3D4 Structural Analysis and Stability FTR

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Introduction

Steel box-girder bridge was a rapid developing new technology in the 1960s and was widely applied for the post-war reconstruction at the time, due to the advantages of high torsional rigidity, relative lightness, relative low cost of its closed section members.
However, four single-cell (no internal webs) steel box-girder bridge failures occurred during their erections from 1969 to 1971, resulting a total of 52 fatalities. They are:

- The Cleddau Bridge at Milford Haven, Wales (1970).
- The West Gate Bridge in Melbourne Australia (1970).
- The South Bridge (Rhine River) in Koblenz, West Germany (1971).
- And a bridge over the Danube in Vienna (1969).

Notably, no two of these collapses were alike. However, all 4 involved with instabilities of thin plates under compressive loads.
At the time of their designs and erections, the complicated behaviours of the slender components which take significant compressive loads (diaphragm, panels of bridge flanges, etc.) within a box girder bridge wasn't well understood. And buckling is therefore a key concern when designing steel/composite box girder bridges.

Origins of box girder bridge

Before (non-modular) box-girder bridge gained its popularity during the roadbuilding expansion in the post-war West. During the second world war, the most extensively used type of truss bridge is Bailey bridge due to its portability and ease in assemblies.

Figure 1. (Left) Doubled sections of Bailey bridging assembled by hand in WW2.
(Right) Transoms of a Bailey bridge (made of I beams) are subjected to vertical loading.

Post WW2, the use of Bailey bridges is dramatically diminished. As shown in the Figure 1. (right), vertical frames and transoms of a Bailey bridge (made of I beams) have their upper (compression) flanges free to displace laterally and rotate. And therefore, when an applied vertical load is large enough to cause both lateral displacement and twisting of a transom or chord, "lateral tortional buckling" occurs, which causes the Bailey bridge susceptible to buckling failure.

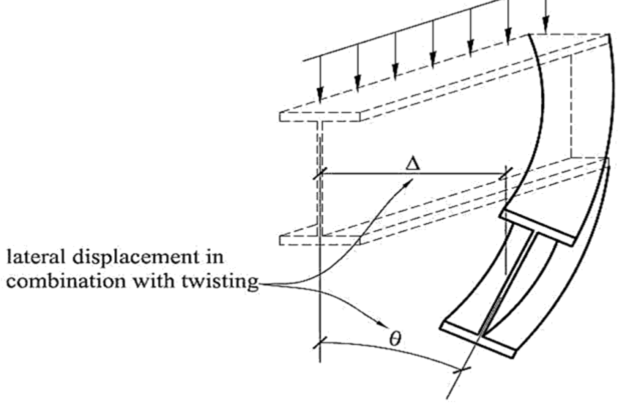

Figure 2. (Right) lateral tortional buckling of a I beam.
(Left) a collapsed Bailey bridge with its Bailey Panels and chord show buckling.

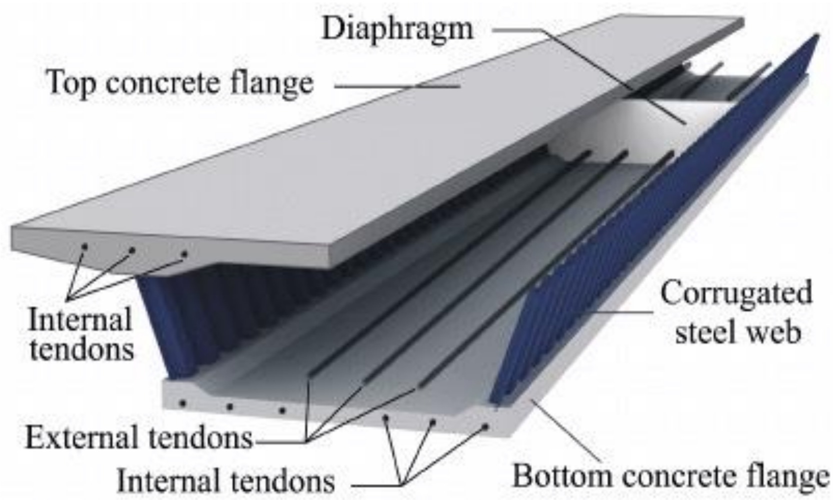

Figure 3. Terminologies (flange, web, diaphragm, rib, stiffener) we will be using for a typical box-girder bridge.

Cleddau (Milford Haven) Bridge (welded box girder, incremental cantilevered launch), Wales (1970)

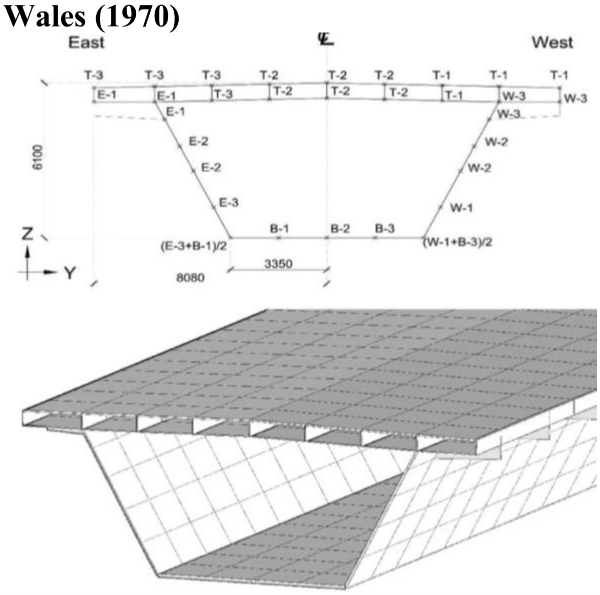

Figure 4. (Left) A sketch of the trapezoidal box-girder cross section of the Cleddau Bridge, and a perspective view of the cross-section in the numerical model.
(Right) Failure of the Cleddau Bridge (Milford Haven). Notice the bottom flange is buckled.

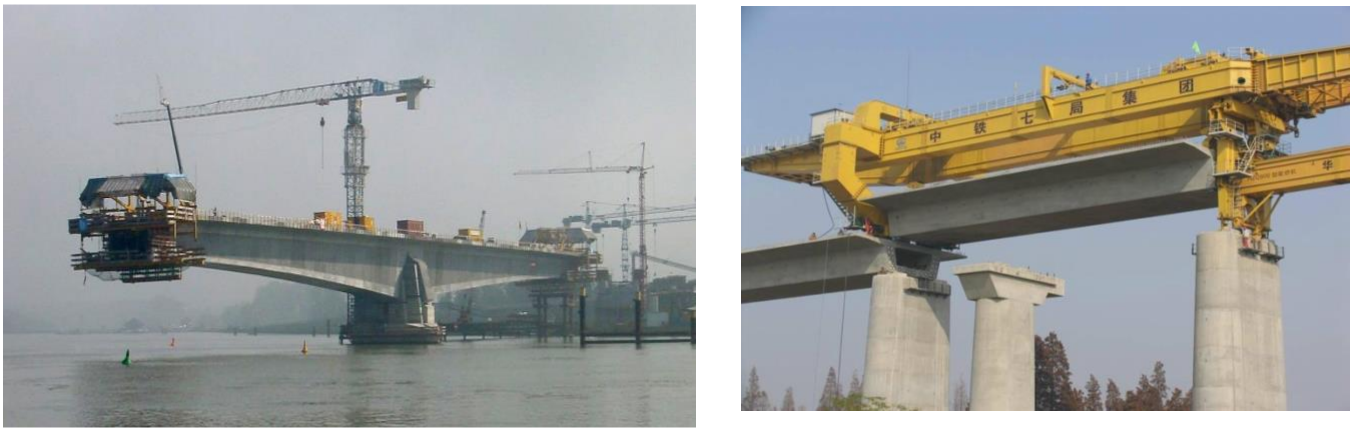

Figure 5. (Left) cantilever construction method for box-girder bridge. (used for the Cleddau Bridge)
(Right) incremental launch method for box-girder bridge. (used for the West Gate Bridge, Melbourne, Australia)

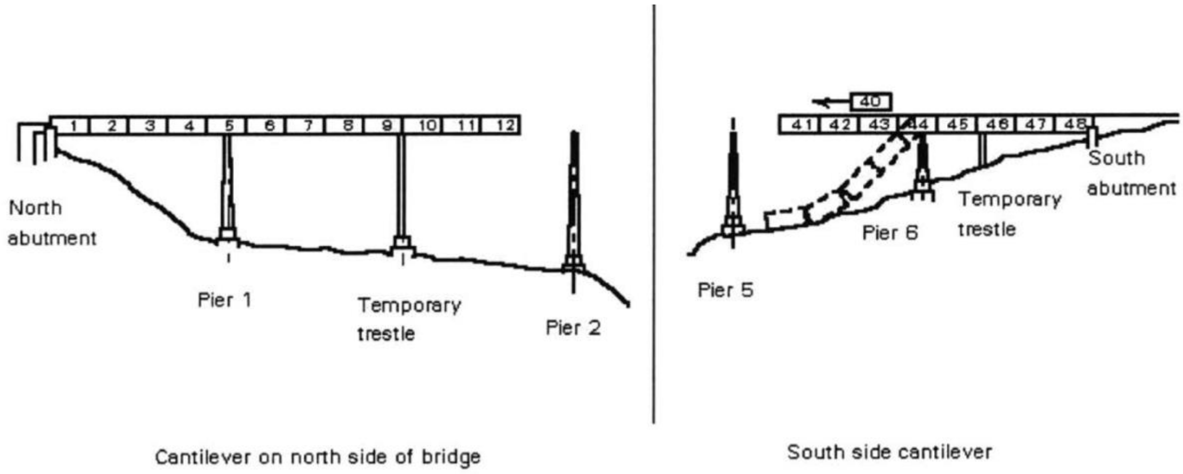

Figure 6. Schematic diagram of Cleddau Bridge construction and its failure.

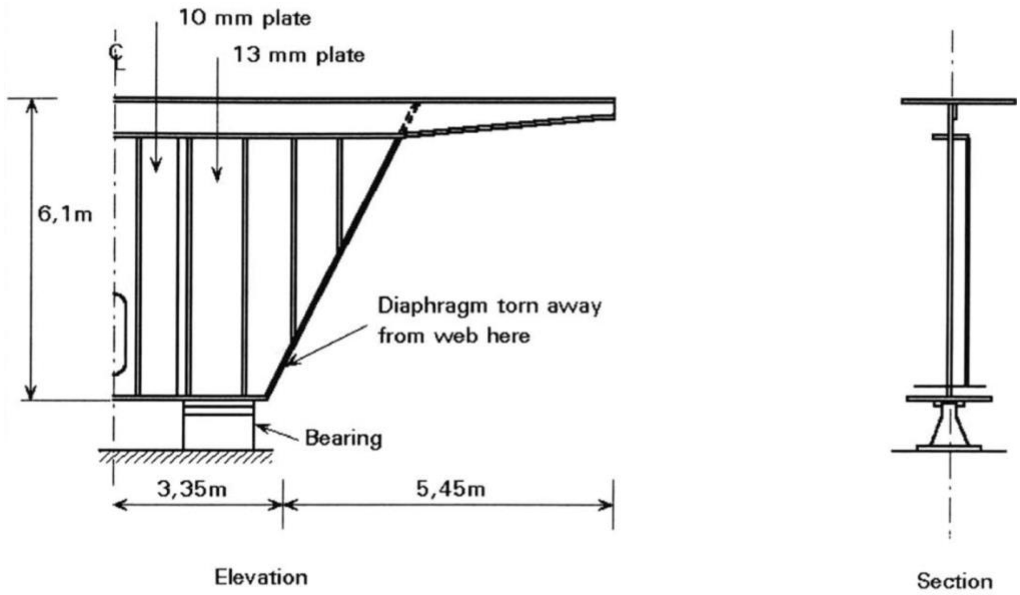

Figure 7. Drawings of the diaphragm over Pier 6.

Brief background

- The Cleddau bridge consists of welded steel trapezoidal box-girder superstructure as shown in Figure 4. (Left).
- As shown in Figure 5. (Left), Figure 6. The construction method adopted for the Cleddau Bridge was to cantilever out pre-assembled deck sections from each side of the river. The bending moment and stresses on the deck is in general much greater in the cantilevered state than after erection.
- The collapse occurred when the last deck section (40) between pier 5 & 6 was moving out along the cantilever.
- It was pointed out by the Merrison Committee of Inquiry that the buckling of the support diaphragm at the root of the erected cantilever initiated the failure, as shown in Figure 4. (Right). After the diaphragm was torn away from the bottom part of the web, bottom flange and the web were allowed to buckle. (shown in Figure.7)

Complex load on the diaphragm

Heavy loads are applied to the diaphragm from the web and flanges at its sides:

- Most apparently, a hogging bending moment and a large vertical shear force were applied to the diaphragm due to the weight of cantilevered section.
- Diffusion of the point load from the supporting bearings below, in addition to the out-of-plane bending effect due to bearing's eccentricity.
- The sloping/inclination of the webs results in additional horizontal compression on the diaphragm.

Result of the buckled diaphragm

According to the later Australian Royal Commission report, as the supporting diaphragm over Pier 6 buckled, the overall depth of the steel box girder was shortened, which caused the bottom flange more likely to buckle due to the shortening of the inter-flange distance. (i.e., as the moment was now carried by a reduced lever arm, greater compression was acted on the bottom flange until the weakened deck box can no longer sustain the bending moment, and the cantilevered section rotates about the now formed virtual hinge.)

There are following ***unforeseen errors*** in the construction of the Cleddau Bridge noted by the Merrison Committee of Inquiry and later studies:

- The buckled support diaphragm which initiated the failure is likely have been as much as $\frac{3}{4}$ inch out of flat, which makes it susceptible to local buckling.
- The bearings in between the pier 6 and the box-girder superstructure were out of line with diaphragm's neutral axis and could have induced bending moment.
- Some bolts which were used to connect longitudinal stiffeners to the bottom flange were tightly installed in their corresponding oversized holes, instead of the designed loose installation. And under load movement, the bolts could have torn off the longitudinal stiffeners from the bottom flange, causing more instabilities under compression.

Fourth Danube Bridge (Steel Plate box-girder bridge), Vienna (1969, no pictures available online)

Fourth Danube Bridge was collapsed in November 1969 due to buckling on the bottom steel flange of the bridge, which causes are considered to be:

- ***The differential temperature change*** in the bridge's deck/steel box sections due to the temperature drop during the night, is believed to be the principle which caused the bridge's web and bottom flange to be compressively stressed beyond their P_{cr}.
i.e., The top roadway deck was heated under hot sunshine, when the free ends of the bridge jointed during the day, the bridge was actually "extended", the overnight cooling caused tension to the bridge's top flange (road deck) and compression to the bridge's web and bottom flange.
  - ❖ Nowadays, BS153 and other codes for bridge designs call for "consideration of stresses on the structures caused by secondary effects (wind, (uniform and differential) temperature changes, supports settlement etc.)"
- ***Deformations*** caused by welding seams in the steel web, the bottom steel flange and its ribs, and the ***variations*** in the thickness of bottom flange made of steel plates, deviate from the assumption of perfect straight plane panels when calculating the theoretical buckling stress.