FRACTURE IN STEEL BRIDGE PIERS DUE TO EARTHQUAKES

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Abstract: The great Hanshin Earthquake o January 17, 1995 caused serious damage to various civil structures never before experienced. A particular emphasis is placed on the fracture in steel bridge piers, and the results of studies up to date are reviewed in this paper.

1. INTRODUCTION

During the Great Hanshin Earthquake of January 17, 1995, steel bridge structures suffered damage of various kinds never before experienced. They were collapse, buckling, fracture and cracking. All of these accidents were far more severe than contemplated in the conventional seismic design method and might essentially invalidate historical seismic design approaches and code provisions.

Steel bridge piers have been adopted to support elevated highways and over pass bridges. Particularly, the piers of expressway in urban areas are in the median strip zones or sidewalk zones of streets because these are placed on the existing streets and piers (Figure 1). There are more than 5000 piers including 2000 piers in Tokyo Metropolitan Expressway and some of these are decrepit. Fracture accidents in these structures during earthquakes may cause not only the service failure of expressways supported by these piers but also a muddle on streets underneath the expressway.

Figure 1 Existing Steel Bridge Piers

2. COLLAPSE

2.1 Cases of Steel Bridge Pier Shafts

These were the steel bridge pier shafts at Iwaya Viaduct (Figure 2) on the national highway No.43 and Tateishi Viaduct on the Hanshin Expressway. According to the report of the investigation committee, the damage mechanism of the pier shaft at Iwaya Viaduct is “estimated to have been due to the occurrence of local buckling deformation of panels between diaphragms including splice plates, as deformation grew larger so that corner welds at this parts began to be torn, and load bearing capacity for supporting vertical force became lost and the collapse occurred directly below in the end.” This means that the cause was breaking of vertical stiffeners at field-joints with high strength bolts, but according to normal thinking in design, vertical stiffeners need not made continuous at this detail. However, the sudden change in rigidity due to the stiffeners being broken, and the stress concentrations at stiffener ends may be said to be cases showing that they can be causes of failure.

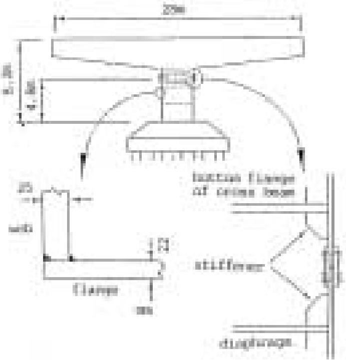


Figure 2 Collapse of Iwaya Viaduct

Corner welds are so-called stitch welds which are for the purpose of assembling members but do not expect loads to be transmitted. Therefore, these corner welds are composed of partial penetration welds and internal fillet welds except in the region of beam-to-column connections. It is conceivable that if the joints had been strong, it could not have led to such collapse. By making these corner welds as full penetration welds, it is possible for the strength to be increased more than the strength with the partial penetration welds and fillet welds detail actually used here. However, what is of importance is that in failure of corner welds this time either the insufficiency of weld size or the root faces as incomplete penetration zone remaining due to partial penetration was involved as cracks.

2.2 Large Size Column Tests (PWRI et-al, 1999)

Figure 3 shows the some of specimens in the cooperative study under the Public Works Research Institute. In order to improve the seismic performance of steel bridge piers and develop retrofitting methods for existing structures, 99 large size column type specimens were tested in total under incremental repeating loads and hybrid response loads.

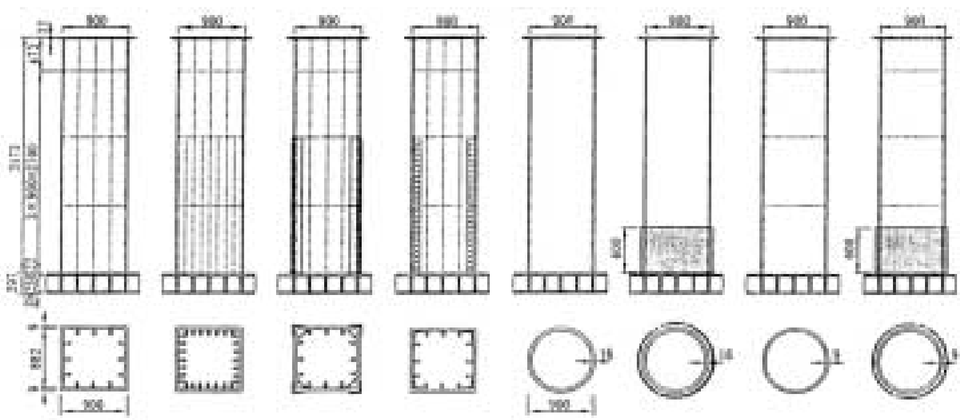


Figure 3 Specimens of Steel Bridge Piers

The followings are the objectives of this series of test specimens.

(a) Piers with rectangular columns

*add corner plates at the four corners of the column by welds or high strength bolts

*fill concrete inside of the column as a retrofitting work.

*lower the width-thickness ratio of stiffened plates, practically, increasing the number of stiffeners on flange plates and web plates of column.

(b) Piers with cylindrical pier shafts

*fill concrete inside of the column shaft as a retrofitting work

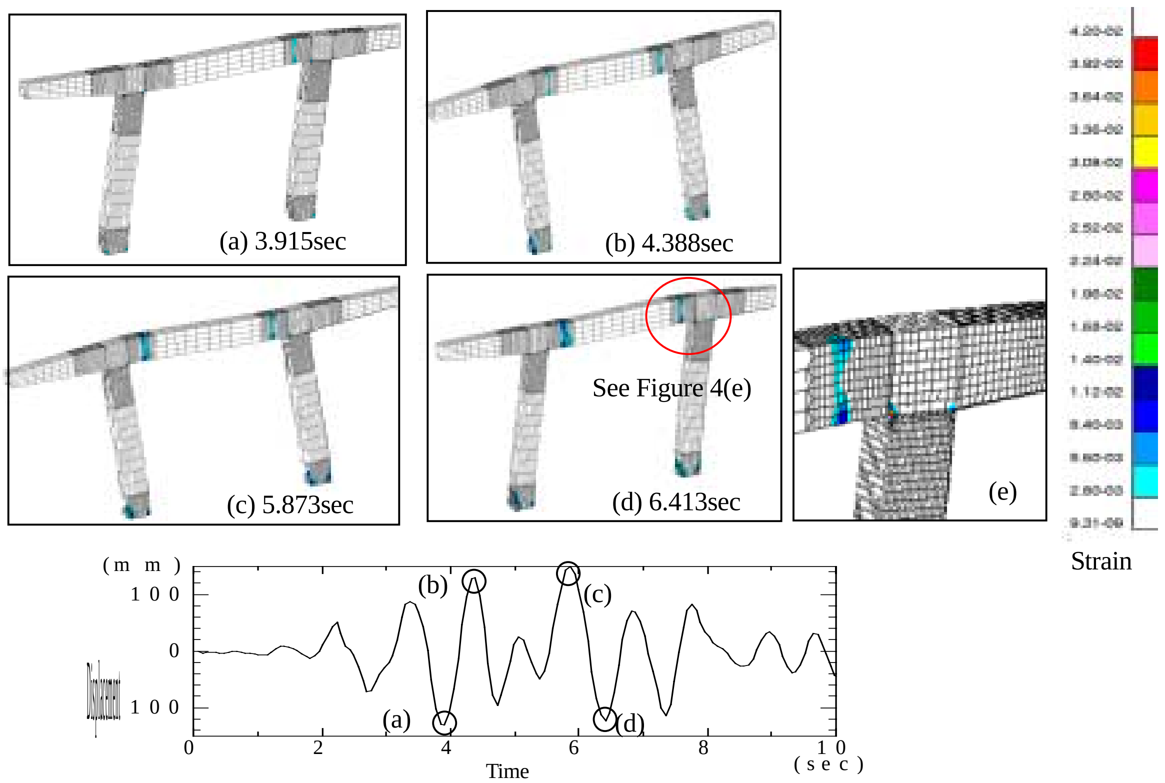
*cover the column shaft by a steel plate with narrow space

Based on these test results, the design specifications of highway bridges were revised (Japan Road Association, 1998). In order to insure adequate deformation capacity, to avoid collapse as brittle fracture mode including tearing of corner welds, and to satisfy the limitation of remaining deflection, provisions of structural details have been established. As the retrofitting works, the concrete fill method and corner plate addition method were adopted and applied to many actual piers. The result of dynamic response analysis is used to decide the necessity of retrofit and suitable retrofit method.

2.3 Seismic Performance as Rigid Frame of Thin Walled Members

In order to clarify the dynamic behaviors of steel bridge bents during earthquakes, the dynamic elasto-plastic FEM analysis was conducted. Some of actual steel bridge bents with general dimensions were chosen, and were modeled with shell elements for all the structural details including ribs and diaphragms. The measured acceleration data in the Kobe Great Earthquake were used in the analyses.

Figure 4 shows the representative result of the FEM analyses. In Figure 4, the time history of the displacement at the center of the beam upper flange was shown representatively, and the change of the deformations and the distributions of plastic equivalent strain were presented. As seen in the Figure 4, at two portions in the beam close to the beam-to-column connections, large plastic strain due to local buckling was occurred locally. Actually, the potions are just at the locations where the connections are there and the thickness of the plates is drastically changed. Especially, large plastic strain occurs at the corner joints of the beam. In contract, in the beam-to-column connections, no buckling occurs because of the large plate thickness due to the consideration in the elastic design of the local stress concentration by the shear-lag phenomenon. However, it should be noted that at the corner of the beam-to-column connections, large plastic strain occurs locally as shown in Figure 4(e) and that the local strain concentration can be one of the main causes of brittle fracture during earthquakes.



Time History of the Horizontal Displacement at the Center of the Beam

Figure 4 Dynamic FEM Analysis Results of A Steel Bridge Bent（JR Takatori Wave）

3. BRITTLE FRACTURE

3.1 Cracking in the Connection Corner of Steel Rigid Frame Bridge Pier (K. Okashita, C. Miki et-al, 1998)

One of the important events of the Great Hanshin Earthquake was the occurrences of brittle fractures in steel bridge structures. A brittle fracture accident during the earthquake occurred in one of the steel rigid frame bridge piers called P75 in the Harbor Highway (Figure 5). Cracks started from the lower corner of the beam and the column shaft, and ran through almost the entire cross section of shafts. The Chevron pattern on the failure surface indicated that the cracks started at the west-north corner of shafts and propagated into both the north flange and the west web plate.

Detailed observations at the crack originated portion revealed that the cracks started from the unwelded-zone at the corner of column-beam connection, so-called delta-zone (see **3.3**). There were two origins of crack, one was the toe of fillet weld on the north flange which was the origin of cracks propagated into the flange plate and another was just inside of fillet weld end on the wed plate which was the origin of cracks in the wed plate. The observations of fractographs indicated dimple patterns were remained in these areas and the sizes of these were about 0.7mm and 0.4mm. Outsides of these areas are typical brittle fracture surfaces.

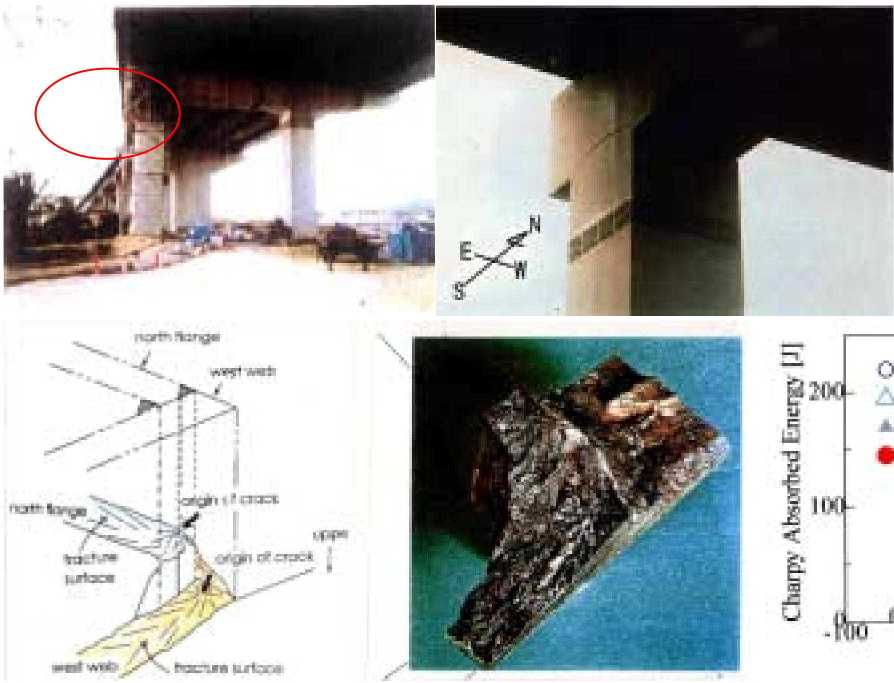


Figure 5 Brittle Fracture in P75 Rigid Frame Pier

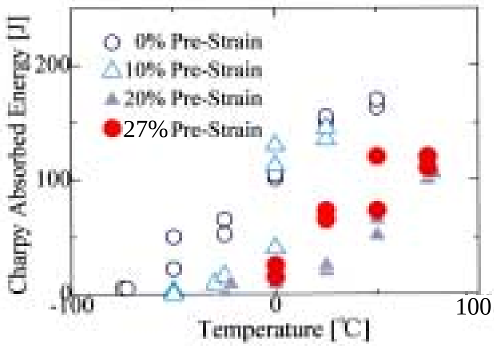


Figure 6 Results of Charpy Impact Tests

3.2 Fracture Toughness Properties of the Steel Used in Damaged Pier

In the vicinity of the corner, paint films were removed and a slight out-of-plane deformation due to local buckling was observed. The local buckling occurred at the welded joint where the plane thickness changed from original 19mm to about 17.4mm. The results of hardness tests indicated that plastic strain of about 20% to 30% occurred in these plates during the earthquake.

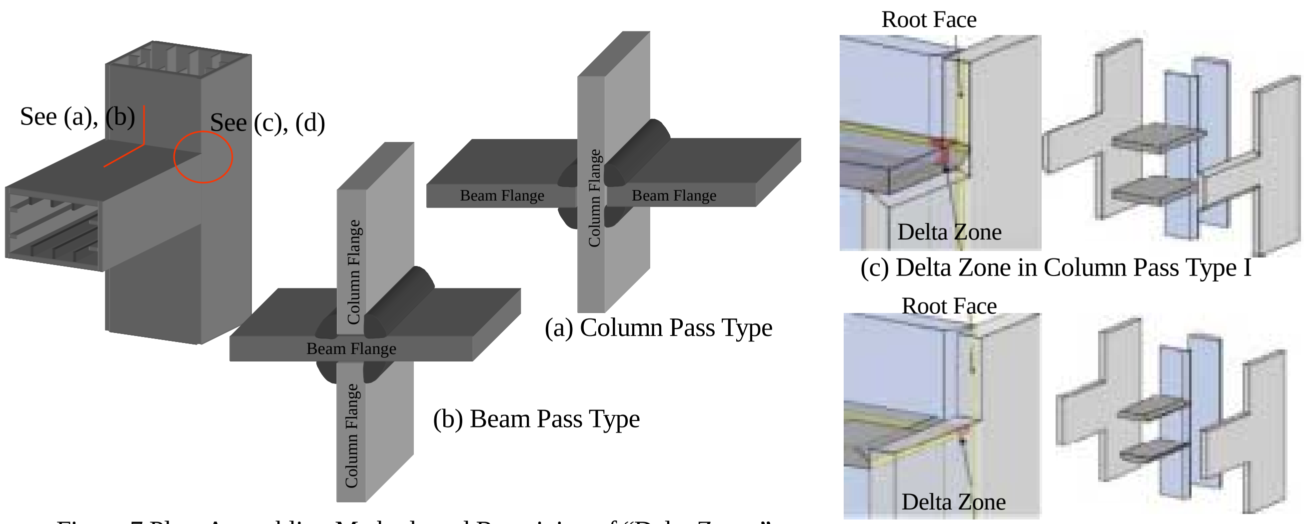
The used steels of this structure are SMA41 (SMA400). The tensile properties and the chemical composition satisfy the JIS requirements. Figure 6 shows the results of Charpy impact tests. Specimens were from the south flange plate of the shafts. In order to examine the deterioration of notch toughness due to plastic strain experience, specimens with pre-strain of 10%, 20% and 27% were tested beside as cut-out specimens. These test results indicate that notch toughness decrease significantly with increasing the experience of plastic strain.

Some more detailed investigations of the cause are necessary, but we can say that this brittle fracture was the result of large plastic deformations at the corner due to shear lag during the earthquake and the deterioration of fracture toughness due to plastic strain. The large unwelded zone "delta zone" existed at the high stress region and the small weld defects acted the origin of brittle cracks.

3.3 Existence of "Delta Zone" at the Corner of Bridge Bent (C. Miki et-al, 2003)

The steel bridge rigid frame structures consist of box section or cylindrical column shafts and box section beams, all of these members are welded closed sections. Figure 7 shows the details of plate assembling of beams and columns. The structural details are roughly classified into two types, one is column pass type and another is beam pass type (Figure 7(a), (b)). However, plate assembling system are more various and complicated.

Detailed investigations of fatigue damaged piers in Tokyo Metropolitan Expressway revealed that un-welded cavities and metal-touch zones existed at the corner of beam to column connections. In most cases, these are kind of inherent defects coming from plate assembling system, poor accessability and improvident welding sequences. Figure 7(c) and (d) show the remaining of delta zones in two different column pass type plate assembling methods We call these inherent defects as "delta zones". The delta zone is the dominant cause of fatigue cracks in many steel bridge piers and also may lead to brittle fractures in the case of suffering strong earthquakes. The combination of existences of delta zones and fatigue cracks more increases the possibility of the occurrence of brittle fracture.



(a) Column Pass Type
(b) Beam Pass Type
(c) Delta Zone in Column Pass Type I
(d) Delta Zone in Column Pass Type II

Figure 7 Plate Assembling Methods and Remaining of "Delta Zones"

4. LANTERN BUCKLING AND CRACKING

4.1 Cases in Cylindrical Steel Pier Shafts

In cylindrical steel bridge pier shafts on the Kobe route of Hanshin Expressway, there were cases where plastic deformation occurred and the wall of pier shafts bulged outward, so-called "lantern buckling"(Figure 8). The sections where lantern buckling occurred were close to the welded joints where the plate thickness was changed. Once these buckling incidents occurred, the load bearing capacity for supporting vertical forces became lost significantly.

Brittle fractures also occurred along the outward plastic deformation in two consecutive pier shafts. One of the pier shafts had exhibited large plastic deformation outward but the plastic deformation was not so large in another pier shafts. The crack seemed to start from the inside of shaft wall.

4.2 Deterioration of Fracture Toughness due to Lantern Buckling

Figure 9 shows the plate cut out from one pier which was suffered lantern buckling. Various material tests were performed on specimens made from this plate. Figure 10 shows estimated plastic strain in the plates from buckled zone and unbuckled zone. The plastic strain on the inner surface of column wall is over 20%. This plastic strain caused deterioration of toughness of the plate, called embrittlement, as shown in Figure 11. Embrittlement due to plastic strain, particularly compressive strain becomes new problem for us to prevent the fatal condition of bridge piers.

Figure 8 An Example of Damaged Circular Pier

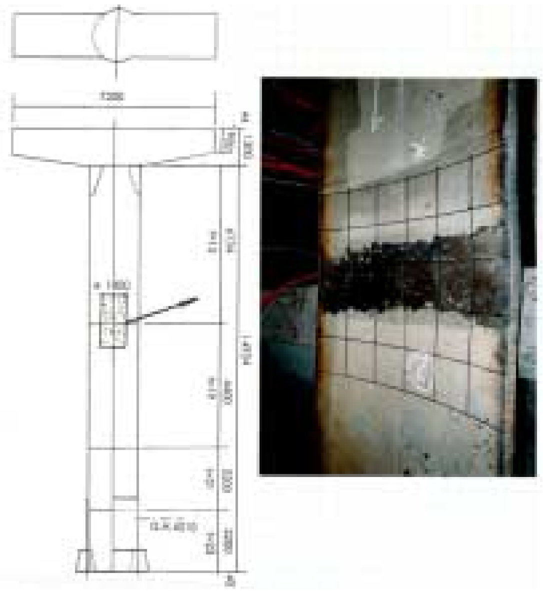


Figure 8 The Steel Plate Cut from the Damaged Pier

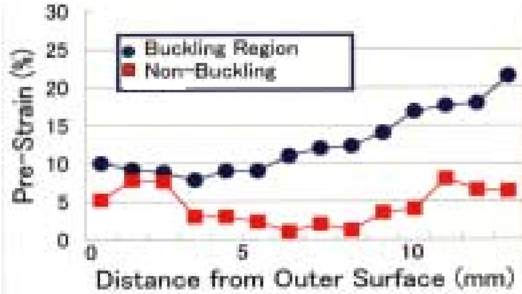


Figure 10 The plastic strain distribution in the plate thickness

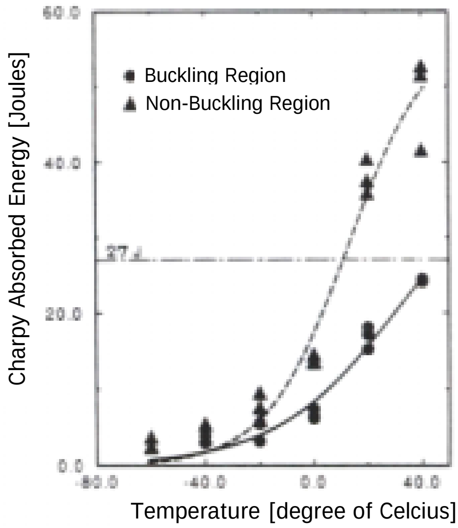
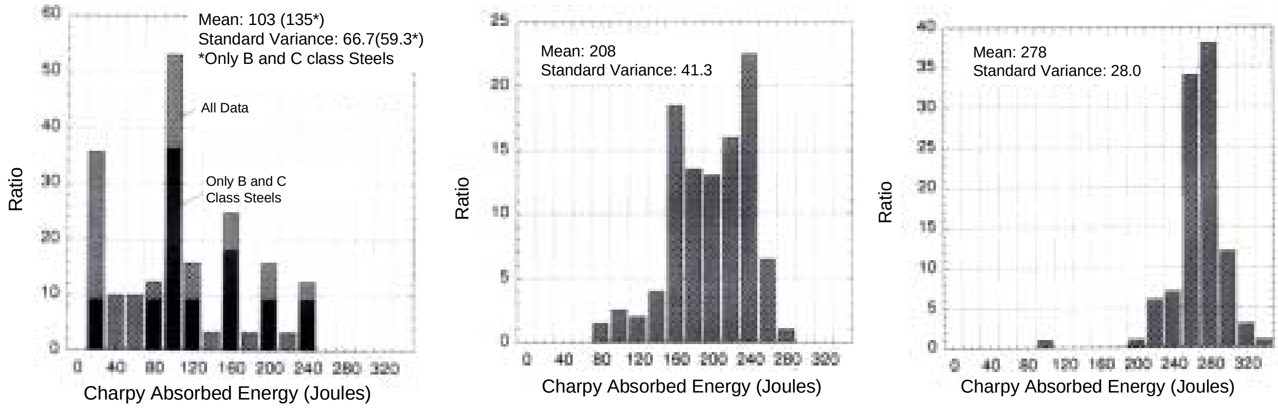


Figure 11 The Charpy impact test results of the steel plate

5. HISTORICAL REVIEW OF STRUCTURAL STEELS

JIS structural steels SM41 (SM400) and SM58 (SM570) have been commonly used for steel bridge piers from the middle of 1960s. Tensile properties of each of the steels have been the same level. However, various innovative technologies in steel making process including TMCP, continuous casting and removal of gases have been improving the performance of steels including notch toughness and weldability.



(a) JIS Structural Steels in 1970s (b) JIS-SM490YB in 1990s (c) JIS-SM570Q in 1990s

Figure 12 Distribution of Charpy Absorbed Energy of Steels Produced in 1970s and in 1990s

Figure 12 shows the results of Charpy impact tests in 1970s and 1990s. The notch toughness property of steels was significantly improved (K. Honma, C. Miki et-al, 1997). We also have to recognize that the fracture toughness of old steels is low in some cases.

Figure 13 shows the transition of capability for removing sulfur (S) from steels. All of steel making factories of the 5 major Japanese steel companies started to produce low S content steels since the middle of 1970s and the content of S in the recent steels is around 0.005%. However, the content of S in steels which were produced before the middle of 1970s might be over 0.01% .The high content of S leads to poor weldability and the occurrence of lamellar tearing in welded joints. In order to prevent the occurrence of lamellar tearing (Figure 14), the reduction of area in the plate thickness direction tensile tests (RAZ) have to be greater than 35%, and the content of S has to be less than 0.005%. That is to say, there is high risk of lamellar tearing in piers which were constructed before 1975. There are about 2000 steel bridge piers only in the Tokyo Metropolitan Expressway System and a half of them were constructed before 1975. We have to pay careful attention to the prevention of lamellar tearing in the retrofitting works against fatigue and earthquakes.

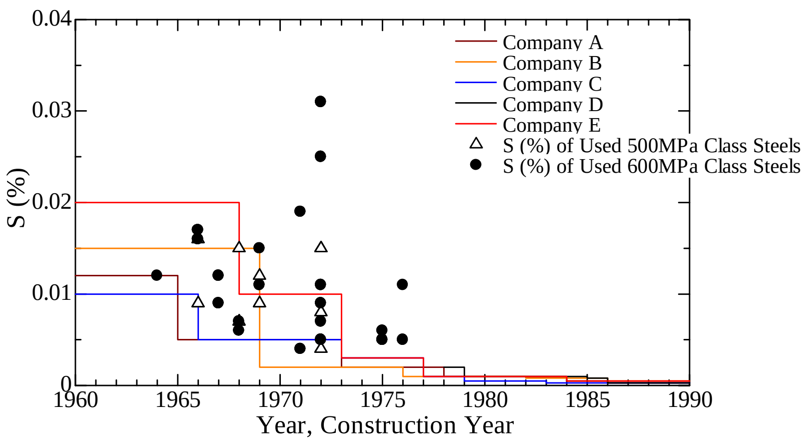
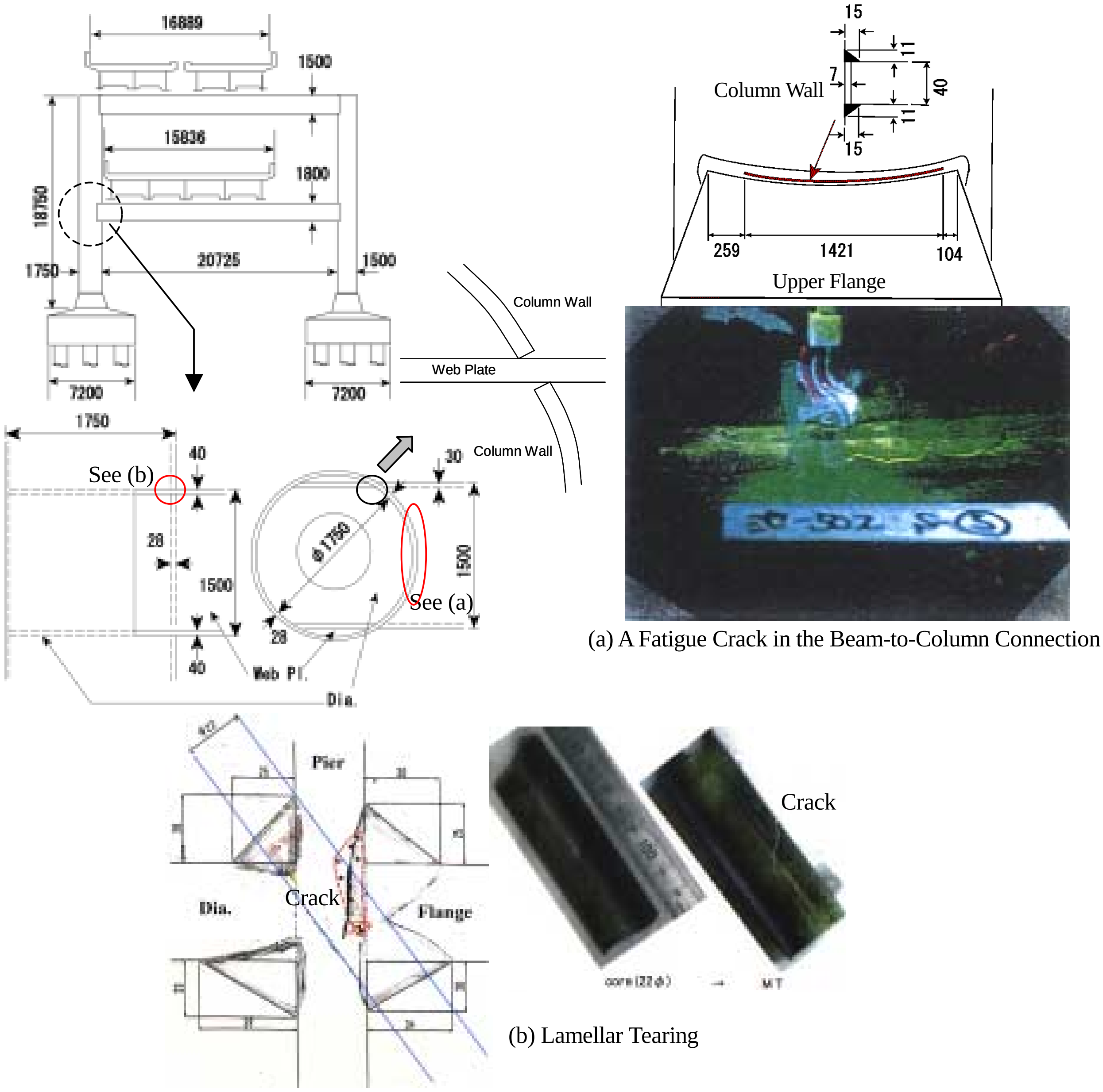


Figure 13 History of Sulfer Inclusion in Steels

4. CLOSING REMARKS

Since the Great Hanshin Earthquake, retrofitting of existing bridge structures has been recognized as urgent issues beside the improvements in the design of new structures. The ascertainment of the causes of experienced fracture accidents is essential to establish appropriate retrofit measures.



(a) A Fatigue Crack in the Beam-to-Column Connection

(b) Lamellar Tearing

Figure 14 The Fatigue Crack and Lamellar Tearing during Retrofitting Works by Welding in a Steel Bridge Pier

The primary reason of damage is very strong ground motions never before experienced. Apart from this, many factors contributed to the damage such as material properties, structural systems, details, and qualities of structural elements. It is also important to evaluate structural responses due to earthquakes such as forces, displacements, stresses, and strains. It can be said that the cause has been ascertained only when the results of examinations from the external disturbance side and the structural resistance side have come together without contradiction.

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