On January 26, 2001, an extremely severe earthquake (magnitude 7.7) struck western India (just south of Pakistan) centered around Gujarat. The epicenter of the earthquake was only 14.1 miles (22.7 km) below the surface. Many buildings, bridges, and other structures either collapsed or were severely damaged. Based on an inspection visit to the earthquake site, the authors provide an assessment of the damage to existing bridge structures and bridges under construction. This damage is attributed mainly to the lack of adequate seismic design and detailing.

A massive earthquake, described as one of the worst earthquakes in India’s history, struck Gujarat on Friday, January 26, 2001, at 8:46 a.m. on India’s Republic Day. The earthquake was of magnitude 7.7, with a relatively shallow epicenter at a depth of only 14.1 miles (22.7 km) below the ground surface. The infrastructure was torn apart and many bridges and culverts suffered damage. About 20 bridges were seriously damaged and several box culvert structures collapsed.

The National Science Foundation (NSF), Earthquake Engineering Research Institute (EERI), and the NSF PEER Center sponsored post-earthquake damage assessment visits to India. As one of the teams, the authors inspected the earthquake site between March 14 and 24, 2001. In the
March-April 2001 PCI JOURNAL, Dr. S. K. Ghosh covered the seismic damage mainly to building structures. This paper focuses on first-hand observations of damage to bridge structures. Such damage is primarily attributed to the lack of adequate seismic design and detailing for both existing bridge structures and bridges under construction.

Typical bridges in the Kachchh region of India have simple spans of approximately 50 ft (15 m) each, with expansion joints at each pier. These structures cross rivers, streams, and railroads. L-shaped abutments are typically used for all new concrete and old masonry bridges. Both old bridges and those under construction suffered extensive damage during the earthquake. Aside from the damage from the earthquake, the condition of cast-in-place concrete bridges is, in general, unsatisfactory and substandard.

Earthquake damage to bridge structures can be attributed to the lack of seismic-based design and detailing. Bridges are typically composed of multiple simple spans supported on elastomeric bearing pads with neither continuity of the superstructure nor fixity at the intermediate diaphragms. The substructures of most bridges are wall piers supported on shallow foundations with no consideration for ductility nor any thought for the use of deep foundations where liquefaction and lateral spreading would be expected in a seismic event.

Precast concrete members are occasionally used in bridge construction. Precast, prestressed concrete bridges performed better than cast-in-place concrete bridges and other types of bridges. The superior performance of precast, prestressed concrete bridges can be attributed to the high quality of fabrication of precast members.

SEISMOLOGY OF THE REGION

The Kachchh region is located in Gujarat in western India, south of Pakistan and north of the Gulf of Kachchh. The region is situated in Zone V, the severest zone on India’s seismic map (see Fig. 1). Gujarat has a long history of severe earthquakes, although such intraplate earthquakes are less frequent than those associated with subduction of the Indian Plate beneath the Asian Plate along the Himalayan front. The last two severe earthquakes in Gujarat took place in 1956 and 1819, with magnitudes of 7.0 and 8.0, respectively.

The epicenter of the January 26 earthquake was a few miles from the town of Lodai in central Kachchh. The tectonic setting of the region is characterized as a relatively stable continental region similar in nature to the Midwest and Eastern United States. The ground acceleration records from the nearest working strong motion station in the city of Ahmedabad (in Zone III), some 135 miles (225 km) away from the epicenter, indicate a peak acceleration of 0.11g. Fig. 2 shows that the thrust fault associated with the earthquake did not rupture the ground surface. East-northeast compression rides and fissures near the epicenter are largely the result of lateral spreading of the soil crust over soil beneath that liquefied.

It has been estimated that 4000 square miles (10000 km²) of highly susceptible deposits in low-lying salt flats, estuaries, intertidal zones, and young alluvial deposits liquefied. Dissipation of pore water pressure from liquefaction over a wide area around Lodai, which is at sea level, caused water to pipe to the surface and erupt through boils and fissures to a reported height of 6 ft (1.8 m). After a couple of days, according to local sources, such flow diminished but persisted for several weeks. As seen in Fig. 3, there was still standing water in places six weeks later, despite the warm climate.

Ground cracks and sand boils with dried ejected sediments were observed throughout the central region. Cracks similar to those shown in Fig. 4 ranged...
from 6 to 24 in. (150 to 600 mm) wide and up to 6 ft (1.8 m) deep. There was little or no vertical separation or elevation difference across these extensional features.

PERFORMANCE OF ROADS AND BRIDGE STRUCTURES

The Gujarat Roads and Buildings Department administers design, construction, and maintenance of roadways, bridges, and other transportation structures. National Highway 8 (NH8) is the most important road in the Kachchh region. The new Highway NH8A connects local ports and towns to India’s highway system. This roadway is still under construction and is planned as a four-lane modern divided toll road (see Fig. 5). This replacement of NH8A is being constructed at a higher elevation than the existing road to better accommodate monsoon flooding.

Damage to Roads

Local roads in Gujarat are mostly two lanes between towns and one lane between villages. Such roads carry a low volume of vehicular traffic with very few heavy trucks. After the earthquake, these roads were crucial for accessibility and emergency response to remote areas.

Newly finished roadway embankments suffered some damage. Longitudinal cracks 2 to 4 in. (50 to 100 mm) wide and 12 in. (300 mm) deep developed along the shoulder and edge of the traffic lanes (see Fig. 6). Settlement of the shoulder edge and holes about 24 in. (600 mm) in diameter and 18 in. (450 mm) deep also occurred. A longitudinal crack separated the entire guardrail system from the roadway shoulder. Stone blocks that were laid and mortared in place on the face of the embankment suffered distress from the earthquake-caused settlement and downward slope movement of the underlying soil.

Damage to Traffic Bearing Drainage Structures

Lack of structural capacity resulted in the collapse of traffic bearing roadway drainage structures (see Fig. 7). These structures included concrete box culverts, concrete pipes, and unreinforced masonry box culverts. To accommodate post-earthquake traffic, temporary detours were provided across adjacent dry season riverbeds. Damaged drainage structures are not repairable and should be rebuilt to proper design and construction standards.

Fig. 2. Type of fault associated with the earthquake.

Fig. 3. Water on the surface near Lodai, six weeks after the earthquake.
Damage to Rudramata Bridge

Poor construction, a harsh environment (monsoon/typhoons and saline ground water accompanied by hot dry weather), and little maintenance contributed to the substandard condition of area bridges. A lack of suitable materials, poor workmanship, concrete deterioration, and reinforcing steel corrosion are common to most roadway bridges.

The Rudramata Bridge, built in 1966, is the largest precast, prestressed concrete girder bridge in the region and one of the few bridges that performed well during the earthquake. It is located on State Highway 45 in north central Kachchh, about 10 miles (16 km) from the epicenter of the earthquake. The main structure of the bridge performed relatively well during the earthquake, but the north end approach suffered damage, resulting in the closure of one lane of traffic.

The bridge is 24 ft (7.3 m) wide and is composed of 10 simple spans of 55 ft (16.8 m) each, with expansion joints at the piers. The superstructure consists of two precast, prestressed girders with cast-in-place concrete decking and diaphragms. The substructure elements are reinforced concrete towers each supported on a large diameter caisson. Fig. 8 shows the Rudramata Bridge and the collapse of the end approach and traffic barrier.

Elastomeric bearing pads support...
the prestressed girders at both ends and at intermediate piers. There were no longitudinal restrainers or transverse stops to control the superstructure movement on the piers. During seismic excitation, each expansion joint shifted off the pier center. Different opening of the expansion joints on opposite sides of the deck indicates that the superstructure rotated. This behavior was typical for continuous bridges made of simple spans supported on elastomeric bearings.

Lateral spreading and ground cracking at the north pier resulted in settlement of the bridge approach. Ground cracking at the north abutment ran parallel to the stream bank (see Fig. 9). Cracks were 6 to 12 in. (150 to 300 mm) wide and up to 4 ft (1.2 m) deep. There was no noticeable displacement of the end or intermediate piers due to ground movement. Gapping or separation between the pier caissons and the surrounding ground of up to 12 in. (300 mm) occurred due to movement of the surface soil.

The tower of Pier 2 suffered cracking (see Fig. 9). The cracks were mostly at the beam-to-column connection and along the exterior face of the columns. Cracks were primarily shear cracks at the connections and are due to the bending of the tower under the seismic loads. The maximum crack size was approximately $\frac{1}{4}$ to $\frac{1}{2}$ in. (6 to 13 mm) in width.

Damage to Sujabari Bridge

The Sujabari Bridge is the longest bridge in the region. Built in the 1950s, it consists of 36 spans crossing the Gulf of Kachchh on Highway NH8. The bridge superstructure is a cast-in-place box girder supported on reinforced concrete wall piers and bearing on elastomeric pads. The substructure is supported by well foundations composed of a stack of precast concrete rings augured into the ground to the desired tip elevation and covered by a cast-in-place reinforced concrete cap for wall pier support. Given the age of the bridge, seismic analysis and detailing were not considered in its design. There was neither continuity nor ductility in the structure; therefore, each span acted independently, and the entire structure experienced a collection of out-of-phase dynamic motions or modes.

Sand boils, lateral spreading, and settlement were widespread in the gulf. The entire area under and around the bridge liquefied as a result of the earthquake. A large number of sand boils up to 2 ft (0.61 m) wide were evident along the bridge alignment. The damage to the bridge indicates that there was both longitudinal and transverse movement of the bridge superstructure and substructure. The embankment at the north end of the
bridge settled approximately 12 in. (300 mm) and moved toward the channel. This settlement and lateral spreading of the embankment extended to the bridge abutment and resulted in settlement of the roadway. The north abutment also moved toward the channel. The bearings and expansion joint at the abutment were completely dysfunctional and may not be repairable.

The Sujabari Bridge suffered significant damage (see Fig. 10). Because of its importance as a major transportation link, the bridge was kept open with one traffic lane a full month after the earthquake and until construction of the new bridge (parallel to it) was completed. The bridge was then closed to all traffic until a decision is made to repair or replace it.

The ground movement and liquefaction caused some piers to move and associated spans to shift on their bearing supports. The superstructure slid off its bearings at several intermediate piers and had to be jacked back to its original position. Pier 14 rocked off its foundation, but it was immediately repaired and limited traffic allowed on it. Both substructure and superstructure moved in longitudinal and transverse directions (see Fig. 11). The shallow well foundation with wall pier sup-
ported on top moved in the liquefied sand; there were no transverse stops to prevent such lateral movement. The excessive lateral load caused the elastomeric pads to yield, which allowed the superstructure to slide off the bearings. Due to the large size of the pier cap, the bridge did not collapse.

The in-span hinges suffered some damage with shear failure of the inclined faces of the hinge (see Fig. 12). The concrete traffic rails and balusters at the expansion joints were severely damaged due to the colliding of the superstructure segments. The expansion joints were closed, rotated, and had popped out at various locations along the bridge. The cracks in the balusters extended into the bridge deck slab. Traffic rail elements sheared off at the connections.

Fig. 12. Failure of in-span hinge and traffic railings.

Fig. 13. Failure of superstructure and substructure.

Damage to Older Bridges

Bridges on Highway NH8A have two lanes for traffic and wide unpaved shoulders. They are composed of multiple spans with an expansion joint at each pier. Spans of 50 ft (15 m) are cast-in-place flat slabs or tee-beams. The superstructure is supported on reinforced concrete wall piers, masonry wall piers, or concrete arches with masonry fascia walls on shallow foundations. Elastomeric bearing pads are typical for all bridges.

These bridges are all in a poor condition, with spalled concrete and exposed corroded reinforcing bars. Past attempts at repair using a shotcrete layer has not protected the structure from deterioration. Despite their poor structural condition, none of the bridges collapsed during the earthquake. This is in part due to the use of the short spans and large wall piers. Some bridges exhibited more damage than others.

The bridge shown in Fig. 13 did not collapse during the earthquake, but it did suffer serious damage to end diaphragms, end pier walls, backwall, bearings, and traffic barriers. The traffic barriers are in most cases post-and-beam type and their connection to the balusters and slab were completely deteriorated even before the earthquake. Given the need to maintain traffic flow on Highway NH8, temporary supports allow for its continued use, even though the damage incurred dictates that it be replaced.

Shallow foundations moved laterally with the dried crust of near-surface soil in which they were embedded. Cracking of the soil surface was likely caused by lateral spreading over liquefiable material below the surface. Such ground separation [openings of 6 to 12 in. (150 to 300 mm)] caused differential pier movements. Because of the lack of fixity on top of the piers, however, no significant bending or joint failures occurred during the earthquake. There was some tilting of wall piers, which, due to the large size of the pier cap, did not cause concern.

The out-of-phase and uncontrolled movement of the bridge elements resulted in banging at the expansion joints and dislocation of the superstructure at the bearings. In some
cases, the expansion joint shifted on the pier wall. With the poor condition of the concrete, damage of the slab at the expansion joint grew to include the cantilever slab. Concrete at the end of the girder was sheared off and the bearing area was significantly reduced (see Fig. 14).

Vertical shear-friction cracking appeared at the end of the girder. The substructure experienced diagonal cracking in the pier cap and vertical cracking in the wall pier. Cracks were \( \frac{1}{2} \) to 1 in. (13 to 25 mm) wide and require immediate repair. Epoxy injection may be considered an appropriate repair remedy.

The masonry wall piers shown in Fig. 15 are supported on a continuous raft footing approximately 3 ft (0.9 m) deep. The superstructure is supported on bearings, which over the years have completely deteriorated. Due to the longitudinal movement of the bridge, the fixed connection failed and the end diaphragm cracked vertically. The end span sagged about 2 in. (50 mm), resulting in cracking and spalling of the concrete at the bottom of the slab.

The continuity of the arches and footing allowed the bridge to act as a continuous structure during the earthquake. Concrete arches performed well, but they suffered some damage to the masonry fascia walls. Minor cracks were observed in the reinforced arches but were closed under the compressive arch action.

**Damage to Bridges Under Construction**

The new Sujabari Bridge (adjacent to the older structure) was nearly complete (two spans to finish) at the time of the earthquake. The bridge was completed within a month after the earthquake, and traffic was diverted from the existing damaged bridge to this new bridge. The new bridge is a cast-in-place tee-beam girder bridge with the same number of spans and expansion joints as the older bridge.

The substructure consists of hammerhead piers supported on well foundations. There are transverse stops at pier caps, but there are no longitudinal restrainers at the expansion joints. Minor damage occurred at the expansion joints of both end and intermediate piers, which was repaired as the bridge was completed. The expansion joint at the end pier is supported by a short cantilever span to an L-abutment (see Fig. 16).

Overall, the new Sujabari Bridge performed relatively well. Its sound performance can be attributed to the structural design of the bridge, such as the use of more flexible hammerhead piers (instead of rigid wall piers) and transverse girder stops at piers to prevent the transverse movement of the superstructure.

There were ten other bridges under construction on Highway NH8A at the time of the earthquake. Some were almost complete while others still had falsework in place for superstructure construction. They are located adjacent to older bridges so that, once finished, each bridge can take two lanes of traffic in one direction.

These bridges were designed and detailed with the same structural concepts as the older bridges. They have 50 ft (15 m) spans with no continuity (expansion joints at each pier) nor provisions for ductility. They have cast-in-place superstructures and substructures with L-abutments at end piers. There was no indication of seismic design and detailing despite the high potential for liquefaction in Seismic Zone V. Shallow foundations, rather than deep ones, were employed. The failure of abutments, piers, expansion joints, and approach slabs was typical among the bridges under construction (see Fig. 17).

Due to the similarity of the structures, damage to these shorter bridges was nearly the same. Unfortunately, the damage to these bridges under construction was more significant than...
that of the existing or older structures. The description of damage given here can be considered systematic to all. Due to the shorter end span of the bridge shown in Fig. 18, the superstructure was changed from a tee-beam to a shallower flat slab.

To accommodate this difference in superstructure depth, an auxiliary cross-beam was provided on top of the main cross-beam. Due to the movement of the pier toward the dry stream bed, the fixed connection between the two cross-beams failed and the top cross-beam slid off the lower cross-beam. The expansion joint shifted about 15 in. (375 mm) from the pier centerline.

The relative longitudinal movement of the bridge superstructure and the abutment caused the superstructure to collide with the abutment backwall, resulting in cracking and spalling of the concrete at the base of the backwall. The push from the superstructure caused the collapse of the abutment backwall, which then pushed into the approach slab. The approach backfill settled and spread out toward the wingwalls. Consequently, the connection between wingwall and abutment wall failed as shown in Fig. 19.

Settlement of the approach backfill of between 12 and 18 in. (300 to 450 mm) was typical. The repair of this end pier requires extensive work, including rebuilding of the abutment wall, expansion joint, backwall, wingwalls, backfill, and approach slab.

The expansion joint shown in Fig. 20 shifted about 12 in. (300 mm) from the pier centerline. The segments of the bridge deck compressed and closed the expansion joint, causing spalling of the concrete at and around the joints. The transverse motion caused flexure, resulting in more damage at the outer edge of the slab than at the centerline. The restraint provided by the superstructure prevented the top of the pier from moving as much at its base (anchored to the moving ground). The stability of this and other piers should be investigated for eccentric loading and tilt. The adequacy of the bearings should be confirmed when the girders are repositioned. The girders on one side of the pier are barely seated on their bearings, which may result in
shear-friction failure of the girder end.

SEISMIC DESIGN
OF BRIDGES

Bridges should be designed and detailed to minimize their potential for seismic damage. Bridges that are designed and detailed in accordance with the seismic requirements may suffer damage, but should have a low probability of collapse due to seismically induced ground motion. Bridges may be classified for their importance by the local jurisdiction. Methods of analysis, minimum support lengths, pier design details, and abutment design procedures should be specified based on the requirements of seismic zones.4

Single-Span Bridges or Continuous Bridges Composed of Single Spans

Connections should be designed to restrain movement between the superstructure and substructure. Seat widths at expansion bearings of multiple-single-span bridges should accommodate the maximum possible accumulated displacement in the longitudinal direction of the bridge, with consideration for the effect of span length, abutment height, and skewed supports.

Transverse stops and longitudinal restrainers should be provided at all expansion joints to prevent excessive movement of the bridge superstructure. Restrainers may be provided between columns or piers at the expansion joints to minimize the relative movement between superstructure and substructure.

Expansion Joints

A primary focus should be the elimination of expansion joints at the bridge ends and at the intermediate piers. At expansion joints, the reinforcement should be epoxy coated, and a polymer or other type of durable concrete should be used.

Ductility

The response of structural components and connections can be characterized by ductile behavior, which is based on significant inelastic deformations before any loss of load carrying capacity occurs. It is uneconomical to design a bridge to resist seismic forces elastically. Ductility factors scale elastic forces to lower inelastic values where seismic forces exceed their design level. The ductility factors for connections are smaller than those for substructure members in order to preserve the integrity of the bridge under these extreme loads. For expansion joints within the superstructure, application of a ductility factor results in magnification of applied forces.

Columns

The column may yield in the transverse or longitudinal direction with plastic regions generally located at the top and bottom of columns. Shear failure of columns due to the seismic loads should be avoided by appropriate design and detailing of transverse reinforcement. The main function of transverse reinforcement is to ensure that the column ends are adequately confined after spalling and to prevent buckling of the longitudinal reinforcement.

The amount and spacing of transverse reinforcement at the confinement regions are important. The concrete’s contribution to shear resistance within the plastic hinge zone is not ensured, particularly at low axial load levels, because of full-section cracking under load reversals.

End Piers

The effect of active earth pressure amplification of the earth mass retained by the abutment wall and wingwall should be considered. Appropriate methods for determining the equivalent static fluid pressures of backfill soils retained by the abutment and for saturated soils susceptible to liquefaction should be used.
Seismic design forces should account for wall inertia forces as well as equivalent static forces. At abutment walls, seismic design forces should also include seismic forces transferred from the bridge superstructure through the bearing supports that do not slide freely.

Wall Piers

Wall piers may be analyzed as a single column in the weak direction and a wide column in the strong direction provided the appropriate ductility factor for that direction is used. Wall piers with aspect ratios greater than 2.5 have low ductility capacity and no redundancy. A small amount of inelastic deformation is expected when subjected to seismic forces. As a result, a lower ductility factor should be used in determining the reduced design forces.

Detailing

Loss of concrete cover due to spalling at plastic hinge zones requires careful detailing of the confining reinforcement. In these zones, a lap splice of transverse reinforcement should be avoided because spirals or hoops may unwind. In seismic zones, it is a desirable practice to avoid a lap splice of the longitudinal reinforcement with dowels at the base of a column.

In wall piers, a minimum reinforcement ratio should be maintained both vertically and horizontally, and should be distributed uniformly in both directions. Two layers of reinforcement are required in wall piers to ensure substantial design shears, based on the premise that two layers of reinforcement will tend to retain the integrity of the wall after cracking of the concrete.

CONCLUSIONS AND RECOMMENDATIONS

The January 26, 2001, Gujarat earthquake was, for bridge engineers, a demonstration of the need for reliable seismic design and detailing, and for greater focus on the quality of construction and the use of durable materials. Based on select information acquired from a brief visit to the earthquake site, the authors offer the following conclusions and recommendations:

1. The main reason for damage to both existing bridges and bridges under construction was the omission of seismic design provisions and detailing. Appropriate specifications with respect to the regional seismic requirements should be considered in the design and retrofit of such bridges.

2. A seismic retrofit program should be considered for bridges currently under construction. The program should provide for longitudinal restrainers, transverse stops, and column strengthening to meet the requirements for shear capacity, ductility, and confinement.

3. Shallow foundations for bridges should be avoided where the potential for liquefaction is present. Deep foundations, including those with driven piles and drilled shafts, should be considered in the design of bridges under construction and the retrofit of bridges.

4. Superstructure continuity and use of integral or semi-integral abutments should be encouraged. The seismic performance of a bridge benefits from the elimination of expansion joints. Repair and maintenance of expansion joints are costly and time consuming. If expansion joints are used, special attention should be given to restraining the adjoining segments, detailing and the quality of the materials employed in construction.

5. More importance should be given to the use of precast, prestressed concrete members. Precast members are more desirable than those of cast-in-place concrete because of the better quality obtained in their fabrication. The practicality of producing, shipping, and erecting precast, prestressed concrete members should be given high priority.

6. Due to the salt-water environment, the use of high performance concrete is recommended for improved durability. Epoxy coated or other type of corrosion-protected reinforcement should be used in bridge construction. Improving initial quality will result in longer service life for bridges and will reduce future maintenance and repair costs. Greater importance should be given to the curing of cast-in-place concrete by specifying continuous wet curing for an extended period of time.

ACKNOWLEDGMENT

The National Science Foundation (NSF) and the Earthquake Engineering Research Institute (EERI) along with the NSF PEER Center sponsored this post-earthquake damage assessment visit to India. The authors greatly appreciate this support.

The authors also want to express their appreciation to the PCI JOURNAL reviewers for their thoughtful and constructive comments.

REFERENCES


2. India Earthquake Internet Websites and Local Sources.