. Wavelet analysis on the other hand will let us transform the time history into the frequency domain whilst still retaining the time discretisation. A full description of the Wavelet method is beyond the scope of this report and is described elsewhere, Newland (1997), Teymur et al (2001).

Newland (1999) developed a special analysis technique called Harmonic Wavelet analysis which gives a better frequency discretisation. This method was applied to the N-S and E-W components of the Bhuj earthquake presented in Figure 9. The results of the Wavelet analysis are presented in Figs. 10 and 11 in the form of 3-D plots. In these Figures the axes for time in seconds and frequency in Hz are marked. The colour and height along the vertical axes is proportional to the magnitude component of the Fourier transform of the acceleration trace.

In Figs.10 and 11 we can see that the N-S and E-W components of the ground acceleration had a large number of high frequency components. However, the frequency content of N-S and E-W components is very different. In the E-W component there are many more 'hill tops' at the same frequency content extending over the duration of significant shaking i.e. 100 seconds indicating that several pulses occurred at the same frequency content but at several instants of time. This type of behaviour could not have been picked up by a traditional FFT analysis.

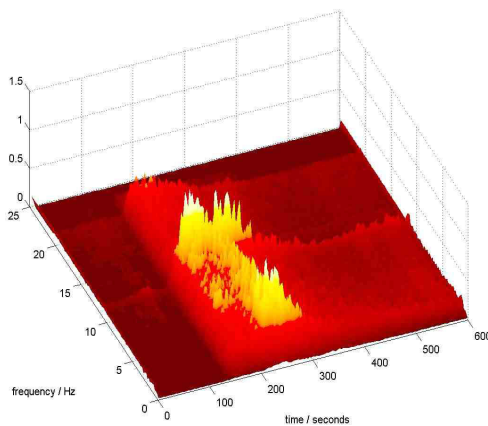


Figure 10 Wavelet analysis of the N-S component

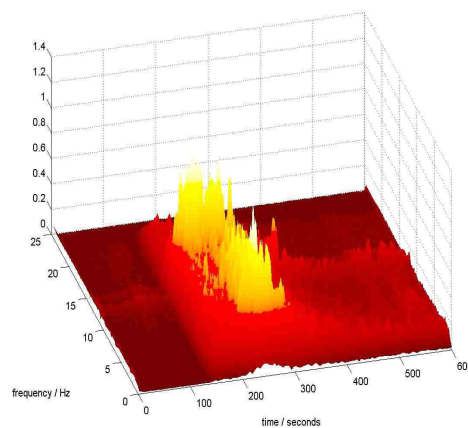


Figure 11 Wavelet analysis of the E-W component

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Teymur, B., Madabhushi, S.P.G and Newland, D.E., (2001), Application of wavelet theory in the analysis of earthquake motions recorded during the Kocaeli earthquake, Turkey 1999, Proc. IV Intl conference on Recent Advances in Soil Dynamics and Earthquake Engineering, University of California, San Diego, USA.

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# Chapter 3 Geotechnical Aspects of the Bhuj Earthquake

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## 3.1 Introduction

The Bhuj earthquake of 26<sup>th</sup> January 2001 had many interesting geotechnical aspects. In Chapter 2 the strong motion data was presented. The paucity of near-field ground acceleration data was considered here. However, based on eyewitness accounts and the far-field records, the duration of this earthquake was established to be longer than other similar magnitude earthquakes such as the 921 Ji-Ji earthquake in Taiwan. It was also seen that the peak ground accelerations in Ahmedabad city were about 0.11g, whilst those in the epicentral region would have been much higher. The strong accelerations in the epicentral region caused widespread liquefaction and much spectacular damage. The local media carried stories and images of water jets gushing out of the ground after the earthquake and potable water turning into saline water (Times of India, 6<sup>th</sup> February 2001, India Today 12<sup>th</sup> February 2001). These aspects will be considered in this chapter. The foundations of bridges in liquefied soil deposits will be investigated and their effect on the superstructure failure mode discussed. The behaviour of foundations of buildings in Ahmedabad where no liquefaction was observed will also be considered in this chapter. However a brief review of soil behaviour under cyclic loading is presented first and the observed behaviour in non-liquefied and liquefied regions will be correlated to the cyclic behaviour of sands.

## 3.2 Cyclic behaviour of sands

The cyclic behaviour of sands has been a topic of research since the extensive liquefaction-induced damage in the 1964 earthquakes in Niigata, Japan and Anchorage in Alaska. While a comprehensive review of the advances made in the understanding of the cyclic behaviour of sands is outside the scope of this report, a brief outline on the current understanding of cyclic behaviour is presented here.

Soil is a complex multiphase material consisting of a solid phase, (the soil grains), a liquid phase, (pore water in the gaps between the soil grains), and a gaseous phase, (air trapped in the gaps between the soil grains). In the case of fully saturated soils, this reduces to a two phase material with no gaseous phase present.

It is known that when a soil matrix is subjected to shear stresses the volume of the soil changes. The volumetric behaviour of dense sands will be dilative as shear strains are applied, whereas loose sands will exhibit contractile volumetric strains. This can be explained by the fact that the soil grains in a loose sand are packed loosely and, when subjected to shear stresses, the particles can move into a more closely packed state thereby resulting in a volume reduction. The converse is true for dense sands. Thus, when subjected to shear stress, dense sands will reach a peak strength before softening to the Critical State. Loose sands, on the other hand, show no peak strength but show a gradual increase in shear stress with strain until they reach the Critical State. This is illustrated in Figure 1, which identifies the contrasting behaviour of loose and dense sands. This monotonic behaviour of sands is also known to be applicable to sands subjected to cyclic loading.

When the sand in question is fully saturated and the application of the cyclic shear stress is rapid, as in the case of an earthquake loading, the tendency of the sands to undergo volumetric strains is exhibited as a change in pore water pressure. Thus saturated loose sands that wish to undergo a volumetric contraction will exhibit a generation of positive pore water pressure, while dense sands will exhibit the generation of negative pore water pressure.

The stiffness of soil is a function of the effective stresses in the soil. The effective stress  $\sigma'$  is linked to the total stress  $\sigma$  and the pore water pressure  $u$  as seen in Eq.1 below. The total stress in the soil does not change significantly during the earthquake.

$$\sigma' = \sigma - u \quad (1)$$

Thus, in the loose saturated sands, as the pore water pressure  $u$  increases the effective stress  $\sigma'$  comes down, lowering the stiffness of the soil. If the increase in pore water pressure is so high that the effective stress  $\sigma'$  falls to a near zero value, we say that the soil has fully liquefied. In this state the soil has no strength and any structure founded upon it will simply sink in or rotate. When the magnitude of the effective stress is reduced due to generation of some excess pore pressure, (but not sufficiently to cause full liquefaction), the stiffness of the soil is reduced. This is termed partial liquefaction. Research indicates that under partial liquefaction conditions, structures may still suffer damage as an initially high frequency soil-structure system may suffer a lowering of its stiffness and natural frequency, drawing itself closer to earthquake driving frequencies, thereby becoming vulnerable to resonance. Dense sands on the other hand will exhibit a generation of a negative pore water pressure during the earthquake, thereby increasing the effective stress  $\sigma'$  and consequently the stiffness of the soil. Thus dense sands are not vulnerable to liquefaction during earthquakes.

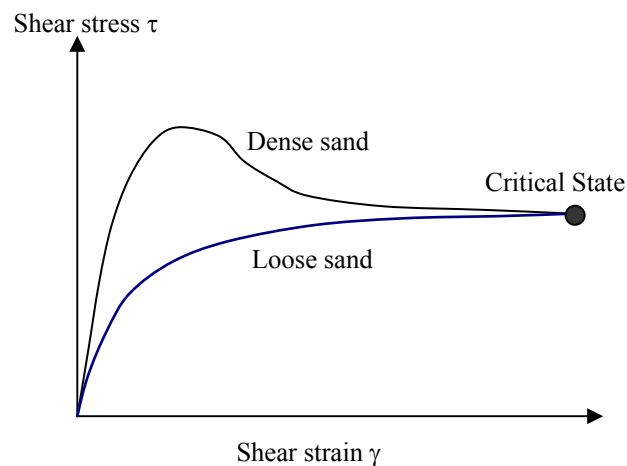


Figure 1 Behaviour of Dense and Loose sands

### 3.3 Foundations in Ahmedabad

In chapter 2 we have seen that the geology near Ahmedabad indicates the presence of deep quaternary and tertiary deposits. The river Sabarmati passes through the city and several bridges cross over this river. A new bridge is also under construction and local geotechnical engineers confirmed that the soil profiles from bore hole data from this site and throughout the city indicate the presence of a layer of dense sand extending to depths greater than the deepest foundations of the bridges and other multi-storied buildings in the city (> 50m). Sites next to the river show medium dense sands in the top 2 or 3 meters underlain by the dense sands with SPT (Standard Penetration Test) values of 30 or more. These soil conditions are ideal for load bearing and the local geotechnical engineers correctly use either simple strip or pad footings for normal buildings and friction pile foundations with some base resistance for multi-storied buildings.

#### 3.3.1 Lack of Liquefaction

In line with the above geotechnical information there was no liquefaction observed anywhere in the city of Ahmedabad. Even sites next to the river showed no sand boils, lateral spreading or other signs of liquefaction. This is consistent with the theory presented in Section 3.2, as the saturated dense sands will tend to dilate and create suctions thereby increasing the stiffness of the sands.



### 3.3.2 Site Effects

The peak ground accelerations (PGA) in Ahmedabad were reported to be 0.11g, as was discussed in Chapter 2. One of the interesting aspects of this earthquake was the fact that such relatively large PGA's were felt in Ahmedabad some 160 miles from the epicentre. This surprisingly high PGA could be due to directivity effects. A good database of ground motions in a future strong event such as this would give the required information to establish such effects.

Observations from past earthquakes and experimental work on geotechnical centrifuges have shown that amplification of bedrock accelerations is a possibility at sites with loose, dry alluvium deposits. Amplification of bedrock acceleration may not happen in dense soil deposits. Given the site conditions in Ahmedabad discussed in Chapter 2, it may not be the soil amplification effect that has caused the relatively large peak ground accelerations. Basin effects may however have been the cause, but this aspect needs further research of the area surrounding Ahmedabad.

### 3.3.3 Site Investigations

At several locations in Ahmedabad where multi-storied structures have collapsed, such as the Mansi complex, (described in detail in Chapter 5), soil testing was being carried out. In Figure 2 the SPT tests being carried out at one of the sites are seen. Trial pits have been excavated to inspect the foundations at several of these sites. Given the experience of the local geotechnical engineers on the more or less known soil conditions through out the city, the need for these site investigations at all the sites of collapsed buildings is unclear. While soil testing at problematic sites is both necessary and often crucial we can say at least with hindsight that soil testing at the collapsed building sites in this instance may not have been the high priority that was assumed.



**Figure 2 Soil testing at the site of a collapsed building**

### 3.4 Liquefaction

In Section 3.2 we have seen the theoretical background to the liquefaction of soils. The Bhuj earthquake has resulted in liquefaction in the Kachchh region. The regions in which liquefaction has occurred can be classified as being the coastal areas next to the Gulf of Kachchh, the Rann of Kachchh which consists of the salt marshes to the north of Bhuj and the little Rann of Kachchh (North east of Bhuj). The liquefaction in the coastal regions on either side of the Gulf of Kachchh is to be expected as in these regions the soil consists of loose sandy soil deposits and the water-table is shallow. We shall consider examples of the liquefaction in the coastal regions in Sections 3.5 to 3.7.

#### 3.4.1 Sand boils

Liquefaction in the Rann of Kachchh is to be expected as this region is known to have a shallow water table. One of the tell-tale features of liquefaction is the occurrence of sand boils. As explained in Section 3.2, when the effective stress in the soil is reduced to near zero magnitudes, fracturing and boiling of the soil can result and often gushes of liquefied soil are observed on the surface. In Figure 3 below, the sand boils observed near the village of Lodhai are presented. It must be pointed out that this region is very dry, as seen in the background in Figure 3, with the local vegetation constituting only small shrubs and bushes. The places where the sand boils occurred therefore show good contrast with the dry soil. In Figure 3 we can see the large size of sand boils. In some instances it was seen that the diameter of the sand boils was well over 6m. Also the presence of moist sandy soil was noticed in the centre of the boiled region more than ten days after the earthquake. This was unusual given the ambient temperature of the region during the day ( $35^{\circ}\sim 40^{\circ}$  C). No rain had occurred between the time of the earthquake and the EEFIT team's visit to the region. One possibility may be that the liquefied soil had flowed for up to several days after the main earthquake. This continual presence of high pore water pressure has been observed in past earthquakes, particularly in the mud flows observed in the Redondo beach area after the Northridge earthquake of 1994 which continued for a long time after the main event. The undrained shear strength of the wet soil seen in Figure 3 was determined by the EEFIT team using a hand held vane shear apparatus. The wet soil had an undrained shear strength of about 10 kPa. However the partially dried out salt crustation showed high undrained shear strength which was in excess of 50 kPa.



**Figure 3 Sand Boils near Lodhai**

Sand boils occur after many earthquakes in regions that are susceptible to liquefaction. Some important features of the sand boiling during the Bhuj earthquake were:

- a) The sand boiling was spread out over large areas extending over several square kilometres;
- b) Crystalline salt deposition was present on the surface.

The above aspects are considered below.

One other feature of the liquefaction in this area was the 'linear pattern' of the sand boils. Most sand boils, as seen in Figure 3 and indeed in the previous earthquakes, show an approximate circular shape and the flow of liquefied material follows the natural indentations on the ground. This fits in with the

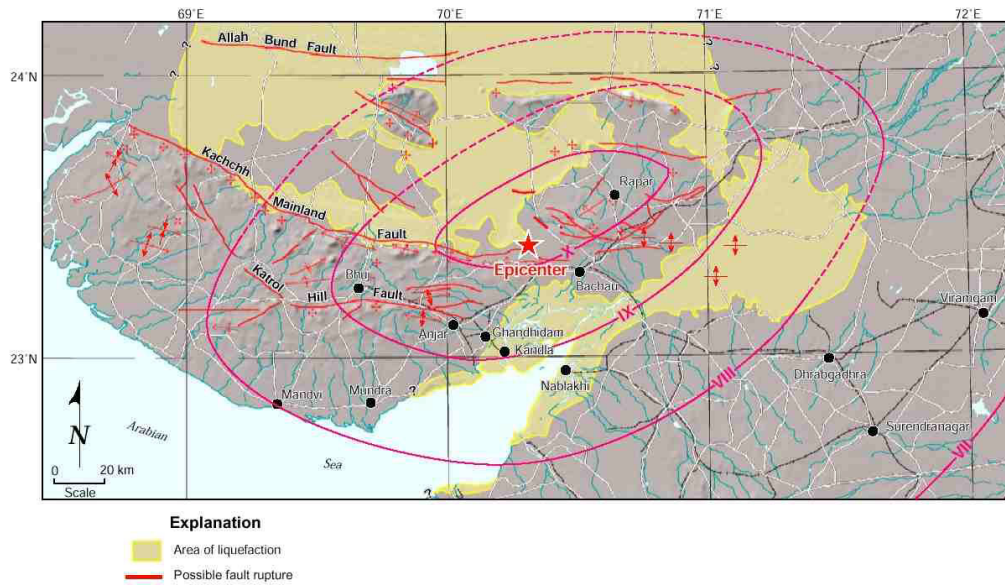
theory in that high pore water pressures forming in the underlying layers of soil would look for a weakness in the overlying soil strata, would puncture this weakness and liquefied soil would flow to ground surface. However, there were several locations where liquefaction showed the pattern seen in Figure 4. These sand boils seem linear (length much greater than width) and run for several hundreds of meters. It is not conceivable that liquefied material has punctured one location and then the material has flowed on the surface for such large distances, particularly due to the dry nature of the region and high permeability of the sandy soils near the ground surface. A possible explanation was the existence of high excess pore water pressure in the underlying soil causing fracture/rupture in the water courses which can contain loose soils deposited during the flash floods in the brief rains that the region receives, these soils may well be looser and hence weaker and more permeable than the soils in nearby areas.



**Figure 4 Linear sand boils following the natural water courses**

The other interesting aspect of the sand boils was the presence of the crystalline salt formations seen in Figures 3 and 4. Clearly the liquefied soil brought salt water to the surface. A study of the hydrogeology of the region would be very interesting from a liquefaction point of view. It appears that the local water conditions were such that the potable water was close to the surface, underlain by saline water deposits seeping from the Gulf of Kachchh. Following the earthquake, local media reported changes in potable water quality with drinking water turning more saline (Times of India, 6<sup>th</sup> February 2001). The formation of crystalline salt, discussed above, also fits with this scenario. It is possible that this region may have settled due to the formation of a graben between the fault systems, increasing the water pressure over a very large area, moving the saline water closer to surface and mixing with potable water. Sand boils then would appear at places of weakness as the high pore water pressure will try to escape. This mechanism is very different from the high pore water pressures formed due to application of cyclic shear stresses to loose saturated soils, as discussed in Section 3.2. Only a major study that establishes the surface levels over the region relative to known benchmarks can clarify this further. Such a study near the Allah bund region was carried out in the past to estimate the ground movements after the 1819 earthquake, Bilham (1998).





**Figure 5 Aerial distribution of the liquefaction**

The extent of liquefaction was very wide, as described above, covering regions from the Rann of Kachchh in the North to Navalakhi to the South of the Gulf of Kachchh. In Figure 5 the areas of liquefaction are identified on a map. The large extent is perhaps due to the strength of this Magnitude 7.9 earthquake.

### 3.4.2 Lateral spreading



**Figure 6 Lateral spreading of a railway embankment**  
[courtesy of EERI]



**Figure 7 Cracks due to lateral spreading**  
[courtesy of EERI]

One of the effects of liquefaction on gently sloping ground is lateral spreading. As the soil stiffness is reduced owing to the increase in pore water pressure during an earthquake, soil in sloping ground can start to spread laterally. If the slopes are steep and provided that the excess pore water pressure is retained sufficiently long, the sloping ground can suffer a flow slide. These flow slides tend to result in movement of the ground by several hundreds of meters. Flow slides are recognised to be gravity driven once liquefaction has initiated the slide. There were no examples of flow slides due to the general flatness of the Kachchh area with only gentle slopes. However man made slopes such as railway embankments have shown such failures as seen in Figure 6. Research has shown that even gently sloping ground (with slope angles as small as  $3^\circ$ ) can suffer lateral movements of tens of meters. This is considered to be an inertial event and is called lateral spreading. The topographic relief of the Kachchh

region with the presence of gentle slopes is particularly vulnerable to lateral spreading. In Figure 7 we present a view of the lateral spreading that has occurred during the Bhuj earthquake. The lateral spread is quite striking in this picture, particularly given the surface dryness seen in this figure. Clearly the liquefaction induced softening happened at some depth below this ground with a crust of non-liquefied soil near the surface. As lateral spreading progressed the crust suffers lateral displacements. Tension cracks appear on the surface due to differential amounts of horizontal strain in the crust (due to local variations of slope etc). Lateral spreading is known to cause significant damage to buildings, especially those with piled foundations and other civil engineering structures such as retaining walls and bridge abutments. In Figure 8 the damage caused to a petrol filling station during this earthquake by lateral spreading is seen. There is clearly a need to adopt liquefaction resistant measures in regions vulnerable to lateral spreading during the reconstruction program in the Kachchh region.



**Figure 8 Lateral spreading resulting in damage to a filling station**

### **3.5 Bridge Foundations**

Bridge foundations are particularly vulnerable from a liquefaction point of view. In this section we present the post liquefaction behaviour of some of the bridge foundations. This is again an area where during the reconstruction and repair of the infrastructure, liquefaction resistant designs need to be adopted.

#### **3.5.1 Foundations of the Rudra Matha bridge**

This bridge is close to the epicentral region and is only a few miles from the Rudra Matha earth dam that will be discussed in Section 3.7. The bridge superstructure is discussed in detail in Chapter 4. The bridge structure has a long natural period ( $\sim 3$  s) owing to its height. In Figure 9 an overview of the bridge is presented. The bridge foundations consist of well caissons sunk into the river bed with framed piers located on them. Liquefaction of the soil has occurred, as suggested by the presence of sand boils next to the water line, as seen in Figure 10. Further, lateral spreading occurred next to the piers closest to the water line. In Figure 11 a close up view of one of the piers is presented. In this figure it can be seen that lateral spreading of the soil towards the water line has left a gap of several inches next to the well caisson. Also tension cracks are visible on the soil surface.



**Figure 9 Overview of the Rudra Matha bridge**



**Figure 10 Sand boils next to waterline**

Despite the liquefaction of the foundation soil, the bridge structure performed well during this earthquake. The bridge super structure must have undergone vibrations during the earthquake but owing to the long natural period of the bridge, there would have been very little amplification within the bridge structure. There would have been some amount of dynamic soil-structure interaction during this period. By and large the liquefaction effects on this bridge were minimal and the well caisson foundations performed satisfactorily.



**Figure 11 Lateral spreading next to the well caisson**

### **3.5.2 Bhachau - Vondh bridge site**



**Figure 12 Plate girder railway bridge**



**Figure 13 Extensive liquefaction at the pier bases**



There are four bridges at this site, two railway bridges and two highway bridges. The location of these bridges is very critical in the sense that the main rail link and an important highway pass through the site, connecting the Bhuj region to the rest of India. Following the earthquake the railway bridges were out of operation for about a week after the earthquake due to the repairs required to the arches, delaying relief trains from reaching Bhuj. One of the railway bridges was an old arch bridge while the other is a relatively modern plate girder bridge. One of the highway bridges was old (about 30 years) and a new bridge was under construction at the time of the earthquake. The alignment of all four bridges was approximately in the E-W direction. The foundation soil at the site liquefied during the earthquake. The behaviour of the foundations of all the four bridges will be presented in this section. The performance of the superstructure of these bridges is dealt with separately in Chapter 4.



**Figure 14 Sand boils at the bridge site**



**Figure 15 Damage to the crown blocks of the arch**

An overview of the bridge site is shown in Figure 11, which shows a train crossing the plate girder bridge. In Figure 12 the liquefaction of the foundation soil at the base of the piers is presented. In this figure the arch bridge can also be seen in the background. Extensive sand boils were seen at the base of the piers. As seen in Section 3.3.1 the sand boils exhibited crystalline salt deposits once dried. The area surrounding this bridge site also showed extensive liquefaction, as seen in Figure 13, in which a view of the region next to the arch bridge is presented.

It is interesting to compare the performance of the arch bridge and the plate girder bridge after foundation soil liquefaction. The arch bridge suffered loosening of the blocks at the crown of the arch, as seen in Figure 14. These blocks were repaired by injecting epoxy resin into the gaps with supporting steel framework under the arch. The failure mechanism leading to this loosening of the arch could have occurred due to differential settlement of the pier foundations. This would have led to a reduction in the lateral restraint of the arch reducing the thrust and allowing the gaps to be opened up between the stone blocks forming the arch crown. The parapet walls of the bridge next to the permanent way also collapsed during the earthquake. The liquefaction of the foundations of the plate girder bridge as seen in Figure 12 provided a different failure mechanism that dictated its behaviour during and immediately after the earthquake. The piers supporting the plate girders show a tilt about the longitudinal axis of the bridge. In Figure 15 a view of the piers, as seen parallel to the longitudinal axis of the bridge, is presented. In this figure we can see that each of the piers has rotated relative to the next pier.

The liquefaction of the foundations begets different failure mechanism in different bridge constructions. It appears that the response of the bridge to foundation liquefaction may depend on the stiffness of the superstructure. An arch bridge would have significant torsional stiffness. As a result, when the foundations liquefy, the superstructure will offer significant resistance to twisting. However, the arch bridge relies on the lateral thrust for its stability and hence is vulnerable to differential

settlements of the piers. The plate girder bridge on the other hand is more flexible in the torsional mode. As a result the liquefaction of foundations of this bridge can result in rotation of the bridge structure, as indicated by the relative rotation of the piers.

Based on the comparison of the behaviour of these two bridges we can say that the stiffness of the superstructure of the bridge can dictate the failure mechanism once the foundation soil has suffered extensive liquefaction. As a corollary to this we must note that once the foundation soil has liquefied the weakest mechanism will be picked up and the structure would progress towards failure in that mechanism. Further investigation of failure mechanisms of bridges located on liquefiable foundations needs urgent consideration to further understand these issues.



**Figure 16 Plate girder bridge piers suffered rotation**

**Figure 17 Relative elevations of the highway bridges (old bridge to the left)**

The two highway bridges have also seen severe damage. The foundations of the old bridge seemed to be well caissons while pile foundations were used for the new bridge. Information on the depth of these pile foundations was not available to the EEFIT team. The old bridge remained operational and was being used by the relief and other traffic. The elevation of this bridge deck was low with only small approach embankments compared to the new bridge. In Figure 16 the approach embankment for the new bridge in relation to the old bridge can be seen. As a result of this lower elevation of the old bridge relative to the new one, the earth pressures acting on the abutments of the old bridge would be smaller.

The old bridge suffered damage with extensive spalling of the shotcrete, as explained in Chapter 4. The parapet walls of this bridge also collapsed, as seen in Figure 17. The foundation soil liquefied, but due to the low height of the abutment walls, the lateral thrust from the earth pressure on the decks was not extensive. The new bridge, on the other hand, has suffered due to the additional earth pressures acting on the abutments. The pile foundations could not restrain the lateral force on these abutments once the soil has liquefied, allowing the abutment walls to move in by about 500 mm, as seen in Figure 18. Interesting bridge deck-abutment-pile foundation interaction ensued. The structural aspects of the failure mechanism are explained in Chapter 4. It was not clear whether the piles had sheared at their heads in order to allow this 500 mm inward movement or if the piles simply rotated, forming plastic hinges and/or translated once the surrounding soil has liquefied. This can only be clarified by knowing the depth to which the piles were extended during construction and the depth to which the soil at this site has liquefied or exposing the underside of the pile cap.





**Figure 18 Extensive spalling of concrete on old highway bridge**



**Figure 19 Failure of the abutment walls of the new highway bridge**

This site has provided many interesting examples of failure mechanisms. Each of the four bridges performed differently, depending on their type of construction, even though all of them had the same foundation problem, i.e. liquefaction of the soil. Valuable lessons can be learned from these and further research can be carried out such as interaction of the bridge abutments, bridge deck and the foundations in liquefiable soils. Based on the findings of that research, future design strategies can incorporate measures to compensate against these failure mechanisms.

### 3.5.3 Suraj Bari Bridges

This bridge site lies to north east of the Gulf of Kachchh and consists of two highway bridges and a railway bridge. The railway bridge did not suffer any damage. The old highway bridge suffered extensive damage. This will be explained in Chapter 4. A new highway bridge was under construction at the time of the earthquake. Again the structural damage to the new bridge will be presented in detail in Chapter 4. In this section we consider the soil-structure interaction effects exhibited by the new bridge.

The bridge structure experienced severe vibration during the earthquake and it was reported that some of the bridge decks were dislodged. The decks were lifted back into place using the cranes on site by the time that the EEFIT team visited this bridge. A view of this bridge is presented in Figure 20. There was evidence of decks colliding with one another as damage to the concrete stops was visible (see Chapter 4). The piers of the bridge are supported on pile foundations. During the vibrations of the bridge, the piers would have interacted with the decks and with the foundation soil. Large gaps of about 75 mm were evident at the base of the piers, as seen in Figure 20, giving an indication of the amplitude of vibration at the pier bases. These gaps seem to extend below the surface to a depth of about 0.8 m. Of course, this could have been greater immediately after the earthquake and have filled up with soil later on.

This bridge provides a good example of the soil-structure interaction that happens during a major earthquake event. The bridge deck-pier-pile foundation system interacts closely and the overall response of the bridge depends on the stiffness and damping of the super-structure as well as the stiffness and compressibility of the soil surrounding the piers and the foundations.



**Figure 20** The new bridge at Suraj Bari



**Figure 21** Large gap opened between the pier and soil

### 3.6 Liquefaction at Ports and Harbours

As we have seen with bridge foundations, ports and harbours and related structures are particularly vulnerable to liquefaction, given their proximity to water fronts and the associated high water tables. The Bhuj earthquake has provided examples of the damage that could befall ports and harbours. In particular we shall consider two specific port facilities, namely the Kandla port and the Navalakhi port. Kandla port is closer to the epicentre and is only a few kilometres from the town of Gandhidham, which saw severe damage to many buildings (discussed further in Chapter 5). However, damage to Kandla port was light and the port was quickly brought back into operation, providing berthing facilities for relief arriving by sea. Almost directly across the Gulf of Kachchh, (see Chapter 2), lies the port of Navalakhi, which saw extensive damage due to liquefaction. Navalakhi is a bulk handling port and deals principally with coal, chemicals etc. It is close to the industrial city of Jamnagar. This port is much further away from the epicentre on the far side of the Gulf of Kachchh. The reason for the extensive damage to Navalakhi port may be that the port was constructed from dredged material from the gulf. As was seen in the Bay area during the San Francisco earthquake of 1983 and at the port island site during the Kobe earthquake of 1995, the reclaimed land in Navalakhi, constructed from dredged material has once again proved to be very susceptible to liquefaction. There were no records of ground accelerations near Navalakhi but, given the distance from the epicentre, this would have been relatively small, barring any site amplification effects.

In the following sections we shall consider the effects of liquefaction on the ports of Kandla and Navalakhi. Again the structural aspects of the damage are covered separately in Chapter 4.

#### 3.6.1 Kandla Port

Kandla port is a major port facility in India and handles about 17% of India's imports and exports. It has five old jetties and five new jetties with plans to construct four more in the future. The old jetties were supported on RC piles of 400mm diameter connected on the top by beams above the water line. The soil profile at Kandla port has soft marine clay at the surface underlain by medium to dense sands extending to a great depth. The depth at which bedrock is present is not clear. The piles were accordingly designed as friction piles with little contribution due to end bearing. A view of these old jetties is presented in Figure 21. Some of the piles have developed horizontal and vertical cracks, as seen in Figs. 22 and 23. In discussions with Mr Jyothi, the Chairman of the Port, it was learnt that the allowable load intensity on these jetties following the earthquake was reduced in an attempt to limit the load transferred to the piles.

The five new jetties located to the western side of the port look modern and the pile foundations supporting these are more substantial. These were 900 mm diameter piles with sacrificial steel jackets left in place. A view of these pile foundations can be seen in Figure 24. The backfill behind the jetty structure suffered subsidence similar to that in Taichung Harbour after the 921 Ji-Ji earthquake. This is clearly seen in Figure 25, which shows nearly 300 mm of subsidence at locations. It appears that this is

a result of densification of the backfill, very little lateral spreading occurring towards the sea, thereby limiting the lateral load on the piles to nominal levels. Liquefaction was more predominant at the eastern end of the port.

The harbour master's office building located on the east side of the port, suffered damage due to liquefaction. The building is of stone construction supported on pile foundations. This building tilted away from the vertical by about 15°. A view of this building is presented in Figure 26. In this figure we can see that the building has separated from the walkway structure connecting higher floors to adjacent buildings. The tilt of the building is believed to be due to the settlement of the piles on the seaward side (relative to the piles on the landward side). This can also be explained by the lateral spreading that occurred in this part of the port (as will be seen in the case of an adjacent building that collapsed). The piles may have suffered settlement due to softening of the soil during liquefaction causing a loss of skin friction. It is also possible that the lateral spreading of soil next to these piles may have contributed to the settlement of these piles relative to those on the landward side, thereby causing the building to tilt. A trial pit was dug to reveal the pile foundation on the seaward side. A view of the pile is presented in Figure 27 and no damage was observable at the pile cap.

An office building about 200 yards from the above building suffered a soft storey collapse. A view of this building is presented in Figure 28. This building is close to the shoreline. The main cause for the collapse of the building was structural and this will be considered separately in Chapters 4 and 5. However, it is interesting to note the soil behaviour particularly behind this building. Lateral spreading has caused subsidence of the soil behind this building. The lateral spreading has also destroyed the parapet wall behind the building, as seen in Figure 29. This figure confirms the seaward movement of the soil with the wall tilting seawards and collapsed parts of the wall distributed towards the seaward side. Even if this building had not collapsed due to structural reasons, it would still have suffered extensive damage due to liquefaction and lateral spreading of the foundation soil.



**Figure 22 A view of the old jetties at Kandla Port**



**Figure 23 Cracks developed in some of the piles supporting the old jetties [courtesy of EERI]**





**Figure 24 Steel jacketed piles supporting the new jetties**



**Figure 25 Subsidence of backfill behind the new jetties**



**Figure 26 The harbourmaster's office suffered tilting**



**Figure 27 Trial pit showing the pile foundation on the seaward side**



**Figure 28 Collapsed building close to harbourmaster's office**



**Figure 29 Collapsed wall due to lateral spreading**

### 3.6.2 Navalakhi Port

As mentioned before, Navalakhi is a bulk handling port owned by GMB Port authority, constructed from dredged material from the Gulf of Kachchh. An aerial view of Navalakhi port is presented in Figure 30. The port is privately owned and the earthquake resulted in severe damage to many of the jetty structures. Liquefaction was extensive in this port and settlements of the order of 3 ~ 4 m were reported. Liquefaction induced lateral spreading has resulting in damage to sheet pile walls and to sea revetments. Examples of this are presented in Figures 31 and 32. In Figure 31 an aerial view of a sheet pile wall that has completely collapsed is seen. It may be noted that there is general loss of ground from behind the sheet pile wall owing to lateral spreading. In Figure 32 the sea revetment is seen to have suffered severe damage with lateral spreading causing the slope to fail and the paving blocks to be dislodged.



**Figure 30 Aerial view of the Navalakhi port**



**Figure 31 Failure of a sheet pile wall**



**Figure 32 Lateral spreading resulting in damage to sea revetments [courtesy of IIT Kanpur]**

The loading areas behind the jetties have shown extensive settlements and severe cracking. In Figures 33 and 34 we present the damage to the loading areas behind the jetties. In Figure 34 we can see the large cracks that have opened up in the adjacent areas to the jetties. The largest of we can see the settlement that has taken place in the backfill forming the loading area. Some of the settlement would be due to densification of the backfill under the cyclic loading, but in this area there was evidence of seaward lateral spreading of soil adding the cracks was nearly 600mm wide, extending deep below the ground surface. Given the damage suffered by the jetties and the areas behind them in most major earthquakes around the world, there is a general need to improve compaction of the backfill areas to minimise the damage suffered due to liquefaction induced lateral spreading.

Navalakhi port is serviced by a broad gauge railway line. This railway has suffered extensive damage due to lateral spreading. A 30 metre section of the railway was almost completely destroyed. A view of this section is presented in Figure 35.

The railway embankment has spread causing the railway line to settle and rotate thereby resulting in twisting of the track. Approaching this failed section from the port side, the loss of ground and resulting settlements were clearly visible. In Figure 36 we can see two successive 'steps' in the track and the parapet wall by the trackside. These steps were each about 0.5 m deep. There is a sharp left turn in the original track alignment from the track seen in Figure 36 leading to that in Figure 35. The steps in the ground elevation resulted in the disappearance of the track below water level at high tide. In Figure 37 the amount of lateral spreading that has taken place next to this section of the track can be seen. A temporary road way was laid next to the track, which was being used to bring the goods by lorries, to service those jetties that were still operational.

Even though the failure of this railway line is spectacular, it may be possible to repair the damaged section of the railway. Given that this railway is only used by goods trains and does not form part of any main route, it is the authors opinion that quick relaying of the track may be prudent with the understanding that future earthquakes would result in similar damage with the need for further repairs.



**Figure 33 Settlement of backfill behind the jetties in Navalakhi port.**



**Figure 34 Severe cracking and lateral spreading of the area adjacent to the jetties**



**Figure 35 Damage to the railway line feeding Navalakhi port [courtesy of IIT Kanpur]**





**Figure 36 Sudden change in ground elevation along the track**



**Figure 37 Lateral spreading next to the track**

### **3.7 Behaviour of Earth Dams**

Earth dams have become very popular in India due to their relatively low cost and ease of construction using locally available materials. Irrigation in many regions depends on the water supplied by these dams. Many earth dams are present in the Kachchh and Saurashtra areas of Gujarat. The arid climate of the region combined with the concentration of the rainfall into a few months of each year necessitated construction of earth dams to enable effective use of water. Many of these dams are of moderate size with the largest being the Rudra Matha dam with a crest height of about 30m. It must also be pointed out that not all of the dams were visited by the EEFIT team due to time constraints, and where possible, information collated by teams from the Indian Institute of Technology, Kanpur and EERI in the USA is presented in this section.

The performance of the earth dams will be considered in this section. Several earth dams have suffered damage during this earthquake. Only a few examples will be considered here. It must be pointed out that due to failure of the monsoon in the previous year, many of the reservoirs had very low water levels at the time of the earthquake. Even though some of the earth dams were damaged quite severely, there was no danger of any breaching immediately after the earthquake. However there is an urgent need for many of the dams to be repaired before the onset of the next monsoon. The Central Water Commission, which is the main body responsible for dam safety and for the performance of the dams in India, carried out inspections of some dams at the same time that the EEFIT team were present at the dam sites.

### 3.7.1 Rudra Matha Dam



**Figure 38 Aerial view of the Rudra Matha dam [courtesy of EERI]**

This is the largest earth dam in the Kachchh region and is only a few kilometres from the bridge site described in Section 3.5.1. The construction of the dam consists of a clay core with rip-rap shell forming the upstream and downstream faces. The upstream slope is 1:3 while the downstream slope is steeper at 1:2.5. It was learnt that the rip-rap shell was not completed for the whole of the dam at the time of the earthquake, even though construction of this dam began in the 1950's. The dam was, however, functional with a low reservoir level at the time of the earthquake. In Figure 38 an aerial view of the dam (to the left of the picture), and the reservoir can be seen. As at the bridge site, extensive lateral spreading was observed at the dam site particularly near to the right-hand side of the reservoir. This however could not have resulted in any significant lowering of the reservoir capacity.



**Figure 39 Longitudinal cracks on the downstream face**



**Figure 40 The largest cracks were over 50 mm wide and extended to the core**





**Figure 41 Settlement of the crest**



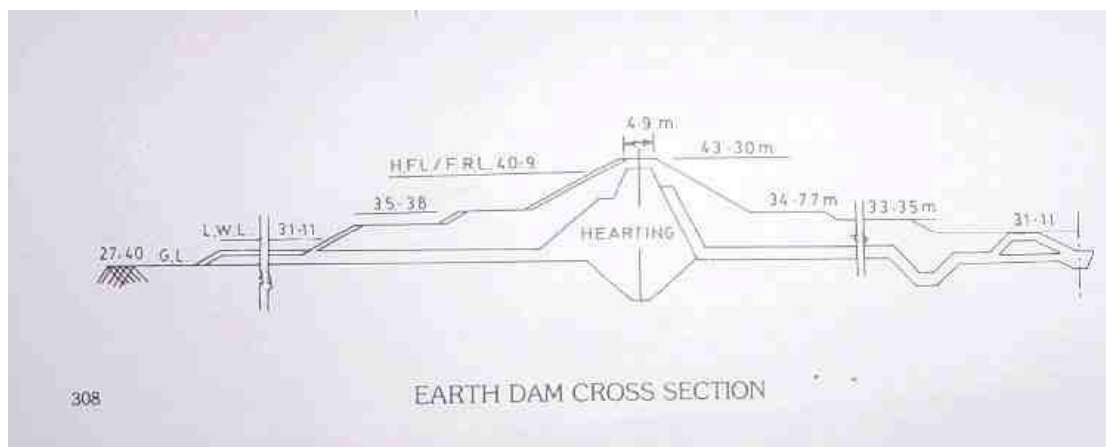
**Figure 42 Failure of the appurtenant structures**

Following the earthquake, longitudinal cracks have developed along the downstream slope close to the crest. In Figures 39 and 40 some of these cracks can be seen. The cracks ran the whole length of the dam and the largest of the cracks were around 80mm wide. The cracks appeared to extend to the core of dam.

The crest itself has suffered a settlement of about 500 mm at the centre relative to the either ends. In Figure 41 a view of the crest of the dam is presented. The control valves that open the sluice gates are located in an intake tower that suffered severe damage. In Figure 42 a view of the intake tower is shown and the walkway leading to the tower can be seen to have been badly damaged. In case of an emergency following the earthquake, it is essential for access to this structure to be maintained. Major cracks were also seen next to the box culverts that lead to the sluice gates. In Figure 41 it is interesting to note the green vegetation on the downstream side of the dam. Given that this growth stands in contrast to the semi-arid shrubbery that grows in the region, (seen in the far background of this figure), it is clear that there must have been significant seepage occurring through the dam even before the earthquake. Eyewitness accounts report an increase in water level in the ponds on the downstream side after the earthquake.

### 3.7.2 Tapar Dam

This is a medium sized earth dam with a crest height of about 15.9 m. A typical cross-section of the dam is presented in Figure 43. In this figure we can see that there is a cut-off wall that extends below the ground level.



**Figure 43 A typical cross-section of the Tapar earth dam**



**Figure 44 Longitudinal cracks extending to the core**



**Figure 45 Severe damage to the intake tower**

The location of this dam was about 60 km from the epicentre. There were no ground motion recording stations near the dam, but it should be expected that the ground motions here would have been very large (in excess of 0.3g). It is usual to have seismograph stations near dams elsewhere in the world. There may be a case to install seismographs near all major dams in Kachchh.

At the time of the earthquake the reservoir level on the upstream side was low (about 10% of reservoir capacity). Following the earthquake deep cracks were formed along the crest of the dam. The largest of these cracks were about 150mm wide and seemed to extend to the clay core. In Figure 44 a view of the longitudinal cracks running almost the whole length of the dam is presented. The upstream face and the down stream face have suffered some damage but not as significant as that suffered by the Fategadhd dam that is described next.

As in the case of the Rudra Matha dam described above, the intake tower housing the control valves to the sluice gates has been badly damaged. A view of the intake tower after the earthquake is presented in Figure 45. In this figure it can be seen that the roof covering the intake tower has completely collapsed. Also the walkway leading to the intake tower was badly damaged. It is clear that design of the appurtenant structures needs special attention given their criticality in avoiding the failure of the dam should the reservoir be full at the time of any future earthquakes.

### **3.7.3 Fategadhd Dam**

Fategadhd dam was probably the most severely damaged earth dam during this earthquake. The construction of this dam could not have been very different from the other earth dams discussed above. However, the damage suffered by this dam was significantly more than that suffered by the other dams. Proximity to the epicentre and some degree of site amplification may be some of the reasons for this.

In Figure 46 a view along the crest of the Fategadhd dam is presented. The extensive cracks that have formed in the crest can clearly be seen in this figure. The cracks were several feet wide and extend almost the whole length of the dam. It must also be noted that a significant amount of shrubbery has grown on either side of the crest (as seen in Figure 46). Even without the earthquake, presence of such vegetation is not desirable and should be removed at all dam sites.



**Figure 46 Extensive cracking of the crest** **Figure 47 Failure of the upstream slope of the dam**  
[courtesy of IIT Kanpur] [courtesy of IIT Kanpur]



**Figure 48 Deep cracks following the failure of the upstream slope**  
[courtesy of IIT Kanpur]

The upstream slope suffered severe slip failure. A view of this slope is presented in Figure 47. In this figure we can see the head of the slope separating out from the crest. Some amount of liquefaction has occurred at the base of the upstream slope. This would have resulted in a loss of 'toe support' to the slope resulting in a clear slip failure.

The downstream slope has also failed. Large cracks that are around 600mm wide and nearly 1.2m deep have opened up on this face. A view of these large cracks is presented in Figure 48. Again the downstream slope seemed to have lost toe support due to softening of foundation soil near toe following liquefaction. This resulted in the significant deformation of this slope.

### **3.8 Summary**

The Bhuj earthquake has many interesting geotechnical aspects. Even though the strong motion data is almost non-existent, the ground accelerations in Ahmedabad city seem to be large in relation to the distance from the epicentre of the earthquake. There is a need to understand the existence of any basin effects due to the topography and geology of this region. This may be important in re-evaluating the

seismic zonation of the areas surrounding Ahmedabad and in establishing recommended peak ground accelerations for designing against future earthquakes.

Liquefaction was observed over very large areas in the Kachchh region. Sand boiling due to soil liquefaction was observed over vast stretches with characteristic salt crustation when dried up. In the Rann of Kachchh this is to be expected due to the presence of saline water close to the ground surface. At other places where there were reports of potable water turning saline, the ground levels and hydrology need to be established accurately to establish if there was any change in the ground elevation. This suggests liquefaction of deeper soil strata. It is possible that the increase in the ground water pressure may be due to settlement of ground between faults rather than the traditional excess pore water pressures generated due to volumetric strains in the soil under cyclic loading.

Liquefaction played an important role in the damage to many civil engineering structures in the Kachchh region and to the Navalakhi port in the Saurashtra region. The Bhuj earthquake provided some interesting insight on the performance of bridges with foundations located on liquefied soil. In particular the Bhachau-Vondh bridge site where there were four bridges of different age and type of construction was very interesting. The superstructure stiffness of the bridges appeared to determine the likely mechanism of failures once the foundations have liquefied. The arch bridge was vulnerable to differential settlement of the piers, whilst a more modern plate girder bridge was susceptible to torsion with piers rotating about the longitudinal axis of the bridge. Higher approach embankments also seem to have caused large lateral forces on the bridge decks in the case of the new highway bridge. Once the foundation soil has liquefied, the abutments could not resist these large lateral forces. The new bridge at Surajbari provided a good example of soil-structure interaction during this earthquake with the foundation soil participating in the vibrations undergone by the bridge piers and decks.

Liquefaction effects were relatively small at the large port of Kandla, with only the harbour master's office building suffering rotation as the pile foundations settled. Lateral spreading was also evident at this site. The old and new jetties have performed well, even though the backfill soil has shown signs of subsidence. In contrast Navalakhi port, almost directly across the Gulf of Kachchh, has seen widespread damage to the jetties and the service areas. The railway line feeding this port has been damaged badly due to extensive lateral spreading. This port was constructed from material dredged from the gulf and there are lessons to be learnt in terms of the compaction of material used in backfill during construction of similar ports. Examples of lateral spreading were present at other sites even where the slope angle of the ground was as low as  $3^{\circ}$  to  $5^{\circ}$ . This confirms the results that were obtained in the recent research into lateral spreading (Haigh et al, 2000).

Earth dams in the Kachchh region have seen severe damage during this earthquake. Longitudinal cracking, failure of the up and downstream slopes, damage to appurtenant structures including the intake towers and walkways leading to the intake towers were observed. The reservoir levels in most of the earth dams were low at the time of the earthquake and hence no inundation occurred. However the design of appurtenant structures, particularly those that will be brought into service when dam safety has to be ensured, must be carried out to withstand large earthquakes such as this. Liquefaction resistance measures need to be undertaken at the upstream and downstream toes of the dam to ensure slope failures do not occur on the scale witnessed in this earthquake.

### **3.9 References:**

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# Chapter 4 Performance of Bridges, Ports and Industrial Facilities

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## 4.1 Introduction

### *Industry*

Gujarat contributes approximately 11% of India's industrial output. The state is a major producer of chemical and pharmaceutical products, cement, steel, lignite, diamond jewellery, salt, soda ash and handicrafts. The state government estimates that earthquake damage to small/medium enterprises (SME's) is around Rs3.25 billion (£48 million), and about Rs1.61 billion (£24 million) for large enterprises. The earthquake damage was largely confined to the districts of Bhavnagar, Jamnagar, Kachchh, Rajkot and Surendranagar, whereas the main industrial belt follows the railway corridor between Ahmedabad and Surat. Nevertheless, these five worst affected districts employ around 360,000 people in the industrial sector and account for 23% of Gujarat's industrial employment. (World Bank, 2001). However, much of the industry is small scale and there are relatively few large facilities.

### *Infrastructure*

The length of the road network in Gujarat is approximately 72,000km, and the rail network (metre gauge and broad gauge) is 5,227km. The Government of Gujarat has decided (Gujarat Industry) to upgrade to broad gauge the metre gauge track which runs north-east from the district of Kachchh towards Delhi and other parts of northern India. This project is due to commence in 2002 with the 307km stretch from Gandhidham to Palanpur. The new railway link will not only connect the Kachchh seacoast with North India, but should also provide business opportunities to the earthquake devastated towns of Anjar, Bhachau and Rapar.

There are 10 airports, one of which, Ahmedabad, has international flights.

Gujarat has a coastline of approximately 1600km, and 41 designated ports. Kandla is the only major port; a further 11 are classed as intermediate and the remainder are small ports. Overall, Gujarat ports handle about 23% of the national shipping cargo.

Road and rail infrastructure in the worst affected region is limited.

## 4.2 Bridges

### 4.2.1 Introduction

The team travelled from Bhuj to Jamnagar by road, skirting the Gulf of Kachchh and crossing a major tidal creek at Surajbari. It was on this route, on National Highway 8A, that the most significant damage to bridges occurred. The NH8A continues on to Ahmedabad, but reports from another reconnaissance team (Singh M. P., Saraf V. K., and Jain S. K., 2001) indicate that there was little or no damage to bridges beyond Surajbari.

A number of the bridges built in the 1960s are in a poor state due to reinforcement corrosion induced by the high salt content in the ground, water and air. At two locations on the NH8A new highway bridges were under construction and almost complete at the time of the earthquake. This allows a comparison between the performance of old and new designs.

At the larger bridges most significant damage related to decks shifting on their supports, and to foundation failure with shifting of supports due to liquefaction. Some smaller bridges failed due to the use of unreinforced masonry supports.

There has been considerable development internationally in the seismic design of bridges during the last 15 years, but this had not been reflected in the Indian earthquake codes at the time of the Bhuj earthquake. The earthquake provided impetus for the early publishing of improvements. A brief review is included of the Indian Earthquake code as it related to bridges at the time of the earthquake.

#### **4.2.2 Rudra Matha (Khari) Bridge**

This bridge is on the road from Bhuj to Lodai, the closest town to the epicentre, and is probably no more than 10 - 15km from the epicentre. The structure was completed in 1966 and carries the highway over a lake. It has 10 simply supported spans of 55 feet (16.8m) supported on reinforced concrete A-frame piers which are built into capping slabs on top of approximately 4m diameter well caissons (see Figure 1).



**Figure 1 Rudramata (Khari) Bridge – view from north side**

Given its proximity to the epicentre, this structure performed remarkably well and suffered only minor damage, which can be summarised as follows:

- there was some settlement behind the north abutment and a section of parapet had fallen off the wing wall (see Figure 2);
- a pier bearing plinth close to the north end of the bridge had fractured (see Figure 3); and
- there had been settlement and lateral spreading of the soil in the river bed, but with no apparent shift of the caisson foundations. Figure 4 shows signs of soil movement around the pier caisson closest to the north end of the bridge. Geotechnical aspects of this site have already been discussed in Section 3.5.1.



**Figure 2 Rudramata (Khari) Bridge – settlement behind north abutment and missing section of parapet**



**Figure 3 Rudramata (Khari) Bridge – fractured bearing plinth**



**Figure 4 Rudramata (Khari) Bridge - settlement and lateral spreading of the soil, but caisson unaffected**

#### **4.2.3 6-Span Bridge between Bhuj and Bachau**

Some 42km to the east of Bhuj, near a town called Dudhai on the road to Bhachau, this bridge is less than 25km south-east from the epicentre. The bridge is oriented approximately east-west, and has six spans of reinforced concrete beam-and-slab deck supported on reinforced concrete piers (Figure 5). It was probably constructed in the 1960s, and almost certainly has traditional well caisson foundations.

Some liquefaction had occurred in the dry river bed (Figure 6), and residues and marks at the base of piers and abutments indicated the amount of settlement of the soil which had taken place relative to the substructure. For example, at the east abutment there were signs of settlement of the ground in front of the abutment (Figure 7). Reinforcement corrosion was quite advanced, and so the shaking and the impact between deck and abutment caused quite severe damage to the abutment ballast wall and its parapets.

The deck had hit the abutment - partly due to shaking of the deck and partly to the abutment moving forward under increased earth pressure combined with ground softening below and in front of the abutment. There had been some pounding between the wing wall and its extension, and some damage to the beam soffit at the bearing.

The steel rocker bearings had successfully resisted transverse deck shaking but in doing so had caused damage to beams at a number of locations (Figure 8). The violence of the shaking would have caused large cyclic shear loads which would result in the transfer of large bursting loads from the bearing and its fixings into the beam. However, the observed beam failure mode may also be associated with high vertical bearing loads, or a combination.

All piers had vertical cracks caused by reinforcement corrosion possibly exacerbated by splitting forces from corroded and locked up bearings (Figure 9). Again, at the base of the pier it can be seen that settlement of the bed of about 150mm has taken place.





**Figure 5 Six Span Bridge near Dudhai – oriented east-west, 25km from epicentre, probably constructed in the 1960s**



**Figure 6 Six Span Bridge near Dudhai – signs of liquefaction in the dry river bed**



**Figure 7 Six Span Bridge near Dudhai – settlement of the ground in front of the abutment, impact damage to ballast wall and loss of reinforced concrete parapet**



**Figure 8 Six Span Bridge near Dudhai – damage to beams at bearings**





**Figure 9 Six Span Bridge near Dudhai – vertical cracks in ends of piers probably due to reinforcement corrosion possibly made worse by splitting forces from corroded and locked up bearings**

#### **4.2.4 Four Bridges Site**

Just to the east of Bhachau on the NH8A there were four bridges over a dry river bed - two road bridges and two rail bridges. Both road and rail were orientated approximately east-west, and the site was some 50km east of the epicentre.

##### **a) Road Bridges**

Figure 10 shows the two road bridges - an old bridge in use by traffic in the foreground and a new bridge at a higher level which had been almost complete at the time of the earthquake.

The old bridge was in a very poor state of repair (Figure 11) and an earlier attempt had been made to reduce the rate of deterioration by applying a coat of sprayed concrete. The earthquake had shaken off much of the coating and the concrete cover, which had already been delaminated by the bursting pressure of corroding reinforcement. Long lengths of the parapets had also been lost and it can be seen that there was severe damage to the abutment at the east end of the bridge.

The new road bridge comprised simply supported reinforced concrete spans. Unusually, there was a change in deck form between the internal spans and the end spans - internal spans were of beam-and-slab construction, while the short end spans were solid slabs.

Looking at the east abutment of the new bridge from beneath the end span it was immediately clear that the wing wall had moved forward relative to the main abutment wall (Figure 12). In the bottom right of

the photo some fill material can be seen which has spilt through the gap that had opened up between the nearby wing wall and the abutment. From the residue and staining on the face of the abutment and the pier it appeared that the soil level had dropped by almost 500mm. At the end pier the first internal span appeared to have driven hard against the step at the top of the pier and the pier had tilted as a result of the impact.

Simply supported spans are considered vulnerable to unseating and collapse. In this case it was clear that the short end span was only saved from collapse by the presence of the stepped pier top. If the end span had been the same depth as the internal spans, then it would have been pushed off the top of the pier. Because the internal span hit the stepped bearing shelf it caused the pier top to move with the spans and avoided loss of support.

It was only when the abutment was viewed at deck level (Figure 13) that the full failure mechanism became clear. In fact, the whole abutment and embankment had moved forward by about 500mm and in the process the ballast wall collided with the deck and was broken off the top of the main wall. The approach slab which was supported on the ballast wall was pushed back relative to the embankment and the subgrade material (already in place awaiting the first asphaltic layer) was pushed into a heap by the approach slab. The first pier had also been shifted towards the “river” by the flow of liquefied soil.. The figure standing on the approach slab provides scale in Figure 14 which shows the tilted ballast wall, and the hump of subgrade material at the end of the approach slab is also given scale by a human figure (Figure 15).

At the western end, a similar mechanism has developed with the abutment shifting forward, but only by about 200mm. Nonetheless, the deck collided with the ballast wall (Figure 16)



**Figure 10 Four-Bridges Site on NH8A just to east of Bhachau - showing the old and (almost complete) new road bridges**



**Figure 11 Four-Bridges Site on NH8A just to east of Bhachau – old road bridge showing badly deteriorated concrete, damage to east abutment, loss of sprayed concrete and parapet on ground. This bridge was carrying all the road traffic on the NH8A**



**Figure 12 Four-Bridges Site on NH8A just to east of Bhachau – new (almost completed) road bridge: collision of deck with pier upstand, wing wall has moved forward relative to abutment, marks on pier and abutment show about 500mm settlement of soil**





**Figure 13 Four-Bridges Site on NH8A, new road bridge: east abutment had moved forward by about 500mm, ballast wall had collided with end of deck, and fill had settled**



**Figure 14 Four-Bridges Site on NH8A, new road bridge: tilted ballast wall**



**Figure 15 Four-Bridges Site on NH8A, new road bridge: hump of subgrade material at end of approach slab**



**Figure 16 Four-Bridges Site on NH8A, new road bridge: the west abutment had moved forward by about 200mm and ballast wall had also been damaged, but not as severely as at the east end abutment**

b) Rail Bridges



Just to the north of the road bridges are two rail bridges, one carrying a metre gauge track and one carrying a broad gauge track. The metre gauge track links Bhuj to Rajasthan to the north-east, and the broad gauge track links Kandla Port and Gandhidam to Ghandinagar near Ahmedabad.

Figure 17 shows both; on the left is a seven span masonry arch bridge carrying the single track metre gauge railway, reportedly opened in 1912. On the right is a more modern steel girder bridge on masonry piers carrying a single track of broad gauge railway.

The old masonry arch bridge had suffered considerable damage in the earthquake:

- a length of the longitudinal side wall on the south side had collapsed and had already been repaired (the repaired section can clearly be seen in Figure 17 (see above) where the white edged coping stones have not yet been replaced);
- the longitudinal side wall on the northern side had bulged (Figure 18); and
- arch blocks had loosened in both end spans (Figure 19). The blocks had been pushed back into position, and cracks between blocks had been filled by injection of epoxy resin injection (Figure 20);
- masonry wing walls had been badly damaged (Figure 21).

The current Indian earthquake code precludes the use of masonry arch bridges with spans of more than 10m, which is about the span of the arches in this old metre gauge bridge.

The steel beam bridge appeared to have performed much better. There had been some modest displacements of the substructure due to the softening of foundation soils and some settlement of fill behind the abutments. It appeared that this had been easily repaired by the addition of ballast under the track, and there were no signs of any shimming under the girders.



**Figure 17 Four-Bridges Site on NH8A, metre gauge and broad gauge rail bridges: masonry arches support metre gauge, and steel girders support broad gauge. Spandrel wall had collapsed and been reconstructed where white coping stones are missing. Note bulges at crown of arches on this south face.**



**Figure 18 Four-Bridges Site on NH8A, metre gauge rail bridge: bulging of spandrel wall on north side of bridge**



**Figure 19 Four-Bridges Site on NH8A, metre gauge and broad gauge rail bridges: joints between loosened arch blocks had been epoxy injected in west end span. Note it had not been possible to jack blocks back into original position**





**Figure 20 Four-Bridges Site on NH8A, metre gauge rail bridge: cracks between arch blocks had been injected with epoxy resin – note the surface sealing mortar and injection nipples**



**Figure 21 Four-Bridges Site on NH8A, metre gauge rail bridge: at east end of bridge masonry wing wall had been badly damaged and joints between arch blocks had been injected with epoxy**

#### **4.2.5 Surajbari Bridges**

- a) Introduction

The existing and the almost completed highway bridges at Surajbari are the most important bridges on the NH8A. The bridges are orientated approximately north-west to south-east, and are situated some 60-70km east of the epicentre. Apart from personal observations, information on these bridges was provided by Mahesh Tandon (Goel R. and Tandon M. C., 2001) of Tandon Consultants PVT Ltd and Dr Nayak of Gammon India Ltd.

b) The Old Bridge

The old bridge (Figure 22) was opened to traffic in the late 1960s, and was the first direct link between the regions of Kachchh and Saurashtra in the state of Gujarat. Without this crossing a detour of 350km must be made using the NH15. It has 37 spans of 32.2m giving an overall length of 1191m. The form of construction is a two-cell reinforced concrete box girder, with alternate spans comprising a simply supported span with cantilevers to the third point of the adjacent spans, and drop-in spans between the cantilevers to complete the deck. The half-joints have unusual inclined interfaces. The substructure comprises reinforced concrete piers on well caissons.

Considerable deterioration had occurred to the structure due to reinforcement corrosion in the extremely hostile environment of salt water combined with high temperatures. A few years previously the structure had been given a coating of sprayed concrete in an attempt to slow the corrosion process.

During the earthquake there was pounding at deck joints, many of the steel bearings were dislodged at pier supports, and Pier P12 tilted in the longitudinal direction, although it is reported that the well caisson was not displaced. This pier required urgent strengthening before the bridge could be re-opened to traffic (Figure 23). The pounding at deck joints resulted in spalling of concrete coating and cover adjacent to joints (Figure 24); the spalled area was greater than might have been expected in a sound structure as much of the concrete was already delaminated due to corrosion. The failure of the bearings meant that the deck displaced vertically and horizontally, with the horizontal displacement reaching a maximum of about 1.5m at Pier P15 (Figure 25).



**Figure 22 Surajbari Old Bridge: 37 spans of 32.2m, reinforced concrete box girder supported on r.c. piers and well caissons**



**Figure 23 Surajbari Old Bridge: Pier 12 tilted longitudinally and had to be strengthened before re-opening to traffic**



**Figure 24 Surajbari Old Bridge: pounding at joint caused spalling of concrete coating and cover. Note severely corroded reinforcement. [Photo courtesy of Mahesh Tandon].**



**Figure 25 Surajbari Old Bridge: deck displaced vertically and horizontally, horizontal displacement was a maximum of about 1.5m at Pier P15. [Photo courtesy of Mahesh Tandon].**

c) The New Bridge



The new bridge, which is 150m upstream of the old bridge, has 39 spans of 32.2m giving an overall length of 1256m. The deck comprises three prestressed I-beams with an in-situ topping, and the substructure consists of twin leaf piers on well caissons (Figure 26).

The twin leaf piers have additional cantilevers to accommodate two possible future water mains of 1.5m diameter. The span between the centrelines of the 8.4m diameter caissons is 32.2m, and the span between bearings is 25.6m. The I-beams have 3.3m long cantilevers to the centre of the caissons.

Neither the caissons nor the piers suffered any permanent displacement. The decks, however, were displaced on their elastomeric bearings with shifts in some places up to 200-250mm. There had been some damage to deck ends due to pounding between adjacent spans (Figure 27), and many girders had collided with stopper blocks which were badly damaged by the violent impact (Figure 28 and Fig4.29). As a result of the displacements some of the rubber inserts at deck joints were badly damaged (Figure 30) and had to be replaced.

Widespread liquefaction and lateral spreading of the surface layers of ground below and adjacent to the embankments had caused considerable damage to the approaches (Figure 31). At the Gandhidam end of the bridge there was 1.3m settlement behind the abutment (Figure 32), and at the other end of the bridge 0.5m settlement.

Because of the precarious condition of the old bridge there was considerable urgency to repair the new bridge and divert traffic onto it, albeit using the approaches to the old bridge until such time as the new approach embankments could be reconstructed. Work to shift the girders back to their correct position was already in progress. Four jacks, each of 100t capacity were used at both ends of the span. A Hillman roller assembly was used to translate the deck (Figure 33). The bearings were tested and if necessary replaced – it was found that only 30% could be re-used. The stopper blocks were being reconstructed with additional reinforcement.

Given the magnitude of the earthquake and low expectations of the Indian earthquake code, the new bridge behaved remarkably well.



**Figure 26 Surajbari New Bridge: 39 spans of 32.2m, prestressed beams on twin leaf r.c. piers and well caisson foundations**



**Figure 27 Surajbari New Bridge: pounding between ends of adjacent spans had caused some damage**



**Figure 28 Surajbari New Bridge: many shear stopper blocks were badly damaged. [Photo courtesy of Mahesh Tandon].**



**Figure 29 Surajbari New Bridge: damage to shear stopper block. [Photo courtesy of Mahesh Tandon].**



**Figure 30 Surajbari New Bridge: deck joints were moved beyond their capacity and some rubber seals were torn out. [Photo courtesy of Mahesh Tandon].**



**Figure 31 Surajbari New Bridge: liquefaction of the underlying soil caused settlement and cracks in the approach embankments**



**Figure 32 Surajbari New Bridge: liquefaction caused settlement of 1.3m behind the abutment at the Gandhidam end of the bridge. [Photo courtesy of Gammon India Ltd].**



**Figure 33 Surajbari New Bridge: decks had been displaced up to a maximum of 250mm. After jacking a Hillman roller assembly was used to move the deck back to its correct position. [Photo courtesy of Mahesh Tandon].**

#### **4.2.6 Minor Bridges**

Several small reinforced concrete slab bridges with spans generally in the range 3m to 6m constructed on rubble masonry wall abutments were observed. These structures fared badly in the earthquake. Figure 34 shows an example that was a few kilometres northwards from the Rudra Mahta Bridge towards the epicentre. The rubble masonry had collapsed due to the self inertial load, combined with the high cyclic friction loads from the heavy concrete slab, and the increased earth pressures.

Another example (Figure 35) was this 6m span structure about 10km west of Bhachau at Sikara. Its orientation was east-west, and the earthquake had shifted the deck southwards on its concrete pad-stones. The abutment walls were about 1m thick.





**Figure 34** Small reinforced concrete slab bridge on road from Bhuj to Lodai: masonry abutments were destroyed by violence of shaking



**Figure 35** Small reinforced concrete slab bridge structure 10km west of Bhachau at Sikara. East-west orientation and earthquake had shifted the deck southwards

#### **4.2.7 Indian Earthquake Code**

Seismic design of bridges in India is covered by three codes (Murty, 2003): IS 1893 (1984) from the BIS, IRC 6 (2000) from the Indian Roads Congress, and Bridge Rules (1964) from the Ministry of



Railways. The Code of Practice “IS : 1893 – 1984 Indian Standard: Criteria for Earthquake Resistant Design of Structures (Fourth Revision)” is the core document, and its development up to the Bhuj earthquake can be summarised as follows (Jain and Nigam, 2000):

- IS : 1893 – 1962 first formal zone map divided the country into seven seismic zones (0 to VI) corresponding to areas liable to MM intensity of: less than V, V, VI, VII, VIII, IX, X and above, respectively.
- IS : 1893 – 1966 first revision; the zone map was revised somewhat.
- IS : 1893 – 1970 second revision: a major revision which reduced the number of zones from seven to five (I to V).
- IS : 1893 – 1975 third revision: seismic zone factors rationalised and importance factors introduced.
- IS : 1893 – 1984 fourth revision: load factors modified and concept of “Performance Factor” introduced.

In the Foreword to IS 1893 - 1984 it stated:

*“It is pointed out that structures will normally experience more severe ground motion than the one envisaged in the seismic coefficient specified in this standard. However, in view of the energy absorbing capacity available in inelastic range, ductile structures will be able to resist such shocks without much damage. It is, therefore, necessary that ductility must be built into the structures since brittle structures will be damaged more extensively.”*

It is normal for codes to adopt force levels that are less than the actual anticipated levels, but good practice requires a coherent strategy for ensuring safety in the event of the design level earthquake.

The decision has been taken to split the single brief IS 1893 document into five parts: Part 1 General Provisions and Buildings; Part 2 Liquid-Retaining Tanks; Part 3 Bridges and Retaining Walls; Part 4 Industrial Structures including Stack- Like Structures; and Part 5 Dams and Embankments. Part 1 was published in 2002, and included a reduction from five to four seismic zones. At time of writing the other four documents are still under preparation. Also prompted by the Bhuj earthquake, the Indian Roads Congress published Interim Provisions to improve its IRC 6 (2000) in 2002.

To indicate how the Indian code design force levels compared with international practice, the design requirements for the most severe Indian zone (IS : 1893 Zone V) have been compared with the requirements for the US as set out in the AASHTO 16<sup>th</sup> Edition and the AASHTO LRFD codes using a seismic coefficient  $A = 0.3$ . This comparison is not scientific in terms of equating seismicity. However, given that India has experienced some of the world’s greatest earthquakes and bearing in mind that US requirements can go to  $A = 0.8$  and beyond close to faults, the results set out in Table 1 below do highlight how low the Indian requirements appear.

|   | IS 1893 : 1984 | AASHTO 16 <sup>th</sup> Ed | AASHTO LRFD |
|---|----------------|----------------------------|-------------|
| <b>Deck/Pier Connection</b><br>(Elastic: e.g. strong direction for wall pier) | 1              | 6.25                       | 6.25        |
| <b>Deck/Pier Connection</b><br>(incl longitudinal hingeing in pier)           | 1              | 2.7                        | 4.2         |
| <b>Framed Pier:</b><br>In-plane peak moments<br>(incl R)                      | 1              | 1.25                       | 1.75        |
| Out-of-plane peak moments<br>(incl R)   | 1              | 2.1                        | 3.2         |
| <b>Wall Pier:</b><br>In-plane peak moments<br>(incl R)                        | 1              | 3.2                        | 4.2         |
| Out-of-plane peak moments<br>(incl R)   | 1              | 2.1                        | 3.2         |

**Table 1 Comparison of Code Load Requirements  
(IS : 1893 Zone V vs AASHTO A = 0.3)**

**Note:** for simplicity this comparison assumes only dead load applies, which is not strictly true for transverse load effects under IS 1893 : 1984.

The inadequacy of the requirements for the transverse connection between deck and pier were clearly demonstrated in the case of the new Surajbari Bridge.

Designers are given no clue as to what force level might be expected for given return periods. Hence there is no guidance on what level of ductility must be built in, nor where to build it in, nor how to achieve that ductility. Nor does the code give advice on what safeguards should be applied to avoid the loss of spans under the increased displacements required by the implied post-elastic demand.

#### 4.2.8 Conclusions (Bridges)

Observations of the performance of bridges and a brief review of the Indian Earthquake Code lead to the following conclusions:

- Liquefaction and lateral spreading of founding soils led to some serious damage, particularly to those structures which were under construction.
- In a number of instances decks shifted on supports. On this occasion no collapses were observed as a result.
- Decks were generally simply supported with no structural redundancy. However, there should be no complacency. As stated above, the reason why there were no catastrophic collapses due to loss of support related more to good fortune rather than good robust engineering practice. For example, the short end span of the new highway bridge at the 4-bridges site on the NH8A was close to collapse. It would certainly have collapsed if the end pier had not had the unusual upstand and foundations should be better suited to the subgrade conditions.
- A somewhat unexpected finding was that the earthquake highlighted the weakness of the commonly used reinforced concrete parapets. In several cases we found that the parapets had

simply been shaken off because either the connection to the deck was poorly designed, or corrosion had weakened the reinforcement at the base of the posts to such an extent that they could not even sustain their own inertia forces. Appendages such as parapets commonly experience significant seismic amplification, but their collapse must raise the general question regarding the effectiveness of these widely used parapets to contain errant vehicles.

- The traditional caisson foundations performed very well.
- Comparison between the requirements of Indian code for Zone V with the US AASHTO requirements for  $A = 0.3$  indicates that the minimum seismic demands prescribed by the Indian code are significantly less than would be required by US practice for bridge design. The difference is most marked in the stipulated deck/pier connection, for which the US requirement was a factor of more than 6 greater than the Indian requirement. Although seismicity has not been rigorously equated in this comparison, the fact that some of the world's greatest earthquakes have occurred within India in recent history supports its validity. Close to known faults the AASHTO requirement rises to  $A = 0.8$  which implies a factor of more than 16 between the two codes.
- While acknowledging that the required design loads are much less than those which might be expected in a strong earthquake, the current Indian code is based on a return period of 475 years for the design earthquake, but gives no clue as to how the design earthquake may vary with return period, and no coherent design strategy for ensuring resilience and a ductile response.

## **4.3 Ports and Harbours**

### **4.3.1 Introduction**

Only two ports, Kandla and Navlakhi, suffered significant damage. Minor damage was reported at Okha, Porbandar and Bhavnagar, although no details are available. The major crude oil import facility at Vadinar was undamaged, as was Sikka.

### **4.3.2 Kandla**

Kandla Port handles approximately 46.3 million tons of cargo annually. The port is therefore of considerable economic and strategic importance to the country, and to Gujarat in particular. Kandla was founded in 1952 and is the 6th major port in India. It serves a hinterland of 1 million square kilometres and deals in bulk cargoes such as coal, sugar, timber, granite, marble, potash, steel coil, containers and oil.

#### **a) Briefing by Chairman of Port Trust**

Prior to visiting the port the EEFIT Team met with Mr. A.K. Jyoti, Chairman of Kandla Port Trust at his temporary office. As a result of the damage to port buildings, the port administration had been moved to offices near Gandhidham, several kilometres from the port. This sub-section summarises the briefing by Mr Jyoti.

The port has 10 cargo berths along a 2km long quay. There are large storage facilities for liquid and dry cargo, 4 jetties for handling petroleum products and chemicals, and 2 single buoy moorings near Jamnagar for very large crude carriers (VLCCs).

Following the earthquake, the first action taken by the Port Trust was to check that the channel which leads to and from the port was clear. At the time of the earthquake thirteen vessels were waiting to use the channel. A coastguard helicopter crew assisted by carrying out a survey from the air on 27 January; and that survey indicated that the channel was clear and fit for navigation. During the 12 days prior to the EEFIT visit, 1.11 lakh tons (111,000 tons) of cargo had been handled.

The next problem for the port management was the inspection of the piles under the berths. There are a total of around 3000 piles supporting the berthing jetties. There are 10 berths, five of which are between 30 and 45 years old, and the other five were built within the last 20 years. The older berths

were designed to carry  $3.3\text{t/m}^2$  ( $33\text{kN/m}^2$ ), and are founded on 2ft (600mm) wide hexagonal piles with an embedment length of 20m into the sea bed down to dense sands. The more recent berths are designed for a live load of  $5\text{t/m}^2$  ( $50\text{kN/m}^2$ ), and are founded on 1000mm diameter circular reinforced concrete piles which extend some 30m into the sea bed and are founded within the layer of dense sand.

Engineers from the Ministry of Surface Transport carried out a post-earthquake structural survey. They found that there were cracks in the older 600mm piles, but that the later 1000mm diameter piles were undamaged. There was some damage to the go-downs (warehouses). Due to the condition of the piles and deck, the assessment downgraded the safe capacity of the older berths from 3.3 to 1.5t/sq m, and so the height of stacked materials had to be kept down. To limit lateral loads, only the smaller vessels were being docked at the older berths.

The cost estimate to repair damage to the port following the earthquake totalled 51 crore rupees (approximately £8m).

b) Observations during Visit to Kandla Port

*Main Berths and Adjacent Filled Ground*

The 10 berths run north-south. Access was possible to the underside of the latest section of the quay which was completed 4 - 5 years ago. The quay is supported on 11 rows of 1000mm diameter reinforced concrete piles cast inside permanent steel casings (Figure 36). The connection at the pile head looked to be less than a full moment connection, and was probably assumed pinned by the designer. Square capitals had been cast onto the pile heads, and these supported a grillage of precast beams onto which was cast the in-situ deck. On the landward edge of the jetty, a retaining wall had been cast integral with the edge row of piles (Figure 37). The piles, the deck and the retaining wall all looked in reasonable condition, and did not appear to have shifted significantly during the earthquake.

The material at the surface beneath this berth appeared to be a silty clay which was stiff enough to walk on. Soil borings adjacent to berths 6 and 7 were reported to indicate 5 to 10 m of soft silty clay at the surface underlain by sand. There were some gunny sacks of sand and cement which had probably been placed to prevent scour in front of the retained fill (Figure 38).

At the end of the jetty the soil appeared to be more a silty sand. Some of this material seemed to have spread from the fill behind the jetty. At this end of the jetty the level of the retained soil had dropped by more than 600mm; partly settlement and partly loss of material by lateral spreading (Figure 39). For much of the length of the berths the retained fill adjacent to the berths had settled by 150-300mm. Whether this was pure settlement, or was accompanied by some lateral spreading beneath the retaining wall was not clear. The fill material was reported to be decomposed rock placed directly onto the silty clay upper strata of the original ground.

It was not possible to gain access to the underside of the older berths. The team could only inspect them from the pontoon of a floating maintenance jetty. The 600mm piles connect into reinforced concrete framing which supports the decking some 3-4m above. The most northerly berth appeared to have undergone substantial refurbishment prior to the earthquake (Figure 40). Inspection reports by others (EERI, 2001) describe and illustrate the damage to the 600mm diameter cylinder piles (Figures 41 and 42). Typically they suffered circumferential cracks about 300mm below their connection with the reinforced concrete framing members, with the cracks wider on the landward side. This could have been caused by lateral spreading of the seabed material.





**Figure 36** Kandla Port: new berth completed 4 - 5 years ago - 1000mm diameter reinforced concrete piles cast inside permanent steel casings. Structure appeared to have performed well with virtually no damage or displacement.



**Figure 37** Kandla Port new berth: retaining wall cast integral with landward edge row of piles. Material had been lost at this free corner due to settlement and lateral spreading. See also Figure 39.



**Figure 38 Kandla Port new berth: gunny sacks of sand and cement had been placed during construction to prevent scour in front of the retained fill**



**Figure 39 Kandla Port new berth: loss of material at end of new berth**





**Figure 40 Kandla Port: oldest berths at north end had been refurbished before the earthquake**



**Figure 41 Kandla Port: 600mm diameter cylinder piles had cracked circumferentially about 300mm below their connection with the r.c. framing members. [Photo courtesy of EERI].**



**Figure 42 Kandla Port: a close-up of crack in 600mm diameter cylinder pile. [Photo courtesy of EERI].**

*Damage to Jetties for Launches and to Armoured Slope*

Down at the other end of the berths, to the north of oldest berths and close to the entrance gate, the team examined a launch jetty which comprised 8 x 4m spans on concrete piles together with a steel truss (Figure 43). The truss was restrained transversely by steel piles, and one end had previously been hinge supported on the piled structure with the free end on a floating pontoon. The steel truss had dropped off its support ledge on the concrete jetty onto bracing between the steel pile guides, and some of the cover to the concrete piles (already partially delaminated by corrosion) had spalled. The truss collapse was not the result of the earthquake, but of a cyclone which hit Kandla on 9 June 1998.

The EEFIT team went onto another similar launch jetty which had a steel truss hinged at one end and supported on a floating pontoon at the other end. The pontoon had previously been restrained to move up and down with the tide by slotted brackets fixed to the pontoon which fitted around vertical rails attached to steel piled frames at each end of the pontoon. One end of the pontoon had become partly submerged and the brackets had broken free from the rails. The link span had jumped from its guides and punched into the adjacent deck plates (Figure 44). The pontoon guides were also damaged (Figure 45), and the pontoon was partly submerged. The concrete piles supporting the landward spans of this jetty showed signs of spalling which had been originally induced by corrosion, but which had been exacerbated by earthquake shaking.

The armoured earth slope between these two maintenance jetties had longitudinal cracks showing that despite the gentle slope (approximately 1:4) there had been some lateral spreading of the fill (Figure 46).

To the north of the control tower another jetty for small craft had been badly damaged (Figure 47). This jetty was L-shaped in plan, and many of the raking reinforced concrete piles had ruptured close to the jetty deck. It was not possible to get close to this jetty, but the structure looked as if it had been in a parlous condition prior to the earthquake (possibly damaged by the 1998 cyclone), and reinforcement corrosion had considerably reduced its resistance to the earthquake.





**Figure 43 Kandla Port, launch jetty: steel truss dropped off its support ledge during a devastating cyclone which hit Kandla on 9 June 1998.**



**Figure 44 Kandla Port, launch jetty: link span had jumped from its guides and punched into the adjacent deck plates**



**Figure 45 Kandla Port, launch jetty: pontoon guides were also damaged**



**Figure 46 Kandla Port: armoured slope had longitudinal cracks showing there had been some lateral spreading of the fill**





**Figure 47 Kandla Port: jetty for small craft to north of the control tower had been badly damaged by the cyclone and then the earthquake. Reinforcement corrosion had already weakened the structure.**

#### *Bulk Liquids Facilities*

The bulk liquids are piped off site to tank farms around the port boundary which are operated by the various owners. The tanks appeared unaffected, although the team could not gain access for close inspection. However, the EERI team noted the following damage (EERI, 2001):

- one tank with a circumferential rupture between the wall and floor
- two tanks with 50-75mm settlement, but no pipe breakages
- one tank where the settlement ruptured the connecting pipework
- a further tank with uneven settlement, resulting in 300mm transverse movement

#### *Port Buildings*

The 2-storey Central Industrial Security Force (CISF) building parallel to the waterfront had lost its lower storey due to soft-storey collapse of the circular columns (Figure 48). The columns had been poorly detailed with laps in the maximum moment zone, and virtually no confining reinforcement – 6mm diameter mild steel hoops at about 250mm centres (Figure 49).

The 10m square 6-storey control tower building, just to the north of the collapsed 2-storey block, looked intact, but had suffered foundation failure and was inclined by a few degrees towards the water (Figure 50). The top of a corner pile had been exposed by excavation, and there was a freshly spalled surface which indicated some distress due to bending (Figure 51). This could be the result of movement of the pile due to lateral spreading of the fill towards the water, combined with the effect of the large inertia loads from the heavy building.

The 4-storey administration building adjoining the control tower looked to be in good shape at the front. The main block is a three storey reinforced concrete frame supported on piles and had suffered repairable damage to masonry infill. However, the single storey annex at the rear was apparently on pad foundations and had suffered from settlement and spreading of the fill and required demolition (Figure 52). Similarly, the ground slab of the main building had settled and required replacement (Figure 53).

In the workshops some masonry partition walls had collapsed and settlement had occurred of the fill under the ground slabs (Figure 54).

Collapse had also occurred of a cantilever overhang of a building under construction in the area behind the administration buildings (Figure 55).



**Figure 48** Kandla Port: 2-storey building parallel to the waterfront had lost its lower storey due to soft-storey collapse of the circular columns



**Figure 49** Kandla Port, 2-storey building: laps in maximum moment zone and virtually no confining reinforcement – a fatal combination





**Figure 50 Kandla Port: 6-storey control tower building was inclined by a few degrees towards the water.**



**Figure 51 Kandla Port control tower: freshly spalled surface of corner pile indicated bending distress due to inertia loads and lateral spreading of underlying soil.**



**Figure 52 Kandla Port: single storey annex at the rear of 4-storey administration building had shallow foundations and had suffered from settlement and lateral spread of founding soil**



**Figure 53 Kandla Port: Similarly, the ground slab of the main building had settled and required replacement**





**Figure 54 Kandla Port: some masonry partition walls had collapsed in the workshops**



**Figure 55 Kandla Port: collapse of cantilever overhang of building under construction**

c) Reports of Damage to Other Parts of Kandla Port

*Access Road*

It was reported immediately after the earthquake that the road into the port was badly damaged by settlement and lateral spreading, with cracks appearing near the north gate to the port.

#### *Oil Jetties*

It was not possible for the team to gain access to the oil jetties. The Gulf Agency Company (GAC) based in Cochin put out regular internet reports (Gulf Agency, 2001) on the effect of the earthquake on the port of Kandla and other ports in the region. Immediately after the earthquake GAC's staff reported that at the oil terminal, pipelines to jetties No.4 and 5 had been ruptured, that the pipeline to jetty No.3 had been damaged, and that jetties 1 and 2 had also been damaged. A report on 2 February confirmed that oil jetty No.1 had been badly damaged and might have to be demolished, and that jetty No.3 had suffered some damage, but oil jetties 2, 4 and 5 appeared to be fine. However, although jetties 4 and 5 were unscathed, it was reported that the shore lines at these two jetties were severely damaged. Similarly, although jetty 2 was unaffected, an onshore fire control tower was reported as listing towards the jetty, and the berth was not used until 9 February. The jetty was closed for 4-5 days from 10 May for repairs to the fire control tower.

On 18 April Kandla Port decided to resume berthing of light displacement vessels (tankers up to 10,000 MT displacement) at oil jetty No.1. As strengthening work progressed, the displacement limit was increased to 21,000MT on 11 May.

#### *Warehouses*

Several warehouses (go-downs) exhibited short column damage and damage resulting from heavy concrete roof trusses.

#### *Cranes*

There was no apparent damage to the dockside cranes, all of which were operating normally. The Chairman of the Port Trust reported that all were working within a few hours of the earthquake, as soon as power was restored.

### **4.3.3 Navlakhi Port**

Navlakhi Port handled about 400,000 tonnes in 1996/97, the last year for which figures are available. It mainly imports coal and exports salt.

The port is located at the top of the Gulf of Kachchh, some 25km due east from Kandla Port but on the south side of the gulf whereas Kandla is on the north side.

The failures which occurred were geotechnical in origin and are described in Section 3.6.2. They are therefore not discussed further here.

## **4.4 Industrial Facilities:**

### **4.4.1 Factories**

Much of the industry in Gujarat is small scale, and there are relatively few factory units. Those which were observed were generally of masonry construction with steel truss roof, or concrete pitched portal frames with masonry infills. A significant proportion suffered damage, as illustrated in Figures 56 to 59.

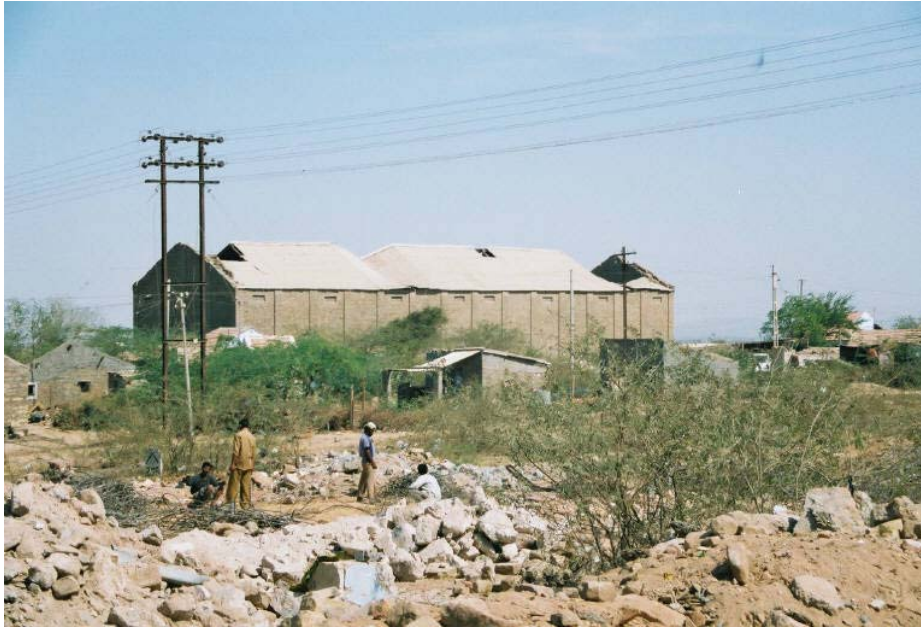




**Figure 56 Collapse of Masonry Walled Bulk Material Warehouse, near Kandla**



**Figure 57 Industrial Unit Near Kandla**



**Figure 58 Factory Unit Near Bachau**



**Figure 59 Factory Unit Near Bachau**

#### **4.4.2 Petrochemical and Chemical Plants**

The major chemical plants identified in the area are:

- Essar Petroleum: refinery at Jamnagar
- Hindustan Lever Ltd (HLL): personal care products plant at Kandla.
- Indian Farmers Fertiliser Cooperative (IFFCO): phosphatic fertiliser plant at Kandla
- Indian Oil Corporation: refineries at Koyali and Baroda and various pipelines across Gujarat

- Reliance Industries: petrochemical plant at Hazira and refinery at Jamnagar.
- Indian Petrochemical Corporation Ltd: refinery at Baroda JR Enterprises
- Krishak Bharati Cooperative Ltd (KRIBHCO): fertiliser plant at Hazira, near Surat
- Meridia Chemicals: plant at Surendranagar
- Nirma Ltd: soda ash plant near Bhavnagar; soap and detergent plants at Mehsana, Ahmedabad and Rajkot; N-paraffin plant at Baroda.
- Reliance Industries Ltd: refinery at Jamnagar and petrochemical complex at Hazira.
- Tata Chemicals: Soda Ash plant at Mithapur

In addition, the Gas Authority of India Ltd operates a gas pipeline from Hazira to Uttar Pradesh, with terminals at Bijaipur and Hazira. They report no damage to the pipeline or termini.

The Economic Times (Economic Times, 2001) reports that a number of these companies have filed insurance claims, but most report that they are working near normally, and the damage would generally appear to be of a minor nature. The state government notes the following plants as requiring significant repair:

- IFFCO, Ghandidham, R500 million (£7.4 million)
- Meridia Chemicals, Surendranagar R150 million (£2.2 million)
- Tata Chemicals, Mithapur R100 million (£1.5 million)

Essar Petroleum reported no damage to their facility at Jamnagar, with production recommenced by 27 January.

The Kandla plant of Hindustan Lever Ltd is reported as suffering damage, but the team did not have the opportunity to inspect the plant or ascertain the degree of damage.

The IFFCO plant adjacent to Kandla port was constructed in 1974 to produce phosphatic fertilisers, and was extended in 1981 and 1999. Current annual production is around 38 trillion tonnes. The plant suffered some damage, including collapse of the bag conveyor and of the bag loading area. The potash storage building also collapsed, and damage occurred to masonry infill panels around the plant. The ammonia pipeline from Kandla to the plant was undamaged, as were the ammonia storage tanks. IFFCO report that the plant was returned to operation on 26 March.

Indian Oil Corporation operates 7 of the country's 17 refineries and has about 40% of the national refining capacity. Its closest refinery to the epicentre is at Koyali, near Ahmedabad, which was constructed in 1965 and currently has a capacity of 0.27 million barrels per day. No damage was reported at this plant. Petroleum products from the Reliance refinery at Jamnagar are piped to Kandla, with the bulk being piped on to Bhatinda, some 1400km distant. IOC reported some damage to the pipeline, which has a capacity of 1.5 million tons pa, but it was repaired within a few days of the earthquake, prior to the EEFIT visit. There was also some damage in the IOC tank farm at Kandla, as noted earlier. Some damage to its Baroda plant was also reported.

Indian Petrochemical Corporation Ltd reports no damage to its plants.

JR Enterprises suffered damage to a 2000m<sup>3</sup> tank of acrylonitrile (ACN), resulting in leakage of the contents from a circumferential rupture between the tank wall and base. The same plant also suffered leakage from a paraffin tank.

KRIBHCO report that they are operating normally.

It has not been possible to establish the extent of damage at Meridia Chemicals.

Nirma Ltd. reported minor damage to its soda ash plant at Bhavnagar, but restarted operations within 24 hours. It reports no damage to its other facilities.

Reliance Industries reported no significant damage at their plants and restarted production on 28 January.

It was not possible to determine the extent of damage at Tata Chemicals.

#### **4.4.3 Other Industrial Plants**

Other major industrial plants in the area include:

- Birla: copper manufacture, cement plant and fibre plant
- Essar: steel plant at Hazira
- Gujarat Ambuja Cement Ltd (GACL): three cement plants at Kodinar, with sea terminals at Muldwarka and Surat
- General Motors: auto plant near Baroda
- HLL: salt plants at Ghandidham
- Larsen and Toubro (L&T): cement plants at Jafrabad, Magdalla and Pipavav
- Paharpur: Cooling Towers
- PepsiCo: bottling plants at Ahmedabad and Rajkot
- Saurashtra Cement: cement plant at Ranavav
- Sikka: cement plant at Sikka
- Tata Chemicals: cement plant at Mithapur

The state government notes the following plants as requiring significant repair:

- Salt refineries: R200 million (£3 million)
- Digvijay Cement Ltd: R400 million (£5.9 million)
- Tata Chemicals: Soda Ash plant and cement works at Mithapur

Birla are reported as having filed an insurance claim (Economic Times, 2001), but state that their plants are working normally; it therefore appears that damage is light. The same applies to Essar. GACL and General Motors report no damage.

HLL's salt plants have suffered 'extensive' damage, but the team were not able to inspect them during the mission.

L&T report no damage to their plants, and no information could be obtained on Pharpur, Pepsico, Suradshatra Cement or Tata Chemicals.

However, the team were able to inspect significant damage at the Sikka cement works (Figures 60 and 61).





**Figures 60 & 61 Sikka Cement Works**

## **4.5 Lifelines**

### **4.5.1 Electrical Power**

Electricity in Gujarat is distributed by the Gujarat Electricity Board (GEB) from some 20 GEB and 7 private generating units. Total capacity is 4166MW, provided from power stations at Wanakbori, Gandhinagar, Dhuvaran, Utran, Panandhro, Sikka, and Ahmedabad (the last being operated by the Ahmedabad Electricity Company). The domestic electrical system is a 220v 60hz supply. Main transmission is at 220kV, which is stepped down to 132kV, then 66kV. Further substations step it down to 11kV for local distribution.

The power station at Panandhro was reported as having suffered cracking in the walls, as did the station at Sikka. This may simply be cracking of masonry infill, but it was not possible to arrange an inspection. Dhuvaran also suffered some cracking as well as a small fire. It appears that the damage was superficial, although no details are available

The 220kV substations at Anjar, Nakhatrana and Nani Kakhar were damaged, as were the 132kV substations at Samkhyali and Bhuj and a further 45 66kV substations.

There was also significant damage to 11kV and 415V feeders. The 220KV transmission line from Panandhro to Anjar was damaged, and the Panandhro station was taken off line temporarily. Power supplies to Bhuj were restored on 28 January via the 132kV line from Lalpur.

The team visited the Anjar substation, which consists of 10 No transformers of 40 to 100MVA. Three of these jumped their rails transversely (east-west, the rails being aligned north-south), breaking the oil cooling pipes. 6 No circuit breakers failed due to broken porcelain insulators. The regional engineer reported that power through Anjar was restored three days after the earthquake, although it was noted during the visit that the transformers were still operating on temporary supports. (Figure 62 and 63).

Numerous URM substation building were damaged, and operations at Anjar were being conducted from tents at the time of the visit.

GEB estimate that repairs will cost £27 million, and plan a further £67 million expenditure on upgrade work.



**Figure 62 Transformer on Temporary Supports at Anjar Substation**



**Figure 63 Repaired Circuit Breaker at Anjar Substation**

#### **4.5.2 Water Supply**

Water supply was disrupted in 18 towns and 1340 villages in the area. Damage was minor in Banaskatha and Surendranagar, but severe in Kachchh, Jamnagar and Rajkot. Water supply in Kachchh is largely groundwater based, with only the Tappar and Shivilakha dams providing surface water supply. The affected areas in Jamnagar and Rajkot rely mainly on surface water supply from Machhu II dam, with a few ground water schemes.

Tappar and Shivilakha dams suffered damage, as discussed in Chapter 3, as did water treatment plants at these sites and at Anjar. Approximately 350 tube wells and 1500km of distribution pipework are also severely damaged.

Many water towers were undamaged, but some suffered cracking in Anjar and Ghandidham and one collapsed.

#### **4.5.3 Sewerage**

In the five districts most affected by the earthquake, water borne sewerage systems existed only in Bhuj and Anjar (World Bank, [www.worldbank.org/gujarat](http://www.worldbank.org/gujarat)). It is known that these have been damaged, but no details are available.

#### **4.5.4 Telecommunications**

Land lines in Gujarat are operated by BSNL, an agency of GOI, which has about 2,500 exchanges. Mobile services are provided by AT&T and Celforce.

Approximately 180 telephone exchanges were damaged, although no details are available. The cellular systems also suffered damage to some aerial towers. The damage has been estimated by BSNL at approximately £7.5 million.

### **Acknowledgements**

The EEFIT Team would like to thank Mr. A.K. Jyoti, Chairman of Kandla Port Trust for making time in his busy schedule to provide an excellent briefing for the team.

The EEFIT Team would like to thank Dr Nayak, Director of the design-and-build contractor Gammon India Ltd for the new Surajbari Bridge, and Mahesh Tandon, Managing Director of Tandon Consultants PVT Ltd who were proof consultants on behalf of the Government of Gujarat for the helpful information provided on the Surajbari Bridges. Mahesh Tandon also kindly supplied an early copy of his paper with Rajiv Goel (Goel R. and Tandon M. C., 2001).

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# Chapter 5 Engineered Building Structures

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## 5.1 Introduction

This chapter of the report deals with the structural performance of engineered multi-storey framed buildings (i.e. rubble construction is not covered herein). The engineered buildings that existed were generally limited to reinforced concrete structures of three or more storeys. The buildings were generally residential, although some were commercial and some mixed use involving residential accommodation above ground floor commercial premises. Failure of the buildings was generally a result of the building form or the detailing of the structural components. Sections covering soft stories, building configuration and short columns, outline the failures relating to building form. Detailing and design, materials, and workmanship sub-sections cover the detailing aspects. The chapter concludes with repair options and general recommendations to reduce structural failures should the area be subjected to further earthquakes in the future.

Ahmedabad, a major city located 220 km from the epicentre with a population of 4 million, had approximately 60,000 buildings that were registered with the local authorities and these varied from three to fourteen storeys. Most buildings were relatively modern and the majority of the failed structures were less than 10 years old. Local engineers advised that this timeframe corresponds with an upturn in the Bombay stock market with more investment in buildings causing a surge in the building industry. Almost all of the buildings sustained some damage and about 60 buildings had collapsed or needed to be demolished.

The buildings in Bhuj were generally three to six storeys high. The overall layout and spacing of the buildings appeared to be less controlled than some of the more modern towns and cities visited. As Bhuj was the closest city to the epicentre with a significant number of engineered buildings, the level of damage was high and there were a number of dramatic collapses.

Gandhidham, located on the North coast of the Gulf of Kachchh, provides accommodation to the work force of the nearby Kandla Port. The structures consisted mainly of residential buildings but also had buildings with commercial on lower floors and residential above.

In Anjar, the buildings were used for residential purposes and were generally of low rise and non-engineered construction and the number of engineered structures was relatively low compared to the other towns. The low-rise buildings were constructed from masonry or rubble; their performance is discussed elsewhere in this report.

## 5.2 Modes of failure

Reinforced concrete framed residential buildings are prevalent throughout Gujarat particularly in the more modern towns and cities, typically four to ten stories in height. Almost all of the modern engineered buildings are constructed of reinforced concrete. Local engineers advised that, in general, buildings were designed for gravity loads only. Poor construction and detailing, combined with soft storey, improper configuration and short column effects, brought about widespread collapse and/or structural damage. It was evident that there were modes of failure common to the different areas that were investigated.

### 5.3 Soft/weak stories

A soft or weak storey is created when the lateral stiffness and/or strength of a storey is markedly more flexible than the floors above and below. This often occurs at the ground floor when it is left open for parking, a shop front, or other reasons. Most of the deformation demand from the seismic event is concentrated at this level and results in large rotation demand in columns that have not been designed for ductility (see Section 5.5). Soft/weak storey collapses have been seen in many past earthquakes. A typical soft storey collapse is shown in Figure 1.



**Figure 1 Typical soft storey collapse in Bhuj**

Soft/weak storey collapses were observed in all the towns and cities visited throughout Gujarat. Figure 2 illustrates the remains of a building (foreground) in Ahmedabad that collapsed due to the presence of an open first storey for parking. In this case, after the first storey failed, the added force of impact at each floor caused subsequent stories to collapse. The almost identical building in the background, although still standing, suffered severe damage. The reason for one building to collapse and the other to remain standing may lie in the fact that the standing building contained more infill walls that are illustrated in figure.2. This wall, although damaged, was able to contribute to the lateral force resistance of the structure, and the combined added strength and stiffness from several of these walls at the ground level most likely contributed to this building's survival.

A number of buildings remained standing yet suffered varying degrees of damage due to concentrated lateral loading at less stiff floors. Although not soft storey failures, the buildings did show how elements behaved at levels that could have been close to ultimate capacity. Often it was the presence of some infill walls that provided an alternative load path or restraint to the structural members thus reducing the transverse loads and displacements occurring at the critical column members.



**Figure 2 Soft storey failure in Ahmedabad**

Figure 3 illustrates the remaining half of a 10-storey apartment building. The surface ground motion in Ahmedabad affected several relatively tall structures, like this one. Ground to first floor column failure due to the soft/weak storey effects combined with lack of continuous reinforcement between attached buildings, caused this structure to collapse. Approximately 70 people lost their lives in this structure; most bodies were found in the collapsed stairwell indicating that the building withstood the initial effects of the earthquake.



**Figure 3 Soft storey failure in Ahmedabad**



Figure 4 illustrates another soft storey building collapse in Gandhidham. The lack of continuity within the structure is evident.



**Figure 4 Soft storey failure in Gandhidham**

Figure 5 illustrates a collapsed basement and first storey levels of an apartment complex supported by open fronted shops. The columns, after shearing off the beam-column connection, punched through the first level slab that had translated under the lateral loading. Throughout Gujarat, poor detailing and materials provided little capacity for the columns to resist the applied shear and moments. This was particularly the case at the top and bottom of the columns where little or no continuous reinforcement was present to provide anchorage and to allow some ductility.



**Figure 5 Soft storey failure in Gandhidham**

In Figure 6 a partial soft/weak storey collapse in Anjar is illustrated. The front of the building collapsed while the back remained standing highlighting poor continuity of reinforcement. The left triangular front wall used to be at the same height as that of the structure to the right.



**Figure 6 Partial collapse of an apartment building in Anjar**

Figure 7 illustrates the results of a soft/weak storey collapse. The failed column can be seen on the photo on the right.

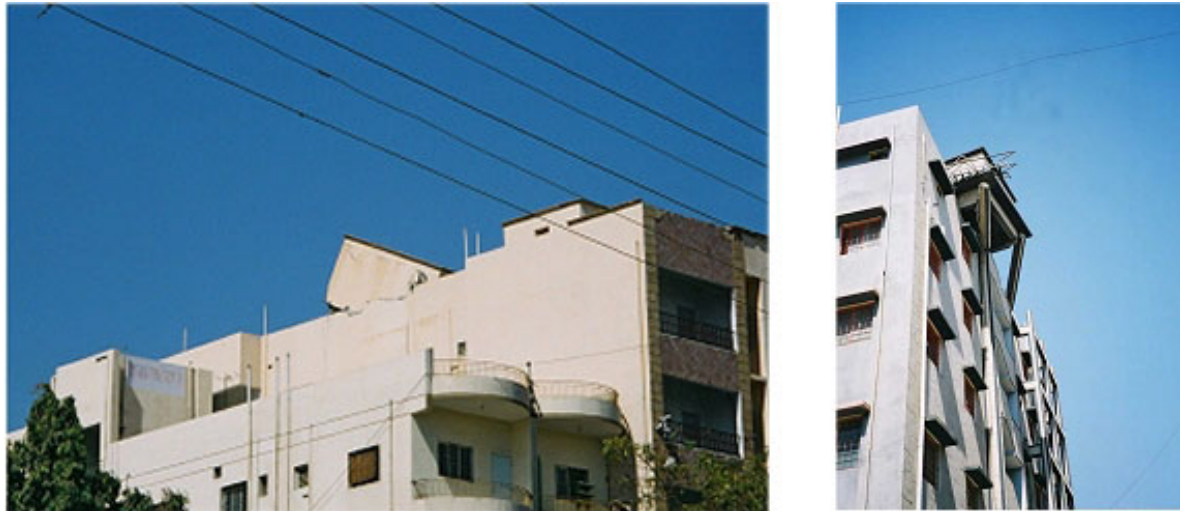


**Figure 7 Soft/weak storey collapse in Bhuj**

In many standing buildings, infill panels constructed of masonry were provided within the RC frame, acting as shear walls to stiffen the building and thus preventing soft storey collapse. It is considered that in the design of the structures the strength of the infill panels was not taken in to account; the walls were provided purely as architectural features. It was evident that the buildings experienced significant sway motions shown by perimeter cracking of the covering plaster along the interface between the masonry infill panels and the main structural frame, and it is considered that, without these walls, the death toll and number of building collapses would have been much higher.

## **5.4 Configuration**

The presence of water tanks supported on top of structures had an important effect on the behaviour of the multi-storey structures. Changing the dynamics of the structure, the poorly supported tanks contributed to several collapsed buildings. In some cases, the water tank simply fell through the building. As Figure 8 illustrates, the extra mass acted as a cantilever appendage, significantly increasing the potential for damage to the structure.



**Figure 8 Water tank failures in Bhuj**

One particular apartment building, Mansi Tower in Ahmedabad (Figure 9), illustrates the result of the effect of extra mass changing the dynamic properties of the structure. On top of the structure on the left (collapsed), a swimming pool was added without authorisation from the planning department. This addition (not included in the original design plans) elongated the natural period of the structure causing this portion of the structure to resonate with the long period ground motion due to the soft soils and collapse. The structure's right half (without the swimming pool, and therefore maintaining a higher natural frequency) survived. The additional load of the swimming pool will also have contributed to potentially overloading the structure under normal gravity loads. Note the lack of continuity between the different parts of the same building. In this case the stairs spanned between the two building blocks but could not provide any structural connectivity. Under seismic loading the two halves of the complex behaved individually with clearly different consequences.





**Figure 9 Mansi Tower (Ahmedabad)**

In Figure 10, the inclusion of extra mass due to the presence of a water tower on the top floor caused this apartment building in Bhuj to collapse. What is shown is one half of the structure on its side. The rubble of the collapsed half of the structure is within the basement and underneath the remaining half that can be seen. The additional mass of the water tank, in combination with a partial soft storey collapse over the failed half, caused the structure to rotate onto itself. The centre of rotation for the collapse was at a point at ground floor level mid-way along the length of the building possibly around a stronger entrance structure. The bottom of this building is shown in Figure 11. Note the ease in which the columns became detached from the foundation. The column reinforcement bars simply pulled out of the foundation concrete, when subjected to tensile forces, indicating a lack of reinforcement to concrete bond strength.



**Figure 10 Overturned building in Bhuj**



**Figure 11 Overturned building in Bhuj**





**Figure 12 Combination of collapses in Gandhidham**

In Ahmedabad, the first floor of many buildings was cantilevered out 1-2m beyond the ground floor. It is suggested that this is because building regulations governing building separations and curtilages apply only at ground level, and developers maximise the useable area by cantilevering the higher level floors. In many cases, the cantilevered beams were substantially damaged in shear during the earthquake.

Building geometry contributed to a number of failures. The lack of symmetry about any orthogonal axes could generate torsional forces within the structure. This would often lead to a catastrophic collapse as displacement demands exceed the ultimate capacity of the structural elements, particularly along the perimeter of the structure. The types of configuration that contributed to the collapses were geometric and non-symmetrical loading particularly due to the position of water tanks on the roof of buildings.

In many cases, buildings collapsed due to a combination of many effects. Figure 12 illustrates an apartment building in Gandhidham that possibly collapsed due to a combination of poor configuration (torsion), unnecessary mass (the inclusion of a water tower), and a soft storey. In torsion, the building shook free of its lateral force resisting system, the elevator core. The part of the building that was attached to the core subsequently failed due to a soft storey (the main building intact used to be the same height as the core). The water tank, sitting on the collapsed left side of the structure is also illustrated.

Through the investigation period it was observed that buildings of similar type and location performed differently which may be attributable to the overall configuration. A relatively small change in the configuration could lead to the progressive collapse of the building due to the over stressing of a critical element under the loading condition during the seismic event. A potential cause of this could be, for example, the unauthorised changes to the buildings during or after construction that could alter the load path through the structure. It was advised that due to a lack of building inspectors, construction sites could not be visited as regularly as they would desire. Some unscrupulous builders would take advantage of this by reducing the quantities of materials, such as reinforcing bars once the inspector had visited without fear of detection.

Although some secondary items were considered as non-structural elements in the initial design the fact they add stiffness and strength to the structure means that they could attract some load. An example of this is thin columns positioned at the outer corners of upper storey balconies. The columns only had minimum vertical reinforcement steel and, as the short straight bars had pulled out of the balcony slab it was evident that this was not correctly anchored in to the balcony.

## 5.5 Detailing and Design

At most of the collapsed sites of reinforced concrete structures it was apparent that there was inadequate strength and confinement in the sections at critical points. By inspection of the collapsed buildings, especially where the concrete had separated from the reinforcing steel, it could be seen that the detailing had not followed standard practices for gravity loading. For example, beams were seen which were detailed for sagging at midspan and hogging at the column connections with only minimal reinforcement in the opposite faces. Under seismic loading it is likely that the joints in particular will experience a reversal of load, placing the opposite face of the beam in tension. The main types of poor detailing that caused failures included:

- lack of continuity between the various structural elements
- insufficient shear reinforcement
- incorrect position and spacing of laps between reinforcing bars (poor confinement)
- lack of suitable anchorage in to the concrete
- inadequate reinforcement for seismic loading (generally not considered in the design)

Local engineers advised that the design of reinforced concrete framed buildings was given little priority in the development of a building. For this reason it followed that buildings would be detailed rather than designed. It appeared that columns contained standard vertical bars and links. This could have contributed to a number of failures as elements were overloaded.

The most common location for failures was at the top of the ground floor columns where buildings had undergone a partial soft storey failure. This part of the column had experienced excessive rotation demands.

In some instances the joint was subjected to high vertical loads during the earthquake. This caused over-stressing of the concrete section leading to bursting of the concrete. The usual spacing of the reinforcing shear links (often as small as 6mm diameter mild steel hoops) was approximately 200mm. This spacing was too large to confine the concrete allowing it to burst sideways in a cone emanating from adjacent shear hoops. This could also be caused by a misalignment of the top link during construction, leaving a region below the framing beam without shear reinforcement. Where the column concrete had burst away it left the vertical steel to resist the vertical loading, leading to a buckling of the vertical bars. The final position of the top of the column relative to its joint varied depending on the cycling of the loading. Transvers displacements of approximately 50mm were seen at the Shikir Tower in Ahmedabad. Figure 19 shows this being repaired.

A number of failed multi-storey residential complexes in Ahmedabad consisted of connected blocks (usually four). At two sites, the Shakir and Mansi Towers, one of the blocks had collapsed leaving the remaining parts standing. In both cases a vertical failure plane was exposed and it appeared that little or no reinforcement protruded from the intact parts. This suggests that the blocks were designed and/or detailed individually. As discussed within the configuration section this would cause the blocks to respond in a discrepant manner. The lack of continuity reinforcement, rather than giving a load path into the other parts of the building, would permit the failure to continue until it became catastrophic.

An example of a localised lack of continuity could be seen at a building in Bhuj, where the slab and the edge beams had separated. The edge detail of the slab had the transverse bars with hooks extending to the outer side of the edge beam but it was located below the top level of beam reinforcement steel. This allowed the slab and beam to respond independently of each other. This separation would also reduce

the effective section that would have been claimed in the design or analysis, where an L beam would have been considered.

Within Kandla Port, a two-storey office building had undergone a soft storey collapse in which the circular ground floor columns had rotated through 90 degrees at their bases (Figure 13). The reinforcement had been lapped just above ground level giving a reduced concrete area in the critical section. The concentration of laps at the critical section allowed the cover concrete to be put into radial tension. Because the lapped bars would tend to separate, this would soon lead to the concrete bursting off the column when subjected to a high flexural load. Ideally the detail should have had staggered laps away from the highly stressed areas and closer spaced links. This highlights that a lack of ductile detailing in the critical columns can contribute to a soft storey collapse, and is the reason why, on seismically qualified structures, couplers are often used on bar diameters of 32mm and above. Figure 13 shows the 350mm diameter column containing T16 bars, with links at approximately 300mm centres. It can be seen that the concrete has broken off very cleanly from the reinforcement.



**Figure 13 Poor detailing at Kandla Port**

Numerous examples of the provision of insufficient lap lengths were seen. This, combined with the weak bond strengths between the reinforcement and the concrete, contributes to a brittle mode of failure. Examples of this can be seen at a three storey building in Ahmedabad (Figure 14) where the imprint of the lapped bar (top left) remains in the column section that had been cut from a failed column. A more dramatic example can be seen in the building shown in Figure 11 in Bhuj that had rotated through 90 degrees and the ground floor columns had been put in to pure tension. The force had pulled the reinforcement out of the foundations. In Figure 12, the reinforcement can be seen protruding approximately 3-4 metres out of the column base. The widespread existence of poor concrete to steel bond was clear by the industry that had built up at the rubble dump sites where people used sledge hammers to break away the structural concrete to allow the steel to be sold for recycling. In most cases the steel was almost clean that would not be expected with high quality concrete.



**Figure 14 Burst concrete cover**

Figure 15 shows a reinforced concrete framed building in Anjar that was partially complete when the earthquake struck. At first inspection it appeared that the building had acted well by standing up. In life saving terms this represents success because the residents could escape. However, on closer inspection of the columns it can be seen that hinges have formed at the top and bottom. This indicates that the columns behaved in a ductile manner (i.e. the steel yielded before the concrete crushed). However, if the earthquake loading had been slightly higher the columns may have failed leading to a catastrophic collapse. Ideally the hinges would form in the horizontal beams rather than the columns to ensure that the energy is absorbed through plastic deflection in the beams, and not the principle gravity support members in the structure (i.e. the columns).



**Figure 15 New building in Anjar**

## **5.6 Materials and Workmanship**

It appeared that the available concrete was of poor quality due to the use of low quality cement, and the lack of quality control during batching. In general, it appeared that the strength used in design was  $f_{cu}$  of  $25\text{N/mm}^2$ . Although the specifications usually called for machine mixed concrete, it was advised that in most cases the concrete was mixed by hand on the site and also placed by hand methods. This resulted in the overuse of the cheaper larger aggregates rather than sand and a high water content to



ease placement. Combined with the high temperatures the concrete was under strength. As the concrete was hand placed, the level of compaction was minimal; for columns it was generally self compacted therefore the top region would be less compacted. This can be seen in the soft storey collapses where this region of the column was heavily loaded in a seismic event.

The reinforcement steel that was collected from the failed buildings appeared to be of a more uniform quality with both mild steel and ribbed high tensile steel encountered in most areas.

## **5.7 Short Column Failures**

A short column failure is caused by its relatively high stiffness in comparison to other columns at that floor level. The transverse forces generated at a floor level are distributed in proportion to the member stiffness, therefore a short column will attract a greater proportion of the load and, when compared to a more slender member, will have less ability to withstand the deflections that will occur over their height.

Classic short column failures could be found throughout the region. Figure 16 and 17 illustrate examples of such failures in Ahmedabad and Kandla Port, respectively. Figure 16 shows diagonal cracking in the façade of a multi-storey building. This shows that the areas between the windows are behaving as short stocky members when compared to the continuous panels above and below the windows. The cycling shear loads across these less stiff zones has resulted in tension cracks forming across both diagonals of each panel.



**Figure 16 Diagonal cracking in Ahmedabad**



**Figure 17 Heavily cracked building at Kandla Port**

Figure 18 illustrates failure of a column due to its length being shortened, thereby increasing its shear demand. This was caused by the inclusion of a non-structural wall but had the effect of creating a fixity point. The stiffness of the column would be higher than the others, causing it to attract a greater proportion of the load. This would far exceed the capacity of the column and the result is clear to see, although other types of failure are likely to have occurred in this building.



**Figure 18 Building Collapse in Bhuj**

## **5.8 Foundations**

Very little evidence of foundation failure related to the building structures was found in Gujarat. Ground conditions were generally of good quality with dense sediments becoming very dense with a

small increase in depth. The quality of the foundation would be far greater and less susceptible to a brittle mode of failure than the minimum capacity of the more critical elements in the superstructure. At Mansi tower a drilling rig had been employed by a Government agency to undertake a ground investigation at the site where the 14 storey building had collapsed with the death of between 35-75 residents. Discussions with the technicians indicated that the ground conditions were good. Reference should be made to section 3 where geotechnical aspects are detailed.

## **5.9 Mechanisms and combination of modes of failure**

The individual failure modes of the buildings have been described above but due to the complex nature of the event and the non-uniformity of the buildings, the failures were more often a combination of at least two of the modes. The most likely combination is lack of configuration, and non-uniform distribution of stiffness around a building structure.

## **5.10 Repairs and short term corrective action**

Where multi-storey buildings had remained standing, but had sustained some damage to either the structural frame or to the infill panels, the owners of the buildings had instigated some corrective action by providing temporary props to prevent further collapse from aftershocks. As a short term measure this method may be acceptable to minimise further damage but would likely be unacceptable for habitation purposes. The longer-term measures included the jacketing of columns and the repairs of more badly damaged columns. In Ahmedabad, sound official advice was given for the remedial measures. However, the repair work undertaken did not generally follow the recommendations.

Where buildings had suffered damage but had remained standing there was a need to undertake repair work both to prevent further collapse from aftershocks and to instil some confidence to the residents. Two types of repair to RC framed buildings were witnessed; the provision of additional masonry infill panels and 'jacketing' of ground floor columns.

The buildings that needed repairing had experienced a soft storey action as described earlier. The ground floor columns had adequate ductility to prevent collapse but the upper joint had gone well beyond its point of plastic rotation. Evidence of plastic hinges forming at the tops of the column, combined with some shearing action, resulted in misalignments of around 50mm. Often the reinforcement was exposed where the cover reinforcement had burst away under the axial load.

The column repairs that were investigated were on the ground floors of ten to fourteen storey buildings in Ahmedabad and they appeared to be inadequate to handle the load transfer to the foundations. The repair undertaken at the Shikar Tower of ten-storey height consisted of the following steps and is shown in Figure 19.

1. Vertical angle sections placed at the corners of the original column tack welded to flat steel bars that framed the column at approximately 200mm centres.
2. Vertical reinforcement consisting of T20 bars was placed around the perimeter surrounded by R6 links at 200mm centres. The vertical steel was placed with its lower end against the ground slab without dowelling into the foundation. The top of the bar was allowed to run past the top of the column and terminated alongside the horizontal framing beam.
3. Steel shuttering was placed around the column giving approximately a 50mm surround of concrete. This was placed by hand through the gap at the top of the shutter without mechanical compaction.



**Figure 19 Column repair in Ahmedabad**

The above repairs were ineffective for the following reasons.

1. The plaster surrounding the original column was not removed therefore there would be insufficient bond between the two types of structural concrete. This would prevent any composite action between the original concrete and the repair work.
2. The vertical steel would not form part of the vertical or moment load path to the foundations from the top of the column, due to complete lack of continuity. To be effective, the repair should be tied in to the foundation and made integral with the beam at the top.
3. The shear links combined with the horizontal flat bars and angles would not offer any significant increase to the bursting resistance of the columns because the quantity of reinforcement is inadequate and the framing was only made by minimal tack welding.
4. Due to the lack of compaction it will be unlikely that the top part of the concrete will achieve sufficient strength to be considered structural. Also the resulting detail at the top of the column would not give a load path to any shearing forces that could be experienced. It can be seen from the figures that it was this part of the column that had undergone the greatest distress during the earthquake.

The Government of Gujarat published sound repair advice in newspapers, giving repair guidelines and recommending the use of a structural engineer. The advice detailed how to undertake repairs covering the use of temporary supports, jacketing columns and repairing heavily damaged columns. The



government advice on repairs would achieve a column greater or equal to the original requirements if undertaken correctly. From the evidence, the repairs failed to meet the recommendations set out by the government, and it was considered that the repairs we witnessed were being undertaken on an ad-hoc basis at the guidance of the building owner without the control of an approved structural engineer. It should be noted that soon after the mission the repair works were stopped under a Government edict.

Where remedial measures were undertaken to provide additional infill panels at the ground floor level to prevent any further displacements, strip footings would generally be provided allowing the in plane forces to be resisted by the masonry. The aim of the infill walls was to provide a more uniform stiffness over the height of the building thus reducing the risk of soft storey collapse. The mechanisms involved with the provision of the shear wall are less complex for the layman or building owner to comprehend.

## **5.11 Administration and Procedural Causes**

Authorisation of building plans were done at a very early stage using architectural plans that were subject to change. Floor plans were likely to change due to alterations in demand of floor space, changing the load path and leading to configuration problems - see above.

It was apparent that site supervision was given a low priority for building projects, allowing some unscrupulous builders and developers to take advantage of the situation, and making it easy for unacceptable practices to continue. In most cases the specifications called for machine mixed concrete but hand mixing methods were usually adopted, and there were reports that, to save costs, the column sizes would be reduced, and reinforcement removed, as construction progressed. These problems could be reduced if more Government employed inspectors were given powers to undertake more detailed inspections. However, this would have to be reflected in the cost of the buildings.

## **5.12 Long term**

At the time of writing, there are design standards in India covering seismic design but these are advisory only. Consideration should be given by the authorities in the region to making them compulsory. Any buildings or structures that are to be constructed in the Gujarat region should include some allowance for seismic loading. Depending upon the importance and safety function this could be relatively simple detailing changes that would improve the ductile performance, giving occupants the time and routes to escape to safety. Alternatively if there is a higher demand on the structure then the design and analysis should follow more detailed approaches. Seismic requirements of structures in Gujarat, follow Indian standards with up to 8% of mass applied as a horizontal load. This would be adequate in many cases providing the load path is correctly assessed. The most cost effective method for use on buildings where the financial return is important would be to adopt better detailing methods. An important factor would be to improve the site supervision during construction to increase the likelihood of the design being implemented correctly. This could be achieved with more building control, but would increase the cost of buildings.

## **5.13 Conclusion**

It is concluded that the engineered buildings of Gujarat performed as expected for an earthquake of the magnitude experienced on 26 January 2001. The most prevalent mode of failure was soft/weak storey collapse, but this was compounded by the use of poor quality concrete and reinforcement detailing that was not appropriate for seismic loading. In other cases of catastrophic failure, the structural designs had only considered normal gravity loading, and the configuration of the structures under earthquake loading had resulted in some critical elements becoming overloaded.

It was apparent that the repairs that were being undertaken were not to a satisfactory standard but there were guidelines available that would be suitable if they were implemented correctly.

To avoid collapses it would be recommended that reinforced concrete framed buildings are constructed with consideration given to providing a level of ductility. This can be achieved by having continuous

reinforcement in all faces of beams and columns, and adequate anchorage and lap lengths. A key requirement for ductility is having adequate strength concrete. This can be achieved by using the appropriate materials and necessary supervision on site, and the application of the existing Indian seismic design standard IS1893.









**Figure 1 Typical rubble stone Type A dwelling in a Dharmarkan, West of Bachau**



**Figure 2 Typical coursed stone with timber floor Type B dwelling where poor tying in of roof has caused upper wall overturning.**



**Figure 3 Typical coursed stone with concrete diaphragms Type C1 building suffering moderate damage in the masonry piers.**



**Figure 4 Typical RC framed Type C2 building in Ahmedabad where ground floor columns have failed in a weak storey mechanism. Note the absence of columns along the perimeter of the building.**

## **6.2 Relative performance of masonry buildings**

### **6.2.1 Survey of masonry buildings**

As part of the ongoing work to relate building damage to earthquake ground motion a number of previous EEFIT Missions have carried out building-by-building ground surveys in the proximity of recording stations. The Gujarat earthquake was only recorded within the whole region by a single set of instruments in Ahmedabad, 225 kms from the epicentre. Here, even though damage was reported, this was not widespread enough for it to be of interest in assessing the seismic performance of buildings to heavy shaking.

Ground surveys of earthquake damage are still useful in other ways to collect information from an earthquake region. The statistics relating to the distribution of damage to defined building types reveal the comparative performance of different forms of construction, expose the possible recurring defects in local construction methods, provide macroseismal mapping information and, more broadly, train the surveying engineer to consider issues such as widely applicable repair techniques and reconstruction options based upon a more thorough database of observed buildings in the region. Apart from considering damage, the survey results can also be used to qualify the distribution of building types within the region or town.

The EEFIT team on mission to Gujarat was faced with a large epicentral region and many cases of structure failure to visit. Only a limited amount of building-by-building damage surveying was made possible in the available time and some of the damage data used in the work which followed the expedition made use of images taken on the field or made available on the Web by other teams.

The ground damage survey focused principally on masonry building types, as these are predominant in the affected villages around Bhuj. The surveying of construction type, number of storeys and EMS damage level covered a total of 245 buildings in the villages of Sukhpur, Barasah, Nananpur, Mirjapur and in West Bhuj in the area of Uplipar Rd (see Figure 7). In order to complement the survey with records of damage from more heavily damage areas aerial photographs of settlements near the epicentre were used to give a rough assessment of which building types were more or less damaged and what size area showed signs of collapsed rubble masonry<sup>2</sup>. This type of interpretation has a high margin of error and not adequate to provide a building type comparative performance data but assists in providing a crude macroseismal mapping of the location.

### **6.2.2 Survey results**

The bar chart in Figure 5 shows the distribution of damage amongst the building types in all the surveyed locations for each damage level. To compare building performance at the same ground motion, Figure 6 shows the survey results for West Bhuj.

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<sup>2</sup> Bardet (2001)

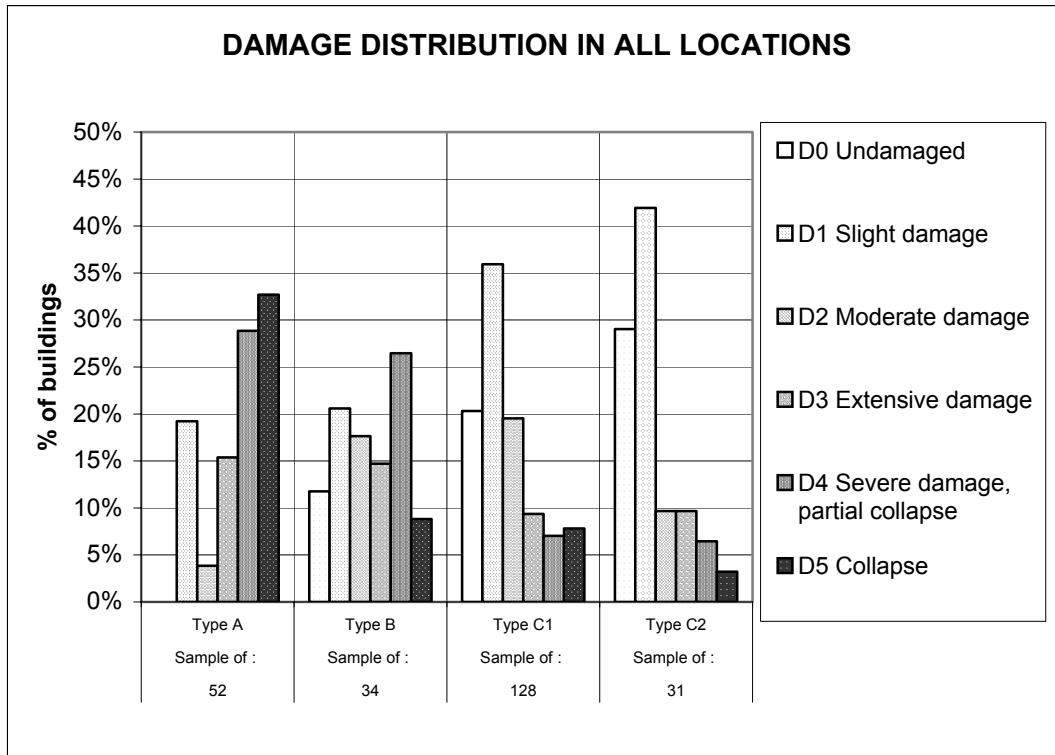


Figure 5 Non-cumulative damage distributions of all buildings surveyed in the various locations in the vicinity of Bhuj

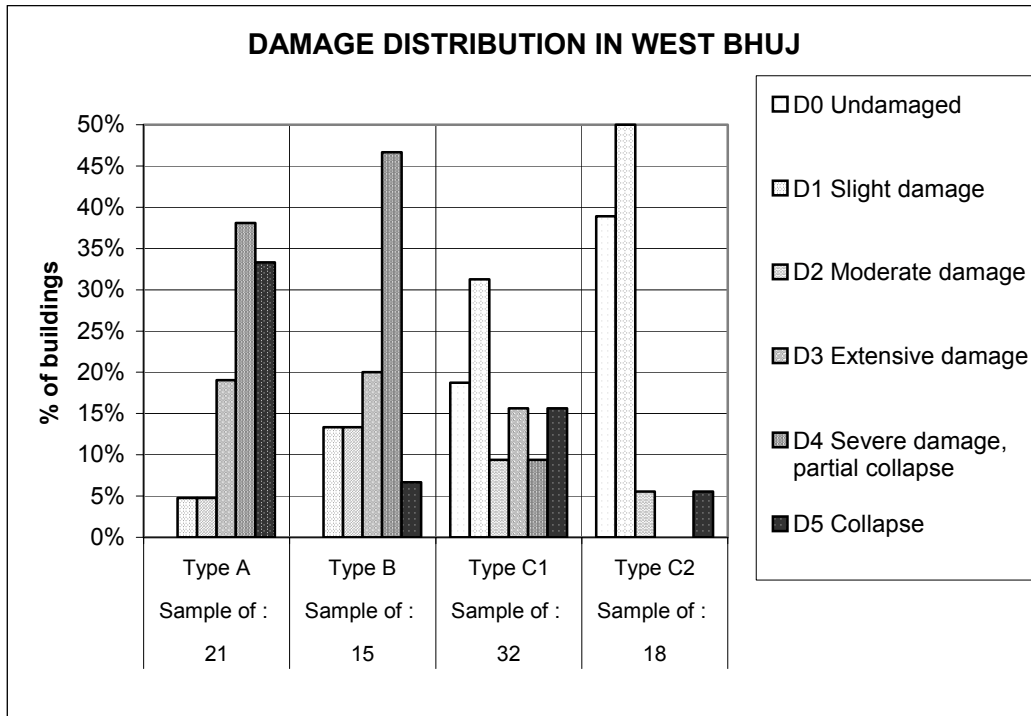
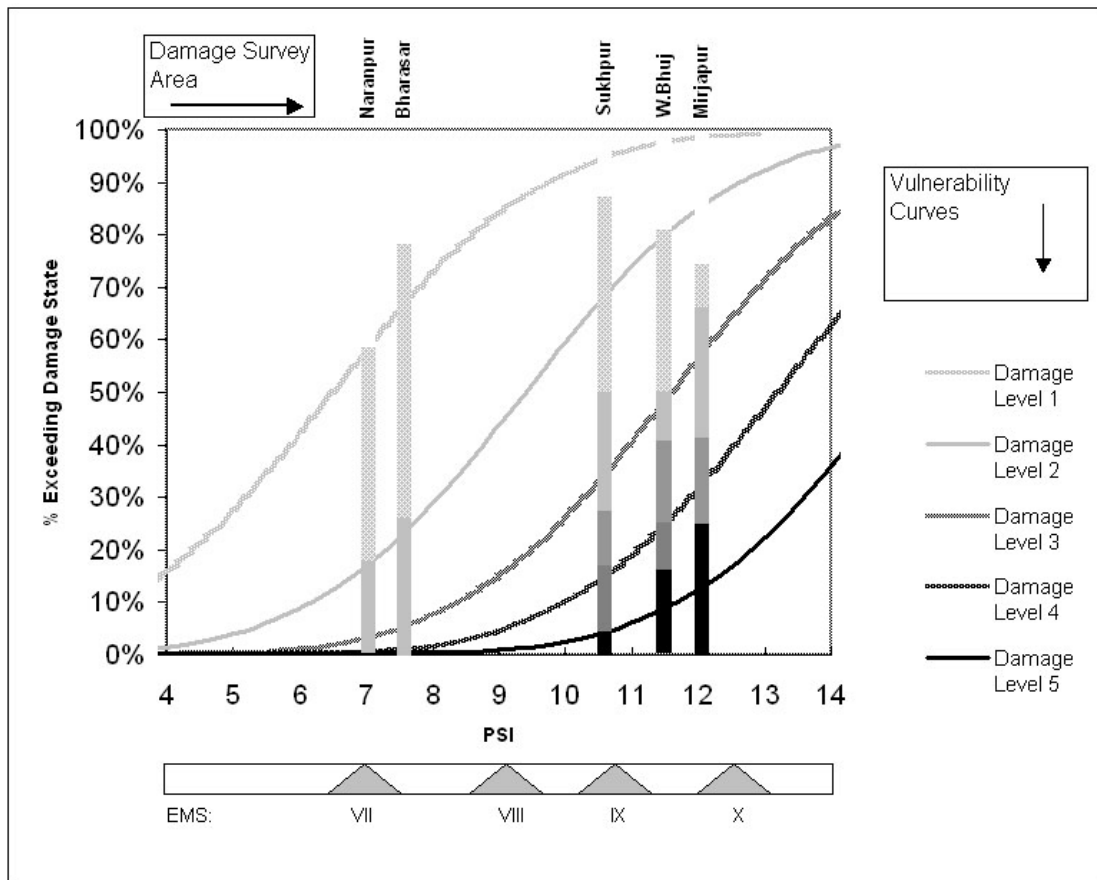


Figure 6 Non-cumulative damage distributions of all buildings surveyed around Uplipar Rd in Bhuj



Data on damage distribution allows an assessment of the EMS intensity according to the PSI (Parameterless Scale of Intensity) using a range of vulnerability functions developed from earlier studies<sup>3 4</sup>. The vulnerability functions are based on damage data from a large number of engineered and non-engineered buildings in previous earthquakes. The definition of PSI is directly from the damage experienced in the earthquake, and can be thought of as a measure of the “damagingness” of the event and hence can be converted into EMS Intensity. Figure 7 visualises the method of quantifying damage to building types in locations with a value of PSI through a best-fit routine. The value of PSI can be related directly to EMS Intensity but the studies at the Martin Centre have also correlated PSI values of damaged areas in the immediate vicinity of strong motion instruments to the parameters based on the instrument recordings. This correlation can be used inversely to estimate the ground motion from the PSI values calculated from a survey in a heavily damaged area in Kachchh, in this case Bhuj, where the greatest number of buildings was surveyed. Given that the majority of the buildings surveyed are residential 1-3 storey dwellings, the most appropriate parameter to describe the ground action is the mean response spectral acceleration in the range 0.1 to 0.3 seconds. This gives a value of MRSA<sup>3</sup> of 700cm/s<sup>2</sup>. Table 2 shows the values of PSI and EMS Intensity per building type in each location.



**Figure 7 Comparison of survey results for building type C1 with globally derived vulnerability functions**

<sup>3</sup> Spence et al. (1991)

<sup>4</sup> Coburn & Spence (1993)

| Location                  | Building Type A |          | Building Type B |          | Building Type C1 |          | Building Type C2 |          | Average PSI per location | Equivalent EMS |
|---------------------------|-----------------|----------|-----------------|----------|------------------|----------|------------------|----------|--------------------------|----------------|
|                           | PSI             | Data set | PSI             | Data set | PSI              | Data set | PSI              | Data set |                          |                |
| Bharasah (Ba.)            |                 |          |                 |          | 7.5              | 19 blgs  |                  |          | 7.5                      | VII            |
| Sukhpur (Su.)             | 10.5            | 20 blgs  | 11.5            | 5 blgs   | 10.5             | 48 blgs  | 9.5              | 4 blgs   | 10.5                     | IX             |
| Nananpur (Na.)            |                 |          |                 |          | 7                | 17 blgs  |                  |          | 7                        | VII            |
| Mirjapur (Mi.)            |                 |          |                 |          | 12               | 12 blgs  |                  |          | 12                       | X              |
| W.Bhuj (Bh.)              | 11              | 21 blgs  | 10              | 15 blgs  | 11.5             | 32 blgs  | 8.5              | 18 blgs  | 10.2                     | IX             |
| Anjar (An.)               | 11.5            | Aerial   |                 |          | 11.5             | Aerial   | 11               | Aerial   | 11.3                     | IX - X         |
| Bhachau (Bh.)             | 13              | Aerial   |                 |          | 13.5             | Aerial   |                  |          | 13.2                     | XI             |
| Kera (Ke.)                | 8               | Ground   |                 |          | 7                | Ground   |                  |          | 7.5                      | VII            |
| Dhamanka/<br>Rampar (D/R) | 11.5            | Aerial   |                 |          | 12.5             | Aerial   |                  |          | 12                       | X              |

Table 2: Survey Results

### 6.2.3 Discussion

#### *Comparison of seismic performance.*

The 26 January earthquake had a devastating effect on non-engineered structures. The length and strength of the shaking brought widespread destruction to those villages whose predominant building type was one or two storey rubble masonry buildings. The mud mortar, used to hold together the random arrangement of stones, offered no resistance to the minute-long shaking, and as the wall crumbled the heavy roof structure of timber logs and tiles collapsed.

The survey results shown in Figure 5 demonstrate the greater vulnerability of rubble type construction (Type A): the dwellings suffered from damage levels D4 and D5 caused by the disintegration of the walls. This disintegration mechanism was found less common in the properly coursed stone buildings (Type B). Here the principal damage mechanisms were the overturning of entire walls and the failure of the eaves due to the lack of connectivity between the vertical and horizontal structures. The vulnerability is greatly reduced with the introduction of reinforced concrete diaphragms (Type C1). Wall overturning is much less frequent and the majority of the damage is exhibited as X-cracks in the wall panels at the lower storey as the masonry fails by in-plane shear. In the extreme cases, the material explodes at the building corners initiating collapse. Another frequent damage mechanism of this form of construction is the sliding of the simply supported RC slab on the masonry walls. Buildings with a uniform floor plan, tied-in RC diaphragms and good quality masonry could be found undamaged even in the most heavily damage areas.

The reinforced concrete buildings included in the survey (Type C2) were located in the urban areas of Bhuj and in the larger villages. These buildings appeared to have a lower vulnerability than the more recent high-rise tower blocks (not included in the survey) in the outskirts. This difference may be a consequence of the presence of infill walls at ground floor level, rather than car parking, the more regular geometry of the frame design, rather than transfer beams and cantilevers, as well as the smaller number of storeys.

The pattern of damage in West Bhuj shown in Figure 6 is similar to that of the combined survey points. Here many of the two-storey type B buildings showed severe damage, ranked D4: overturning of walls, collapse of roof, loss of floor bearing. The mid-rise RC buildings performed well, the majority of them showing no or slight visible damage and one complete collapse of a 6-storey block.

### Comparison with Macroseismic Survey

The values of PSI computed for each building type should be similar at each location, since, as discussed above, PSI is a measure of “damagingness” of the shaking. Even with small data sets, the deviation in the sets is satisfactory. The comparison with the macroseismic mapping by the National Information Centre of Earthquake Engineering (NICEE)<sup>5</sup> holds quite well though some interesting observations may be made.

To the South of Bhuj, one may infer that the geology has a mitigating effect on the ground motion. The town of Kera (Ke.) lies directly on the Southern Kachchh Ridge, and is located directly on a basaltic rock outcrop on the Katrol Hill Fault. Nananpur (Na.) a little further north and Bharasah (Ba.) are both located on sedimentary sandstone formation with weathered sandstone outcropping at various locations within these villages. In these three villages, the PSI was found to be equivalent to an EMS Intensity of VII, one or two points below the values expected on the Nerula map.

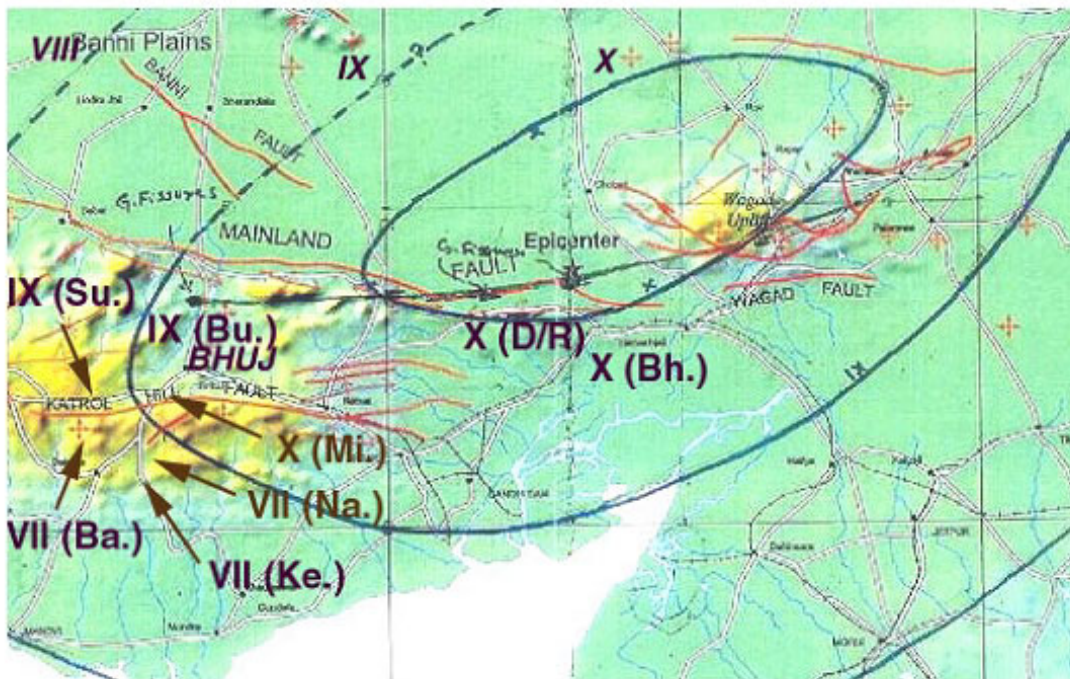


Figure 8 Seismotectonic Base map of the Kachchh area marked with preliminary isoseismals prepared by P.L. Narula of the NICEE. Map shows EEFIT survey estimates of intensity in marked locations. Refer to Table 2 for the full location names.

## 6.3 Relief and reconstruction

### 6.3.1 The relief operation and the social impact

At the time of the EEFIT visit clearing up of the debris had been underway of a fortnight, while the surviving inhabitants of the collapsed buildings slept outdoors under makeshift tents or in camps set up

<sup>5</sup> Narula et al. (2001)

by relief organisations. This was practicable as the temperatures had not yet reached the soaring summer values and the monsoon was still a couple of months away. These living conditions are anything but comfortable and the efforts of aid agencies and camp hospitals, after the initial emergency relief, were still needed weeks after the disaster. The earthquake produced a pattern of population migration. On one hand, the survivors were leaving the region, taking their belongings to stay temporarily with relatives outside the region. On the other hand, the free distribution of health care, medical material, blankets and food attracted an ever-increasing number of people from neighbouring regions. For this reason, some of the aid agencies had interrupted food distribution. The influx of relief material was such that, coupled with bad management, piles of food and medical supplies were, on February 18<sup>th</sup>, standing unused on Ahmedabad station<sup>6</sup>.

Notwithstanding these conditions, a strong spirit of endurance was displayed in the many ways of coping with the disaster. Soon after the many dead were mourned, the economic activities were restored or re-invented, whether it be on the plinth of the damaged building, on the rubble of the collapsed building or on the opposite pavement.

The reconstruction effort required is massive and no single strategy or approach could be delineated at the time of our visit. While on one hand international agencies are ‘adopting’ villages, the Government of Gujarat has pledged to pay 700 Rupees (£10) for each “fully destroyed hutment”, 200 Rupees for the repair of a half inch crack, and 40,000 Rupees (£600) for the reconstruction of a “Fully destroyed semi-pucca house”<sup>6</sup>. At the same time, fundamental lessons about relocation of whole towns and issues to do with confidence of the population in the newly built structures are there to be learnt from previous Indian earthquakes, specifically Latur. Indian NGOs have experienced these realities and are seeking to participate actively in the decisions concerning these issues.

### **6.3.2 Repair of moderately damaged masonry buildings**

Due to the spread in damage to the buildings and the varying intensity of the earthquake, buildings over a large area suffered moderate and extensive damage levels. When these buildings are repairable, the works have to be carried prior to the owners reoccupying the dangerous building.

The vast majority of residential buildings of masonry construction are owned, and often built, by the house dweller. This creates a situation where owner/builders are taking in their own hands, perhaps with advice from the local builder, the repair of dangerous constructions, with very little knowledge about repair and seismic resistance of masonry. The methods being used for these repair works were of concern to the structural engineers on the EEFIT team. Concrete columns, unconnected to any structure were being cast in building corners, sections of wall rebuilt without consideration of the reaction to horizontal forces, cosmetic repairs to cover structural damage.

The consequence of non-engineered repairs to the damaged buildings is two fold: hard earned savings are spent in a time of great need for works which do not restore or improve the safety of the dwelling; secondly any incorrect approach to repair of one villager may be followed by the remaining community when “expert advice” is claimed to lie behind the works carried out. Figures 9 & 10 show an example of the type of building being repaired and the way concrete elements are inserted in the structure.

Engineered approaches towards repairing damaged masonry buildings have been studied and applied in countries throughout the world and EEFIT members have been involved in developing and diffusing this knowledge. The situation called for EEFIT or EEFIT members to get involved beyond the pure reporting nature of the mission. To this effect participants of the EEFIT mission have offered the experience gained from the trip to India, coupled with knowledge of repair techniques to a UK-based NGO involved in the reconstruction projects in Kachchh<sup>7</sup>.

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<sup>6</sup> The Sunday Times of India (18.2.2001)

<sup>7</sup> Patel et al. (2001)





**Figure 9: Making good of damaged masonry corner.**



**Figure 10 Repair works to damaged masonry building with RC floor. The new structural element has little connection with the existing structure.**

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# **Chapter 7 Summary & Recommendations**

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## **7.1 Summary**

The powerful earthquake that rocked the western state of Gujarat in India at 08:46 hours (Indian Standard Time) on the 26<sup>th</sup> January 2001 had a Moment Magnitude of 7.9. The epicentre was near the major town of Bhuj in the Kachchh region of Gujarat. The EEFIT mission to the region that took place from 9<sup>th</sup> to 17<sup>th</sup> February 2001 consisted of 9 members from the UK Universities and Industry.

In this chapter we summarise the main conclusions from the various chapters of this report. For detailed observations and conclusions the readers are urged to look at the specific chapters. Similarly the main recommendations from various chapters are outlined in Section 7.2.

### **7.1.1 Overview of the Bhuj Earthquake**

The large magnitude of the earthquake combined with poor construction quality contributed to large-scale damage to the building stock and a high number of casualties. An overview of the location of the earthquake and the damage it inflicted on the building stock is presented in Chapter 1. The mission objectives and the route taken by various sub-groups of the EEFIT team are also presented in Chapter 1.

### **7.1.2 Seismological and Geological Data**

The Geology of the Kachchh region and that near the city of Ahmedabad are discussed in Chapter 2. There are no significant peculiarities or sudden variations in the regions effected by this earthquake. The existing faults and plate tectonics of the Kachchh region are presented and significant earthquakes of the past are discussed. The Kachchh region has witnessed major earthquakes in 1819 near the Allabund fault and 1953 near the town of Anjar.

Unfortunately there was a lack of near-field strong motion records due to the paucity of instruments in the region. The seismographs from Bhuj were unavailable as the seismometer moved during the strong motion. However, the strong motion records that are available from the Passport Office building in Ahmedabad and those recorded in the far-field near Mumbai are presented in this chapter and were analysed using FFT's and Wavelets. Following this earthquake there is a strong mood in the Indian Seismological community to increase the number of strong motion arrays in this region.

### **7.1.3 Geotechnical Aspects**

The Bhuj earthquake had many interesting geotechnical aspects. Even though the strong motion data is rather sketchy, the ground accelerations in Ahmedabad city seemed to be large in relation to its distance from the epicentre of the earthquake. There is a need to understand the existence of any directivity issues that has led to the relatively large ground accelerations in Ahmedabad. This may be important in re-evaluating the seismic zonation of the areas surrounding Ahmedabad and in establishing recommended peak ground accelerations for designing against future earthquakes.

Liquefaction was observed over very large areas in the Kachchh region. Sand boiling due to soil liquefaction was observed over vast stretches with characteristic salt crustation when dried up. Liquefaction played an important role in the damage to many civil engineering structures in the Kachchh region and to the Navalakhi port in the Saurashtra region. The Bhuj earthquake provided some interesting aspects on the performance of bridges with foundations located on liquefied soil. The superstructure stiffness of the bridge appeared to determine the likely mechanism of failure once the



foundations have liquefied. Evidence of dynamic soil-structure interaction during the strong shaking was also observed.

Earth dams in the Kachchh region have seen severe damage during this earthquake. Longitudinal cracking, failure of the up and down stream slopes, damage to appurtenant structures including the intake towers and walkways leading to the intake towers were observed. The reservoir levels in most of the earth dams were low at the time of the earthquake. However the design of appurtenant structures, particularly those that will be brought into service when dam safety has to be ensured, must be carried out to withstand large earthquakes such as this. Liquefaction resistance measures need to be undertaken at the upstream and downstream toes of the dam to ensure slope failures do not occur on the scale witnessed in this earthquake.

#### **7.1.4 Performance of Bridges, Ports and Industrial Facilities**

In this chapter the damage inflicted by the Bhuj earthquake on Bridges, Ports and Industrial facilities located in Gujarat are discussed. Some of bridges, as explained in the previous section have suffered significant damage due to liquefaction of the foundations and/or lateral spreading of the ground. There was also evidence of decks shifting on the supports and damage to abutment structures. The decks were largely simply supported and this has led to their translation during the strong motion.

While the major port of Kandla has performed reasonably well and was quickly reinstated, the bulk handling port of Navlakhi suffered severe damage. The geotechnical aspects of these are described in the previous section. Damage to service structures adjacent to the port such as bulk storage facilities and oil jetties was also witnessed.

Major industrial facilities including the large petrochemical facilities in Jamnagar have performed reasonably well. However, the ground shaking in Jamnagar would have been relatively modest given its distance from the epicentre. A cement silo has suffered catastrophic failure during the earthquake. Damage was also observed to lifelines such as water supply, electrical substations and the telephone network.

#### **7.1.5 Performance of Multi-Storied Buildings**

The Bhuj earthquake resulted in severe damage to many high rise buildings. There were some significant failures of multi-storied buildings in the cities of Ahmedabad, Bhuj, Gandhidham and in the smaller towns of Anjar and Bhachau. This chapter discusses the modes of failure of high rise buildings and presents some of the causes.

Poor construction and detailing combined with the presence of soft stories has led to many of the failures observed in the high rise buildings. The presence of heavy structural items such as water tanks on the building tops, which failed during the strong shaking has also contributed to some of the failures. The concrete that was used was of a lower grade but the design has taken this into account. Some of the retrofitting observed was either cosmetic or poorly engineered.

#### **7.1.6 Performance of Low Rise Buildings**

Building damage surveys were undertaken both in Ahmedabad and in areas surrounding the city of Bhuj. Severe damage to low rise buildings constructed using local materials was witnessed. These buildings were largely constructed from locally available sand stone forming the walls with either tiled roofs or RC slabs. There are two types of masonry constructions one used properly coursed stones while the other made use of stone rubble. The building survey indicated that the rubble walls suffered catastrophic failures and performed badly in relation to the buildings with properly coursed stone walls. These are essentially non-engineered structures but their poor performance has significantly increased the death toll. Measures to improve their seismic performance that are effective, cheap and that can adopted with available technology need to be investigated.

## **7.2 Recommendations**

A number of recommendations have been suggested throughout this report, these include:

- There is a need to improve the strong motion recording arrays in the Kachchh region. This has also been the feeling of the Indian Seismological Community and it is understood that steps are being taken to redress this issue.
- The seismic zonation of the region needs to be re-evaluated with micro-zonation maps being produced for densely populated areas i.e. The cities of Ahmedabad, Bhuj, Gandhidham etc.
- There is a need to upgrade the Peak Ground Accelerations (PGA) used for design in the various seismic zones.
- The reasons for the high ground accelerations felt in the city of Ahmedabad which is some 160 miles from the epicentre need to be investigated.
- Liquefaction mitigation measures need to be employed for important bridges, ports, lifelines and industrial facilities as part of the repair and reconstruction process and for any new structures planned in this region.
- Seismometers need to be installed in the vicinity of all the major earth dams in the Kachchh region.
- Earth dams and their appurtenant structures such as intake towers in the Kachchh region need to be designed against earthquake damage. Liquefaction of up and downstream toes of these dams must be prevented by using known liquefaction resistant measures such as in-situ densification, provision of displacement piles and pore pressure relieving features such as vertical stone columns.
- Earthquake resistant design of bridges needs to be considered both as retrofit measure for existing bridges and for any new bridges planned in the region. Planned structural redundancy must be considered.
- Implementation of the building code regulations for multi-storied structures needs to be given a high priority. While improvements to the current Indian codes dealing with earthquake loading can be brought about, it is felt that significantly better performance of the multi-storied buildings would have resulted during this earthquake even if the existing codes were implemented properly.
- Quality control measures must be put in place for all new buildings.
- Ductility based design of structures must be incorporated.
- Structural detailing particularly near the beam-column junctions must be improved with adequate shear reinforcement being provided.
- Performance of the low-rise buildings constructed using locally available materials must be improved. This factor could lead to a significant reduction of casualties in future earthquakes.
- Research is needed to investigate and improve the performance of the above buildings.
- Low cost repair measures that improve the earthquake resistance of the local building stock are necessary. A repair guide that is freely available on the internet has been put together. (see <http://www.arup.com/geotechnics/HTML/Articles/DesignGuide.htm>).

