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The collapse of the Struve Slough Bridge during the 1989 Loma Prieta earthquake in different seismic code perspectives

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ABSTRACT: The collapse of the bridges on California Highway 1 near Watsonville, known as the Struve Slough Bridge during the 1989 Loma Prieta earthquake in Northern California is described. The Struve Slough Bridge consisted of two parallel structures erected in 1964. This paper describes both the structural and the geotechnical aspects of the damage, and presents a comparison of the American and the Canadian codes. Reference is made to the American seismic design code practice at the time of the bridge design and construction.

1 INTRODUCTION

On October 17, 1989, 17:04 Pacific Daylight Time, an earthquake of Richter magnitude $M_L = 7.0$ shook the large area between Santa Cruz to San Francisco Bay. It originated from the San Andreas fault. It had a major effect on transportation routes affecting 1,500 bridges in five counties. Of these structures, more than 80 suffered minor damages, 10 required temporary support and another 10 were closed due to collapse or major structural damages (Loma Prieta Earthquake Reconnaissance Report, 1990). One of them was the double-bridge viaduct called the Struve Slough Bridge near the city of Watsonville situated about 12 km from the epicenter. This area experienced high intensity shaking with horizontal acceleration reaching 0.39 g and the vertical on 0.66 g. Duration of shaking was about 16 seconds.

Shortly after the earthquake, the Institute for Research in Construction, National Research Council Canada (IRC/NRCC) organized a team of engineers to visit the San Francisco Bay Area to evaluate earthquake damages and overall impact. It covered various types of structures including transportation structures (Jablonski et al., 1990, Lau et al., 1991). Performance of bridges during the Loma Prieta earthquake has been presented in Loma Prieta Earthquake Reconnaissance Report, EERI, (1990) and by Mitchell et al. (1991). A discussion of the damaged bridge structures is also included in the report by Housner et al. (1990). This paper presents the geotechnical and structural aspects of the damage to the Struve Slough Bridge during the 1989 Loma Prieta earthquake.

2 DESCRIPTION OF UNDAMAGED STRUCTURES

The Struve Slough Bridge was a part of Highway 1 between Watsonville and Santa Cruz. It consists of two separate parallel structures erected in 1964. One structure carried northbound and the other southbound traffic. The southbound bridge had 22 spans, each 10.36 m (34 ft.) wide and 11.28 m (37 ft.) long. The northbound bridge had 21 spans of similar dimensions. The total length of both structures was about 243.84 m (800 ft.). The deck of each bridge was constructed as a four-span continuous reinforced concrete slab, 19 cm (7-1/2 in.) thick supported by five beams (28 x 94 cm or 11 in. x 2 ft. 9 in.). These beams were supported by 22 bents oriented at an angle of 30° 30' to the longitudinal axis of the bridge. The decks had three expansion joints. In 1984 cable restrainers were installed at each expansion joint. At the two ends of the bridge were monolithic diaphragm abutments. Each bent had four piles, which were driven to full length up to 27.43 m (90 ft.). Each pile was extended by a reinforced concrete column to support the transverse cap beam. The columns were lightly confined with No. 3 wire at about 12 in. pitch (30.48 cm).

Existing borehole log indicates that the subsoil profiles vary along the length of the bridges. At the south abutment, a 42 ft. (12.8 m) thick layer of compact medium to fine silty sand overlies a stiff blue-grey silty clay. The estimated liquefaction strength of the silty sand is close to the dynamic shear stress induced during the earthquake. At the middle section of the bridges, the subsoil is composed of a thick (up to 64 ft. or 19.5 m) layer of very soft grey clay peat with occasional pockets of fine sand. This

is followed by a 14 ft. (4.3 m) thick layer of compact to dense sand that is underlain by a very stiff grey silt to silty clay. The subsoil at the north abutment appears to be similar to that of the south abutment except that the thicknesses of different soil layers are different. Figure 1 shows the plan, elevation, typical bent of the bridge and typical soil layers for its vicinity.

The bridge described above was designed most probably according to the 1961 AASHTO provisions which included the provisional seismic requirements first mentioned in 1958. The 1961 AASHTO provisions were modeled on the 1943 Caltrans seismic design procedures.

In the 1943 Caltrans procedures, a seismic force was calculated as F = CW, where C = 0.06 for structures founded on pile foundations and W = total dead load of the structure. More detailed seismic requirements were introduced by Caltrans in 1965 and later in 1971 after the San Fernando earthquake (Housner et al., 1990).

3 DESCRIPTION OF DAMAGED STRUCTURES

The closest strong motion seismograph station was located in the city of Watsonville about 3 km east of the bridges. The station measured the following ground acceleration: 0.28 g (45°), 0.39 g (135°) and 0.66 g (up) (Shakal et al.). The city of Watsonville and surrounding areas experienced high intensity of ground shaking. As a result of it and associated geotechnical phenomena, the two Struve Slough bridges were severely damaged and one of them collapsed.

Cable restrainers installed in 1984 performed as intended holding the separate decks together during the earthquake. Thus, T-decks did not disintegrate in both northbound and southbound structures. However, the central portion of the deck of the southbound bridge collapsed completely due to loss of supports and columns punching through the deck.

Figure 2 presents a general view from the northern abutment. The collapsed southbound structure is seen with displaced columns. A significant amount of movement can also be observed at the northbound structure. The top of the deck with protruding columns is shown in Figures 3 and 4. Those columns of 381 mm (15 in.) diameter (see Figures 2 to 7) were extensions of piles into transverse cap beams. Most columns suffered severe cracking at the tops, buckling of the longitudinal reinforcement and fracture of vertical confining wires. Some of the columns were sheared off at the start of the transverse cap beams. Seven spans of the northbound structure collapsed and dropped almost 1.52 m (5 ft.) onto the damaged columns. Ten spans of the southbound bridge dropped approximately 2.44 m to 3.05 m (8 ft. to 10 ft.) to the ground surface. A few columns sheared inside the transverse cap beams, were displaced by almost 30.48 cm to 60.95 cm (1 ft. to 2 ft.) and punched through the deck. Although the collapse was in the vertical direction, the southbound structure was displaced horizontally by about 60.96 cm (2 ft.).

The shaking of the earthquake induced settlements and lateral displacements of different degrees. At the approach fills, the settlement was estimated to be 7.62 cm (3 in.) No apparent distress, however, was observed in the vicinity or with the superstructure as a result of the settlement. Thus, it was unlikely that the subsoil liquefied during the earthquake. At the section where the bridges were damaged, there were large lateral movements up to 610 mm (2 ft.) and settlements up to 508 mm (20 in.). Such movements caused some piles to fail below the ground surface (Housner et al., 1990) and damaged the columns for the deck. The subsoils at this location are largely composed of a very soft clay and peat which normally do not suffer liquefaction failure during earthquakes. Hence the large movements were likely caused by the softness of the soil and the large amplified ground motion induced by the earthquake waves traveling through the soft soil.

4 SEISMIC CODE REQUIREMENTS

The model building code in the U.S. is the Uniform Building Code (UBC) published by the International Conference of Building Officials and is periodically updated with respect to latest seismic requirements. In the 1961 edition of the UBC, the basic equation for calculating the seismic force was the same as in the 1949 edition:

$$F = CW \tag{1}$$

where, W = total dead load of the structure, and C is given as follows:

C = 0.02 when the foundation rests on soil having a safe bearing value of 1 tonne/ft^2 (100 kPa) or more in Zone 1 (only three seismic zones were introduced).

C = 0.04 when the foundation rests on soil having a safe bearing value less than 1 tonne/ft^2 (100 kPa) or is supported on piles, in Zone 1.

$C = \frac{0.15}{N + 4.5} \times 4$ where N is the number of stories above the story under consideration, in the case of Zone 2.

For Zone 2, regions of intermediate seismicity, the values of C should be multiplied by 2. Zone 4 was

not introduced until the 1971 San Fernando earthquake.

The Caltrans (formerly the California State Highway Department) material requirements were similar to that given in the UBC. At the time of the design/construction of the Struve Slough Bridge, the 1943 Caltrans requirements were in place. They followed the same formulae as above, but the descriptions for the coefficient C were slightly different from those included in the UBC.

C = 0.02	for structures founded on spread footings with a bearing capacity exceeding 4 tonnes/ft^2 (400 kPa) or better.
C = 0.04	for structures founded on spread footings with a bearing capacity less than 4 tonnes/ft^2 (400 kPa).
C = 0.06	for structures founded on pile foundations.

The 1961 AASHTO (American Association of State Highway and Transportation Officials) edition essentially incorporated the 1943 Caltrans requirements (Housner et al., 1990).

The Struve Slough Bridge was constructed before acquiring the present knowledge in seismic loading and ductile behaviour of structures. At the time of the design, the horizontal seismic force was calculated as a percentage of the dead load in a quasi-static analysis. In the case of structures on pile foundation, this was taken as 6%. The local soil effects on seismic performance of structures were not recognized.

In Canada, at the time of the design of the Struve Slough Bridge the 1953 second edition of the National Building Code (NBC) was in use, in which the "horizontal force due to earthquake" was calculated similarly as in Eq. 1 as follows:

$$F = CW \tag{2}$$

where F = the horizontal force
W = the total dead load plus one-half the live load
C = a constant depending on the type of construction

In the case of poor soil which is described in Table 1 in Appendix H of the 1960 NBC edition, C = 0.04 when allowable bearing value of soil is 2000 lb/ft^2 (90 kPa) or less.

The 1960 NBC introduced similar formulae for C as included in the 1949 edition of the Uniform Building Code:

$$C = \frac{0.15}{N + 4.5} \tag{3}$$

where N = the number of storeys above the storey under consideration provided that for floors or horizontal bracings, N shall be only the number of storeys contributing loads.

After the 1971 San Fernando earthquake of magnitude 6.6, a number of studies were initiated, which led to improved versions of the AASHTO requirements in 1983. The Applied Technology Council (ATC) earlier published seismic design guidelines for bridges followed by retrofitting guidelines (ATC-6, 1981, 1983). The first step in the design process is a choice of the so-called seismic performance category (SPC). The SPC is a function of the zoning and the importance of the bridge structure. The SPC varies from "A" for all bridges in lower seismic zones to "D" for important structures in higher seismic zones. Also, the soil effects are taken into account as well as provisions in order to reduce the possibility of loss of support (see Mitchell et al., 1991). Thus, the 1971 San Fernando event prompted a radical change in the design philosophy including the need for detailed dynamic analyses for strategic bridge structures.

The Ontario Highway Bridge Design Code (OHBD) of 1983, uses the equivalent static load method as follows:

$$Q = C\,I\,F\,W \tag{4}$$

where F = 1.0 for structure where single columns or piers resist the horizontal loads;
F = 0.8 for structures where continuous frames resist horizontal loads applied along the frame
I = 1.3 for all bridges designed for post-disaster service and 1.0 for all other bridges
C = the response coefficient for various values of acceleration ratios (0.02 g + 0.012 g) given by Figures 2-4.8.2 (OHBD, 1983).

The acceleration ratios reflect the 1980 NBC seismic zones.

The 1988 CSA (Canadian Standard Association) standard for bridge design uses also an equivalent static force method for the design for most bridges (in zones less than 4). The 1988 CSA horizontal force is based on the 1985 NBC edition assuming that an estimated fundamental period of a bridge is less than 0.25 s. For irregular or important bridges, or for bridges in higher seismic zones than 4 and for poor soil conditions, full scale dynamic analysis is required (CSA, 1988).

The Struve Slough Bridge was placed on skewed foundations. In earlier codes little attention was given to the necessity of a 3-D dynamic analysis of such structures. Although the 1988 CSA standard supports a dynamic analysis for "important, complex, or highly skewed structures..." no guidelines are provided.

The primary cause of the observed damage of the bridge discussed was a massive lateral displacement of the foundation soil. It appears that there are insufficient provisions in any existing codes to address this problem. There is a need to properly recognize local soil effects in bridge design.

5 CONCLUSIONS

1. The collapse of the Struve Slough Bridge showed the typical deficiencies of the design of a single viaduct structure with skewed bents: light confinement of circular bent columns and inadequate connections of them to the cap beams of the decks.
2. The Struve Slough Bridge belongs to the low class of bridges (w.r.t. height) and most of the damage resulted from the excessive soil deformation.
3. The introduction of new seismic requirements for bridges after the 1971 San Fernando earthquake changed substantially the bridge design practice and sparked retrofitting which improved the seismic performance of bridges.

ACKNOWLEDGEMENT

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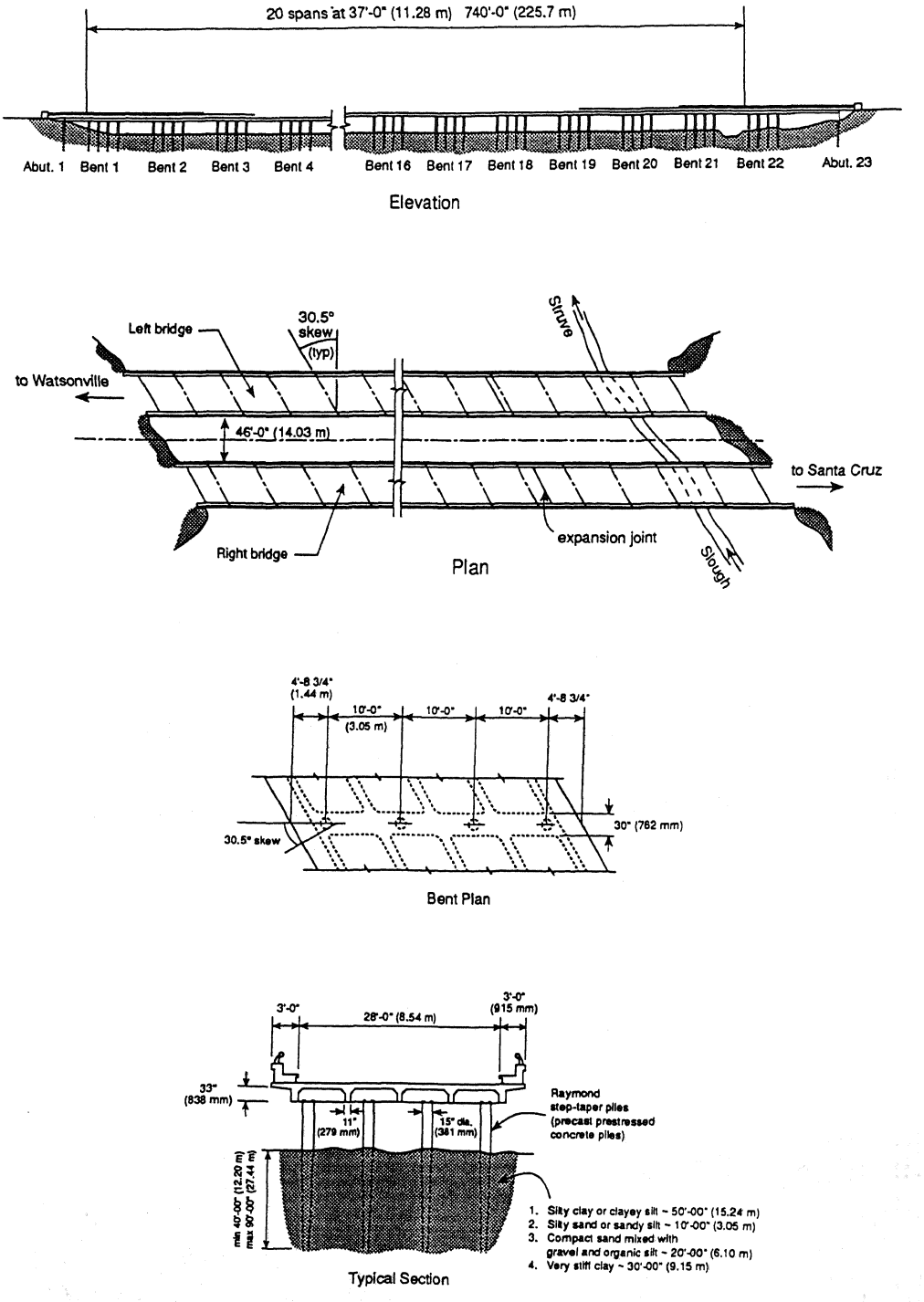

Figure 1. Elevation, plan view, bent plan and cross section of the Struve Slough Bridge (courtesy of CALTRANS)

Figure 2. General view of the damaged Struve Slough Bridge near Watsonville

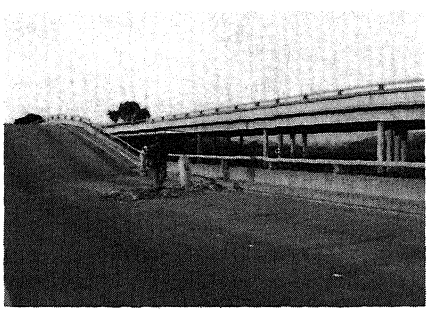

Figure 3. Columns punched through the southbound deck

Figure 4. A column remained in the vertical portion after piercing through the deck.

Figure 5. A side column of the bent, sheared off and displaced

Figure 6. Disfigured southbound structure

Figure 7. The central portion of the northbound structure severely damaged