up to 1.3 m (50 in) in amplitude. For the most part, these oscillations were not considered to be dangerous. Studies of the bridge vibration problem were undertaken, including wind tunnel testing of models at the University of Washington at Seattle, and modifications were made, such as the installation of cable ties attached to concrete anchors. These cables broke three or four weeks before the collapse. Deflector vanes to change the aerodynamic characteristics had also been developed, but their installation was under negotiation at the time of the collapse. After the failure, a board of engineers was appointed by the Administrator of the Federal Works Agency to determine the causes of failure.

Lessons Learned

The Board of Engineers concluded that the bridge was well designed and built to resist safely all static forces. Its failure resulted from excessive oscillations made possible by the extraordinary degree of flexibility of the structure. The Board determined with reasonable certainty that the first failure was the slipping of the cable band on the north side of the bridge to which the center ties were connected. This slipping may have initiated the torsional oscillations. The Board recommended more studies to understand the aerodynamic forces acting on suspension bridges.

Thus, incompetence or neglect was not the cause. The failure was due to the torsional oscillations made possible by the narrow width and small vertical rigidity of the structure. Those actions and forces were previously ignored or deemed to be unimportant in suspension bridge design. This failure emphasized the need to consider aerodynamic effects in the design of a suspension bridge. Modern bridge decks are designed to eliminate the aero-elastic instability that pushed Galloping Gertie to failure.

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PEACE RIVER BRIDGE
(1957)

The Peace River Bridge on the Alcan Highway in British Columbia failed on October 16, 1957, when the north concrete anchorage block moved forward some 3.7 m (12 ft) on its shale base.

The Peace River Bridge was part of a rush wartime program to complete the Alcan Highway connecting the United States with Alaska. The suspension bridge had a main span of 283 m (930 ft) and the side spans between the towers and cable bents were 142 m (465 ft) each. Simple truss spans connected the cable bents to the anchorages. The roadway was 7.3 m (24 ft) wide and the center-to-center spacing of the cables was 9 m (30 ft). The cables, made of twenty four 5 cm (1 7/8 in.) strands, were arranged in rectangular form with dimensions of 15 cm by 10 cm (6 in. by 4 in.). The stiffening trusses were 4 m (13 ft) deep.

The Bridge was designed and constructed by the Bureau of Public Roads, the predecessor of the Public Roads Administration. Because of the rush nature of the job, no piling was used to support the anchorages. The sliding of the anchorage on the shale base caused slacking off of the main cables, tipped over the cable bent, dropped the side span suddenly, ripping loose from its 6 cm (2.5 in.) hangers. The first indication that the anchorage was moving came about 12 hours before the collapse when the water supply line crossing the bridge for the new scrubbing plant of the Pacific Petroleum Company was cut. The bridge was immediately closed to traffic. A large crowd gathered to witness the collapse, which was thoroughly photographed. The Canadian Army Engineers put a small ferry 16 km (10 miles) downstream to provide essential transportation for Yukon and Alaska.

Lessons Learned

In order for a suspension bridge to support the applied loads in the intended manner, it is essential that the anchorages be securely fixed to the ground. Any horizontal motion of an anchorage will cause slackening of the cables with the possibility of collapse of the structure. The Peace River anchorages were supported on footings which did not stay fixed at the intended location.

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THE SECOND NARROWS BRIDGE
(1958)

On June 17, 1958, the falsework of the partly completed Second Narrows Bridge in Vancouver, British Columbia buckled and plunged two spans of the bridge into Burrard Bay. Fifteen men died in the collapse and twenty were injured. The six-lane cantilever truss bridge was to be an important link between the cities of Vancouver and North Vancouver. The main cantilever structure was 620 m (2034 ft) long consisting of a 335 m (1,100 ft) cantilever span, and two 142 m (467 ft) anchor spans. In addition the bridge had four 87 m (285 ft) steel truss and nine 37 m (120 ft) prestressed concrete approach spans. The construction of the bridge began in February 1956 and was to be completed by the end of 1958 at a cost of $16 million.

The two sections that fell were the partly erected north anchor span and a completed simple truss span adjacent to it. The workers were moving additional steel to the overhanging end when the collapse occurred. The bent supporting most of the 20 MN (2,000 ton) anchor span buckled, dropping one end of the span into the water. The impact moved the top of the permanent concrete pier by a few ft, plunging the adjacent simple span into the water.

Lessons Learned

To determine the reason for the failure of the temporary supports, an investigation was carried out under the British Columbia's Supreme Court Chief Justice, Sherwood Leu. The investigation revealed that the bent supporting the cantilever bridge section was not properly designed. The grillage was designed by comparatively inexperienced engineers without effectively checking the calculations. The bridge was completed in July 1960.

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KING STREET BRIDGE

(1962)

The King Street Bridge was an all-welded steel girder structure consisting of three main sections, a high level section and two lower level spans which flanked both sides of the high level portion. The spans served to carry roadways over the Yarra River and were completed on April 12, 1961. On the morning of July 10, 1962, brittle fracture failure occurred at points 4.9 m (16 ft) from the ends of one of the 30 m (100 ft) long approach spans under a load of 470 kN (47 tons), which was within the permissible design limits for the bridge, at a temperature of -1 degree Centigrade (30 degrees Fahrenheit). Three of the four girders fractured at points 4.9 m (16 ft) from both the southern and northern ends whereas the fourth one failed only at one position, namely 4.9 m (16 ft) from the southern end. The failure of the four girders was attributed to a combination of three factors: inappropriate steel for welding, unsatisfactory design details and low ambient temperatures.

The steel used, British Standard 968.1961, is similar to ASTM A 440 and was commonly used in riveted and bolted construction. Welding of such high carbon steel often results in weaknesses being generated in the heat affected zones and the triggering of lamellar tearing failures. Lack of preheating in the short transverse welds at the ends of the cover plates which terminated at the position of fracture is thought to have contributed to crack initiation.

The thickening of the flanges at the points of maximum tensile stress by the addition of cover plates was not a favorable design feature. The temperature on the day of collapse was below that at which the transition from ductile to brittle steel characteristics occurs. Brittle behavior favors crack initiation and propagation by increasing the stress intensity factor at any surface or interior flaws. These conditions were found to contribute to the failure of the King Street Bridge. The bridge was repaired by externally prestressing the girders with steel cables.

Lessons Learned

Inappropriate steel selection, undesirable design details and unusually low temperatures were the main contributory factors leading to the failure of the King Street Bridge. While the low temperature could not have been avoided, the other aspects were within the control of those involved in the bridge's inception.

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POINT PLEASANT BRIDGE—SILVER BRIDGE

(1967)

The Point Pleasant I-bar suspension bridge between Point Pleasant, West Virginia and Kanagua, Ohio which was built in 1928, failed at 5:00 p. m. on December 15, 1967. Forty-six people died in the accident and thirty-seven vehicles on the bridge fell with the bridge.

The center span was 213 m (700 ft) long and the side spans were 116 m (380 ft) each. The bridge was unique in that the stiffening trusses of both the center span and the two side spans were framed into the eyebar chain to make up part of the stiffening truss.

Investigation of the failure indicated that the collapse of the Point Pleasant Bridge was caused by a defective eyebar at joint 13 of the north chain, approximately 15 m (50 ft) west of the Ohio Tower. The bar which connected Joint 11 to Joint 13 developed a cleavage fracture in the lower portion of its head. Once the continuity of the suspension system was destroyed the bridge collapsed suddenly.

Lessons Learned

An eyebar suspension bridge is not a redundant structure. Failure of one eyebar is sufficient to cause collapse. If a bridge of this type is to be constructed, close and frequent inspection of the structure is necessary.

The tragedy of the failure of the Pleasant Point Bridge led to the national policy for bridge inspections. In 1968 the United States Congress enacted the National Bridge Inspection Standards (NBIS).

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ANTELOPE VALLEY FREEWAY INTERCHANGE
(1971 & 1994)

A few structural failures can be considered milestones in that they have had a far reaching impact on design codes and construction techniques. One such failure was the collapse of the Interstate 5/14 Freeway south connector overcrossing during the 1971 Sylmar earthquake. The overpass was in the final stage of construction, of prestressed concrete box girder design, 411 m (1349 ft) long over nine spans, having a section 10.3 m (34 ft) wide and 2.1 m (7 ft) deep. The longest column of the overpass, which was 42.7 m (140 ft) high, had an octagonal section 1.8 m by 3 m (6 ft by 10 ft) with no enlargement where the column intersected with the bottom of the beam section. The foundation for this column consisted of a 6 m (20 ft) deep, 2.4 m (8 ft) diameter cast in place drilled concrete shaft founded onto bedrock. Reinforcement for the column consisted of fifty-two 57 mm (No. 18) bars longitudinally, tied by 13 mm (No. 4 bars) at 30 cm (12 in.) on center. This column supported the center of a 117 m (384 ft) long section of the overpass which was connected to the rest of the bridge by way of two shear key type hinges on both ends of the box girder section. The shear keys were 17.8 cm (7 in.) deep vertically and 35 cm (14 in.) long. The sections were also tied together by three 3.8 cm (1.5 in.) diameter steel bolts that were added to equalize the longitudinal deflections in the superstructure arising from creep and temperature effects. This section was different from the rest of the bridge in that it was supported by one column instead of two.

On February 9, 1971, at 6:01 a.m. an earthquake assessed at Richter magnitude 6.6 occurred in the mountains behind Sylmar. The interchange suffered horizontal accelerations that were estimated as high as 0.6g. The 10 to 15 seconds of strong motion caused the superstructure of the 117 m (384 ft) section of the overpass to jump out of the shear key seats and induced the column and bridge deck to act as an inverted pendulum. The capacity of the column was found inadequate and it failed in bending at the base.

It was generally agreed that the overpass was of superior construction and did not fail as a result of any defects in workmanship or construction techniques. Prestressing elements survived the earthquake loading well and were intact in the debris.

As a result of this experience, significant changes in bridge design criteria were made including very large increases in beam seat sizes to allow for much greater longitudinal and lateral horizontal movements, the requirement for placement of hinges so that there are at least two columns between adjacent hinges along the bridge, the incorporation of spiral reinforcement to confine the longitudinal steel within the columns, the elimination of lap slices at the base of the columns, the reduction of skews in overpass structures, the increase in the amount of reinforcement at the column / deck connection to provide greater resistance to punching shear, and the elimination of the use of rocker type bearings.

On January 17, 1994, the Richter Magnitude 6.4 Northridge earthquake again caused failure of portions of this interchange (Figure 4-1). On this second occasion, some of the most severe damage occurred to sections that had been repaired following the 1971 earthquake and in other instances spans that had been under construction in 1971 failed this time. The fact that some spans were supported on columns of greatly dissimilar heights was thought to have contributed to the failures. The shorter columns, being much stiffer, were considered to have attracted disproportionately large shear forces resulting in their being overloaded with an inevitable subsequent domino effect. Also inadequate seat lengths at the ends of several spans contributed to collapse. Apparently the interchange had been scheduled for a seismic upgrade but the 1994 earthquake occurred before this had been started.

Lessons Learned

The failure of the Interstate 5/14 interchange in 1971 represented a turning point in seismic design of freeway bridges and prompted a radical change in the seismic design provisions for such structures. However these changes were not applied to the I-5/14 interchange itself. The failure in 1994 reemphasized the dangers of procrastination in undertaking seismic retrofitting once the need for such action has been established.

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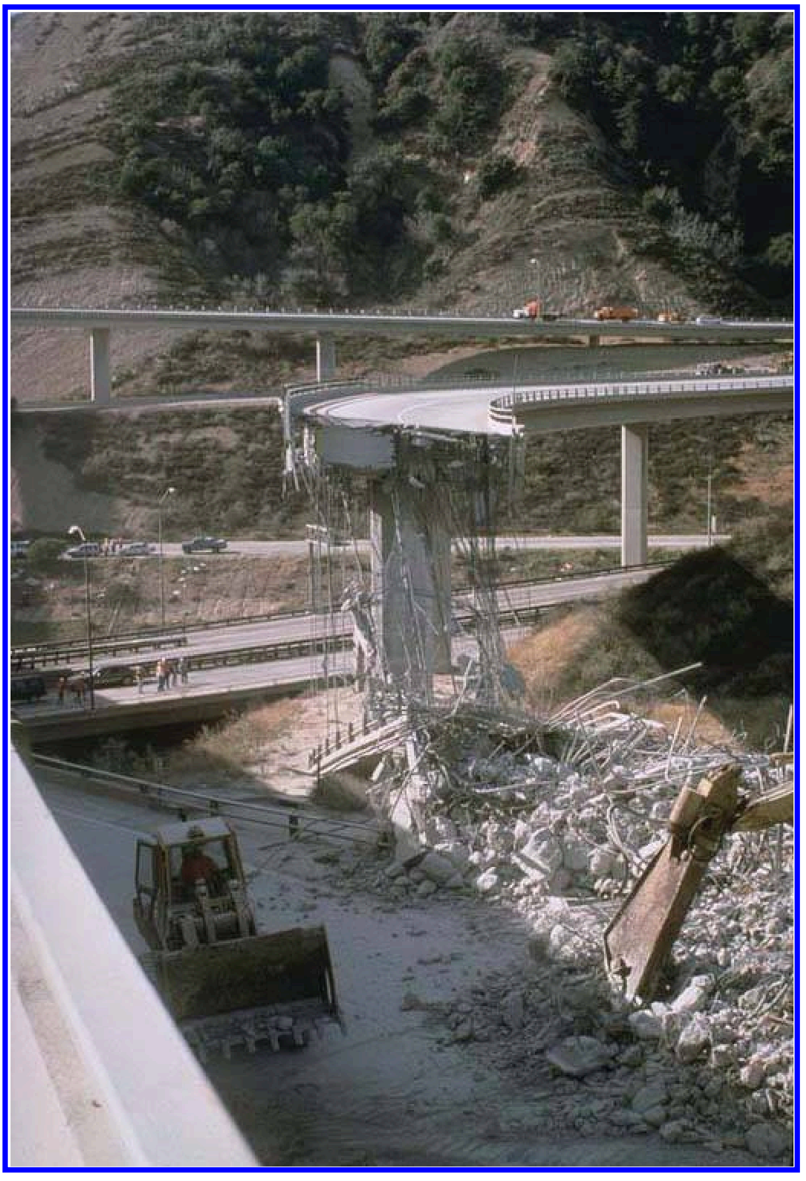

Figure 4-1. Antelope Valley – Interstate 5 Freeway Interchange Failure – 1994.
Source: J. Dewey, USGS.