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Failure Study of a Skew Box Girder Bridge Model

Essai à la rupture d'un pont biais en béton armé

Bruchversuche an einer schiefwinkligen Betonkastenbrücke

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SUMMARY
Lessons from the loading to failure of a large scale skew, continuous two span, four cell reinforced concrete box girder bridge model are presented, with special reference to the effect of skewness and skew diaphragms on the response to ultimate loading. It is found that skewness can lead to smaller midspan moments and that by withstanding part of the longitudinal bending, skew center bent diaphragms favorably modify the structural behavior at the center support. The collapse load can be predicted with reasonable accuracy on the basis of postulated collapse mechanisms.

RESUME
Dans cette étude, on présente les résultats d'un programme de chargement à la rupture d'une maquette à grande échelle d'un pont en caisson à quatre cellules, biais et continu, construit en béton armé. Une attention particulière est attribuée à l'influence du biais et des diaphragmes biais sur le comportement structurel. On met en évidence que le biais peut contribuer à diminuer les moments fléchissant et, par sa participation à la résistance de flexion, le diaphragme biais d'appui central modifie favorablement le comportement. Avec les mécanismes de ruine postulés, la charge ultime est prédite avec une précision suffisante.

ZUSAMMENFASSUNG
Erkenntnisse aus einem Grossversuch über das Bruchverhalten einer zweifeldrigen, schiefwinkligen, vierzelligen Hohlkastenbrücke sind dargestellt, unter besonderer Berücksichtigung des Einflusses der Schiefwinkligkeit der Auflager und des Mittelstützenquerträgers auf das Tragverhalten. Es wird gezeigt, dass die Schiefwinkligkeit eine Herabsetzung der Feldmomente bewirkt, und dass schiefwinklige Stützenquerträger — durch ihre Mitwirkung in der Abtragung der Längskräfte — das Tragverhalten im Stützenbereich günstig beeinflussen. Die Bruchlast kann mit ausreichender Genauigkeit mit Hilfe der angegebenen Bruchmechanismen vorhergesagt werden.

1. INTRODUCTION

For certain standardized classes of structures, such as reinforced concrete box
girder bridges, testing to failure has been accepted as an economic approach for
the establishment of optimum design criteria. For this reason, large-scale, two-
span, straight, curved and skew box girder bridge models have been built, instru-
mented and tested both under working and ultimate loads [1,2,3]. These studies
were feasible as the large number of bridges needed in the California highway
system had produced a high degree of bridge standardization in structural design,
geometric characteristics and construction techniques.

2. DESCRIPTION OF 45° SKEW BOX GIRDER BRIDGE MODEL

Basic dimensions of the skew box girder bridge model including the designation of
transverse sections and longitudinal girders are presented in Fig. 1. The bridge
was a 1:2.82 scale model of a typical California 200 ft (61 m) long two-lane box
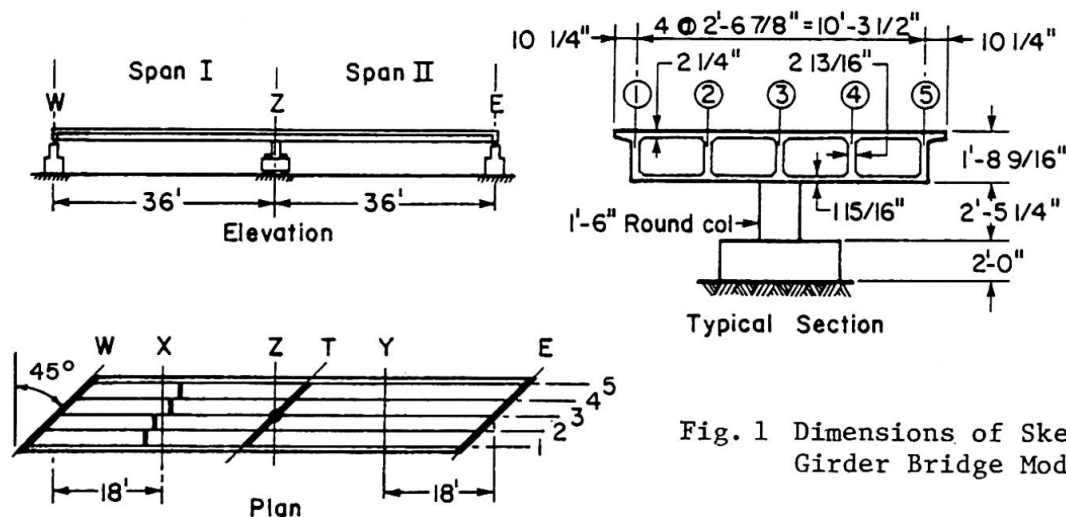girder bridge with two spans and a skew bent providing the center support. Span I

Fig. 1 Dimensions of Skew Box
Girder Bridge Model

had a staggered diaphragm at midspan to compare its structural effect as opposed
to no diaphragm in Span II. The chosen scale guaranteed true representation of
material behavior and was determined from the sizes of the reinforcing bars used
in prototype and model. The test structure was constructed in the same manner
as prototype structures in the field.

3. STRUCTURAL BEHAVIOR OF SKEW BRIDGE AT FAILURE

For the failure studies, each girder
was loaded simultaneously at midspan
sections X and Y. A typical load-
deflection graph for the exterior
girder at section Y (location 5Y) in
the undiaphragmed Span II is given in
Fig. 2 for the first two loading cy-
cles. According to strain measurements
in the tensile reinforcement local
yielding commenced at sections X and Y
at a total span load of 160 kips
(712 kN). Under increasing span load
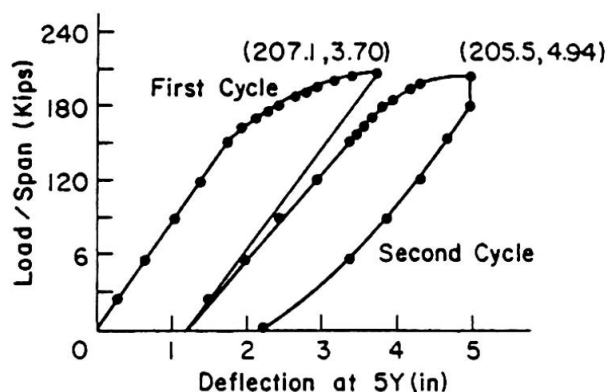the effects of shear, torsion and slab

Fig. 2 Experimental Load-Deflection
Diagram at 5Y for Ultimate
Loading Cycles

transverse bending became apparent, resulting, at a load of 176 kips (783 kN), in spalling of the concrete from outer girder 5 between midspan section Y and skew section C near the center bent (Fig. 5). Span II collapsed at a maximum load of 207 kips (921 kN) due to the formation of hinges at the midspan and near the center support, and further spalling of the bottom slab. The transverse bending effect resulted in higher deflections at 5Y than at 3Y or 1Y (Fig. 1).

In order to draw general lessons from the observed behavior, it is essential to evaluate the effects of bridge skewness and other structural parameters on the load carrying capacity of the skew box girder bridge.

4. EFFECT OF SKEWNESS ON STRUCTURAL BEHAVIOR

To assess the linear behavior of the skew bridge, it has been analyzed as an elastic homogeneous uncracked concrete structure using a specially developed finite element computer program called CELL [4]. The theoretical individual reactions under the end support diaphragms and the center bent column due to 20 kip (89 kN) loads placed over each of the five girders at sections X and Y are shown in Fig. 3a. The resultant reactions along the centerline of the bridge are shown in Fig. 3b.

(a) Individual Support Reactions

(b) Resultant Support Reactions and Moments Along Bridge Centerline

Fig. 3 Skew Box Girder Bridge Reactions for Span Loads of 100 kips

In general, the high individual reactions at the obtuse corners (Fig. 3a) reflect the tendency of the skew bridge to span a-cross the shortest distance between supports. Also, the resultant centerline reactions (Fig. 3b) indicate the presence of significant end moments and torques. The resultant end moment, torque and vertical reaction can be replaced by a single resultant vertical reaction with an eccentricity of 3.45 ft (1.05 m) from the centerline towards the obtuse corner. This indicates that skewness tends to move the resultant vertical reaction so as to shorten the effective span.
Under the given load the external moment at midspan section Y, as calculated from the reactions and shown in Fig. 4, was 519 kip-ft (704 kN-m). Similarly, the center bent support moment at the right section Z was calculated as -640 kip-ft (-868 kN-m). Under the same load a straight box girder bridge of identical cross-section and centerline dimensions would exhibit comparable moments of 562 kip-ft (762 kN-m) and -676 kip-ft (-917 kN-m), respectively. Hence, the introduction of skew end supports helps reduce the midspan and support moments. This reduction also exists under dead load alone.
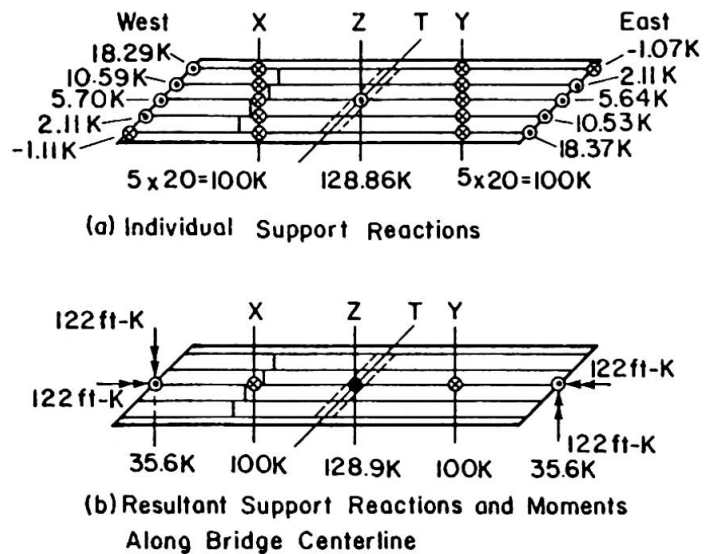
Also presented in Fig. 4 are the internal moments, which are derived from the internal longitudinal forces as calculated from the CELL program, through integration over the box girder cross-section. Comparing these results indicates that the negative internal moment in the range of the skew center bent diaphragm is significantly less than the moment derived from the external reactions, with a value of -545 kip-ft (-739 kN-m) versus -640 kip-ft (-868 kN-m) at section Z. This difference is due to the participation of the skew center bent diaphragm in withstanding the total longitudinal moment. Hence, the skewness of this diaphragm

has the substantial, beneficial effect that the bridge portion between sections
B" and C" (see Fig. 5) acts as a kind of broad support for the continuous box
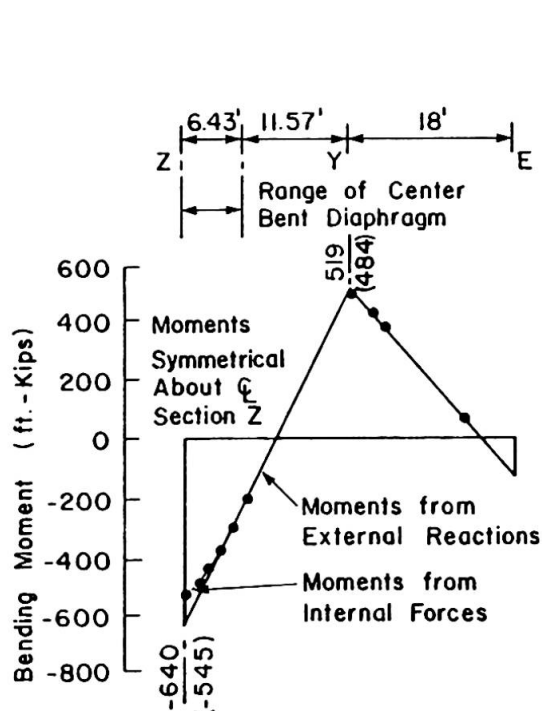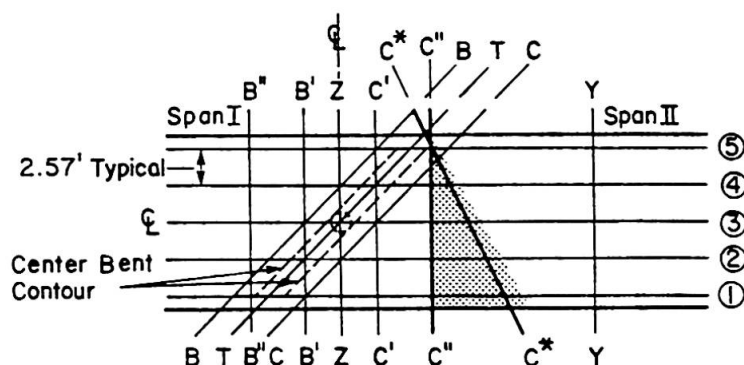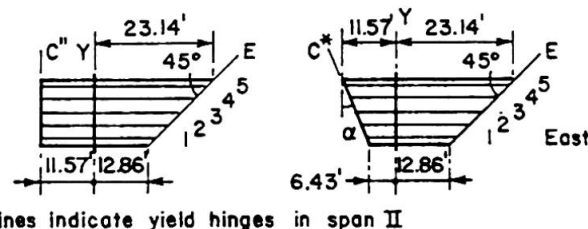structure and thus attenuates the internal box girder moments.

Fig. 4 Theoretical External and
         Internal Moments for Span
         Loads of 100 kips (CELL)

(a) Possible Critical Bridge Sections for Support Hinges

Bold lines indicate yield hinges in span II

(b) Collapse Mechanisms

Fig. 5 Yield Hinges and Postulated
         Collapse Mechanisms

5. INITIAL YIELDING OF THE SKEW BRIDGE STRUCTURE

Because of the skewness of the bridge and different lengths of bar cut-offs, it
is not immediately obvious which sections should be chosen as suitable locations
for the formation of yield hinges. However, as the loads are applied at the
right midspan sections X and Y, the span hinges should form at these sections.
In the region of high negative moment, Fig. 5, the experimental crack patterns
for the top deck after failure of the bridge model in the undiaphragmed span
indicated that possible yield hinges lay in the cracked triangular area of the
deck shown stippled in Fig. 5 and bounded by section C", the first right section
beyond the center bent diaphragm, and the oblique section C* as shown.

Considering the whole region of negative moment in the neighborhood of the sup-
port without the benefit of the known experimental failure pattern in this re-
gion, it is observed that several sections of interest exist which may become
critical for other bridges. In the present case, it is noted that the instru-
mented skew sections B and C at 2.57 ft (0.78 m) on each side of the center bent
skew section T have a high moment capacity because of the large amount of longi-
tudinal reinforcement over the skew center support, and hence failure has not
occured there. The associated right sections B' and C' intersect the thick,
heavily reinforced, skew center bent diaphragm in the region of girders 2 and 4,
respectively, rendering failure along these sections difficult, and necessitat-
ing high collapse loads. Section Z, between B' and C', can similarly be dis-
counted as a possible critical section. With the elimination of these possible
critical sections, collapse mechanisms based on the span hinge at section Y and
support hinges at the sections C" and C* remain to be considered. These collapse
mechanisms are schematically shown in Fig. 5b. From the dimensions it may be

observed that for the section C* the distances 6.43 and 11.57 ft (1.96 and
3.53 m) between sections C* and Y bear the same ratio to each other as the
distances 12.86 and 23.14 ft (3.92 and 7.05 m) between sections E and Y.

An estimation of the bending moment capacity for the various right sections of
the bridge, based on the yield moment capacity of the longitudinal reinforcement
only, assuming a lever arm of 1.5 ft (0.46 m) is given in Fig. 6. The moment
diagram for the dead load of the bridge is also shown. In order to attain the
moment capacity of 1010 kip-ft (1370 kN-m) at section Y, it can be calculated,
using the results from Fig. 4, that a live load of 181 kips (805 kN) per span is
required. The external moments for this load case are shown in Fig. 6 by the
solid line, with a maximum negative moment value at the section Z of -1314 kip-ft
(-1782 kN-m), a value short of the yield moment of -1372 kip-ft (-1860 kN-m).
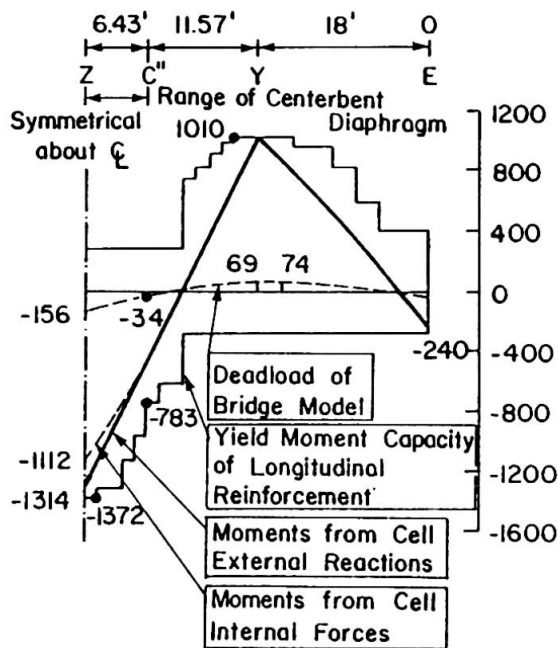
Fig. 6 Resisting Moment Capacity and
       Moments due to Dead Load and Live
       Loads of 181 kips at Sections X
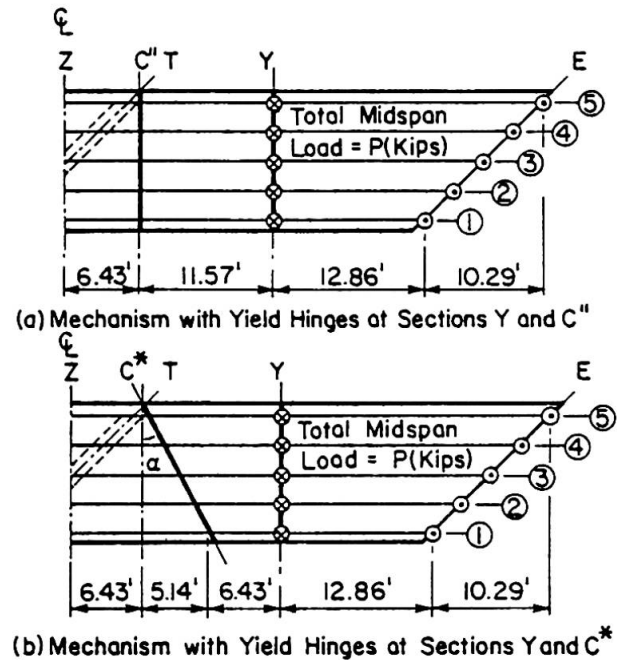   and Y (from CELL Computer Program)

Fig. 7 Failure Mechanisms for
       Span II

The internal moments obtained from the integration of the longitudinal forces
over the box girder cross-section are also shown in Fig. 6. Results indicate
that the participation of the skew center bent diaphragm in resisting the longi-
tudinal bending, reduces the maximum negative moment to -1112 kip-ft (-1508 kN-m).

6. FLEXURAL COLLAPSE MECHANISMS

Considering the model dead load, the live load moment capacity for section Y may
be calculated according to Fig. 6, as 1010 - 69 = 941 kip-ft (1276 kN-m) and at
C" as -783 + 34 = -749 kip-ft (-1016 kN-m). The failure mechanism is shown in
Fig. 7a with yield hinges at these two sections. Only half of the bridge is con-
sidered because of the identical loading in the two spans. The east abutment is
considered at collapse to rotate about the end support skew section E by an angle
θ. The vertical deflection at locations 1Y and 5Y due to this rotation may be
calculated from Fig. 7a as·

$$\delta_{1Y} = 12.86 \ \theta/\sqrt{2} = 9.10 \ \theta \quad \text{and} \quad \delta_{5Y} = 23.14 \ \theta/\sqrt{2} = 16.40 \ \theta$$

The resulting rotations at the support hinge C" can be calculated assuming that some out-of-plane twisting of the bridge segment between sections Y and C" can occur:

$$\phi_{1C''} = \delta_{1Y}/11.57 = 0.79 \; \theta \quad \text{and} \quad \phi_{5C''} = \delta_{5Y}/11.57 = 1.41 \; \theta$$

Using the plastic moment values of 941 and -749 kip-ft (1276 and -1016 kN-m) and evaluating the virtual work for the postulated collapse mechanism, the collapse load (per span) is found to be 198 kips (882 kN). A similar calculation for the collapse mechanism of Fig. 7b, with a live load moment capacity at section C* of 563 kip-ft (763 kN-m), results in the span collapse load equal to 226 kips (1005 kN). The experimentally measured values of 207.1 and 205.5 kips (921 and 914 kN) actually sustained by the bridge are in good agreement with the calculated collapse loads. The actual collapse mechanism probably lies between the two postulated mechanisms.

If the skew box girder bridge were provided with a right diaphragm at the center support instead of a skew diaphragm, the support hinges would normally form at either edge of the 1.96 ft (0.60 m) thick diaphragm. The collapse load for this type of mechanism can be evaluated as 188 kips (836 kN). This indicates that the skewness of the center bent diaphragm effectively decreases the span and thereby increases the collapse load.

7. CONCLUSIONS

Excellent structural behavior was observed during the ultimate loading of the large scale skew box girder bridge model. The model sustained an ultimate load equal to dead load plus 4.5 times design live load, illustrating its excellent overload capacity.

For dead load and uniform overload conditions, the skewness of both the end abutments and the center bent diaphragms reduces the maximum positive and negative moments as compared to a similar straight bridge. The importance of the skew center bent diaphragm lies in transferring the support failure zone from the transverse centerline section Z to section C" or C*, which has the effect of raising the collapse load.

The model test to failure provided reliable experimental data on skew box girder bridge behavior and verified the analytical predictions of the ultimate load response and collapse load of skew reinforced concrete box girder bridges.

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