Investigation of the Double-Composite

Box Girder Failure Criteria

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Overview

The FDOT Structures Research Center performed testing to evaluate the concept of double composite action in steel bridges in October of 2008. The testing consisted of fatigue, service, and ultimate tests. The fatigue test was completed by loading the specimen to approximately 5.6 million cycles from 5 to 105 kips. No immediate distress to the specimen was detected after the fatigue test. The service test involved three load cases with the 1^{st} and 2^{nd} load case being repeated 5 times. The loads for the 1^{st} and 2^{nd} load case were 421.0 kips and 638.8 kips, respectively. The load was held each time for a brief period before retracting. The final load case for service, which became the ultimate load case, involved loading the specimen to 894.2 kips. It was intended during this load case to hold the load at 894.2 kips for several minutes, for examination of the specimen, and then continue until failure or 1200 kips, whichever came first. During the first minute the load was being held, due to the nature of the hydraulic system, a small percentage of the load, approximately 12 kips, was lost. While attempting to regain the 12 kips of load a sudden failure occurred in the specimen. Buckling of the bottom steel plate and concrete failure were observed near the support or maximum moment region. The specimen cross-section and elevation are given in Figures 1 and 2, respectively. An examination of the recorded load, strain, and displacement data was made by the Research Center to determine the cause of the failure.

Failure Synopsis

A visual examination of the failed specimen found that the bottom flange plate buckled between shear stud lines near the support, which were longitudinally spaced at 23 inches. Also, the concrete failure occurred at the general location of the first and second shear stud lines in the same general region of the buckled plate. A depiction of these locations is shown in Figure 3. It was noted that the bottom plate buckled at other locations along the beam also between stud lines; however, this location was the most severe.

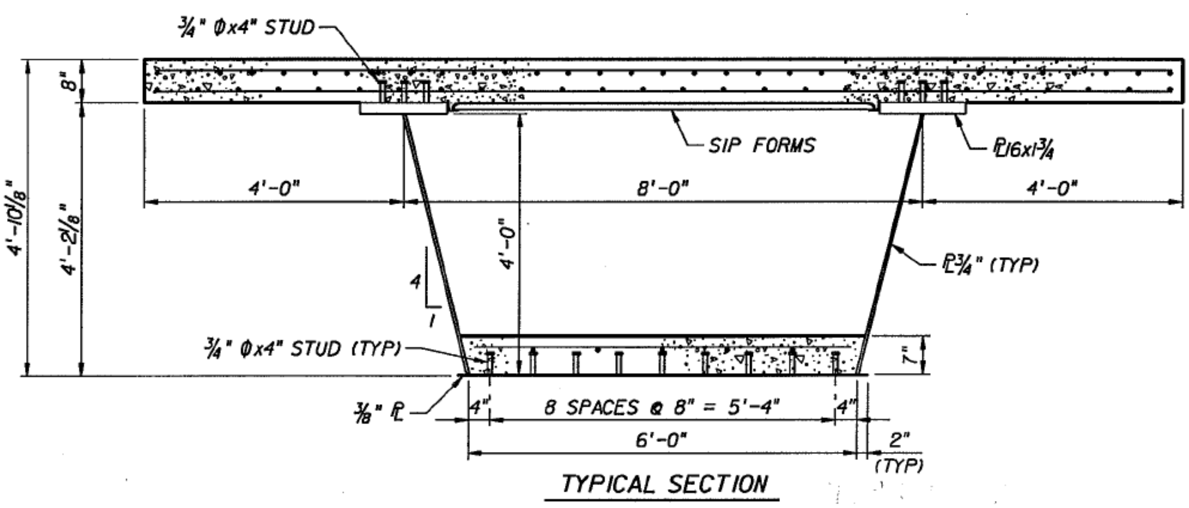


Figure 1: Typical Cross-Section

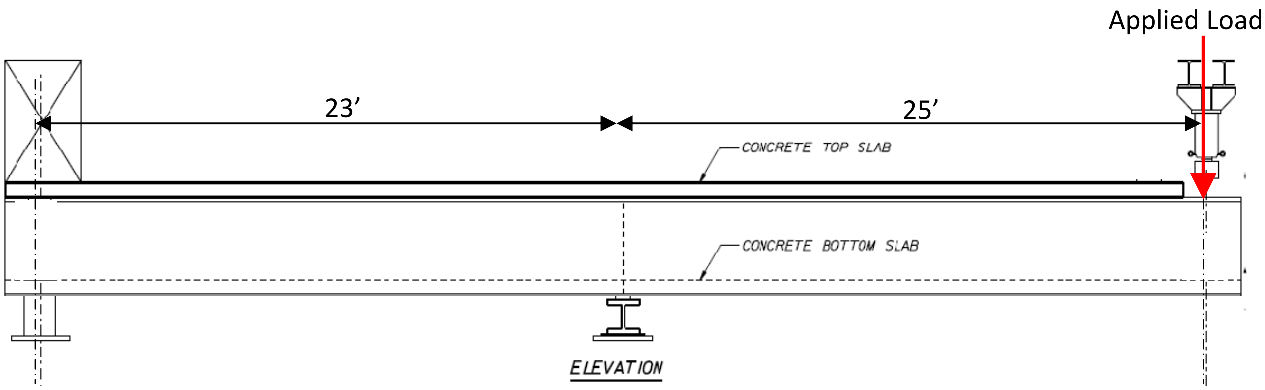


Figure 2: Elevation

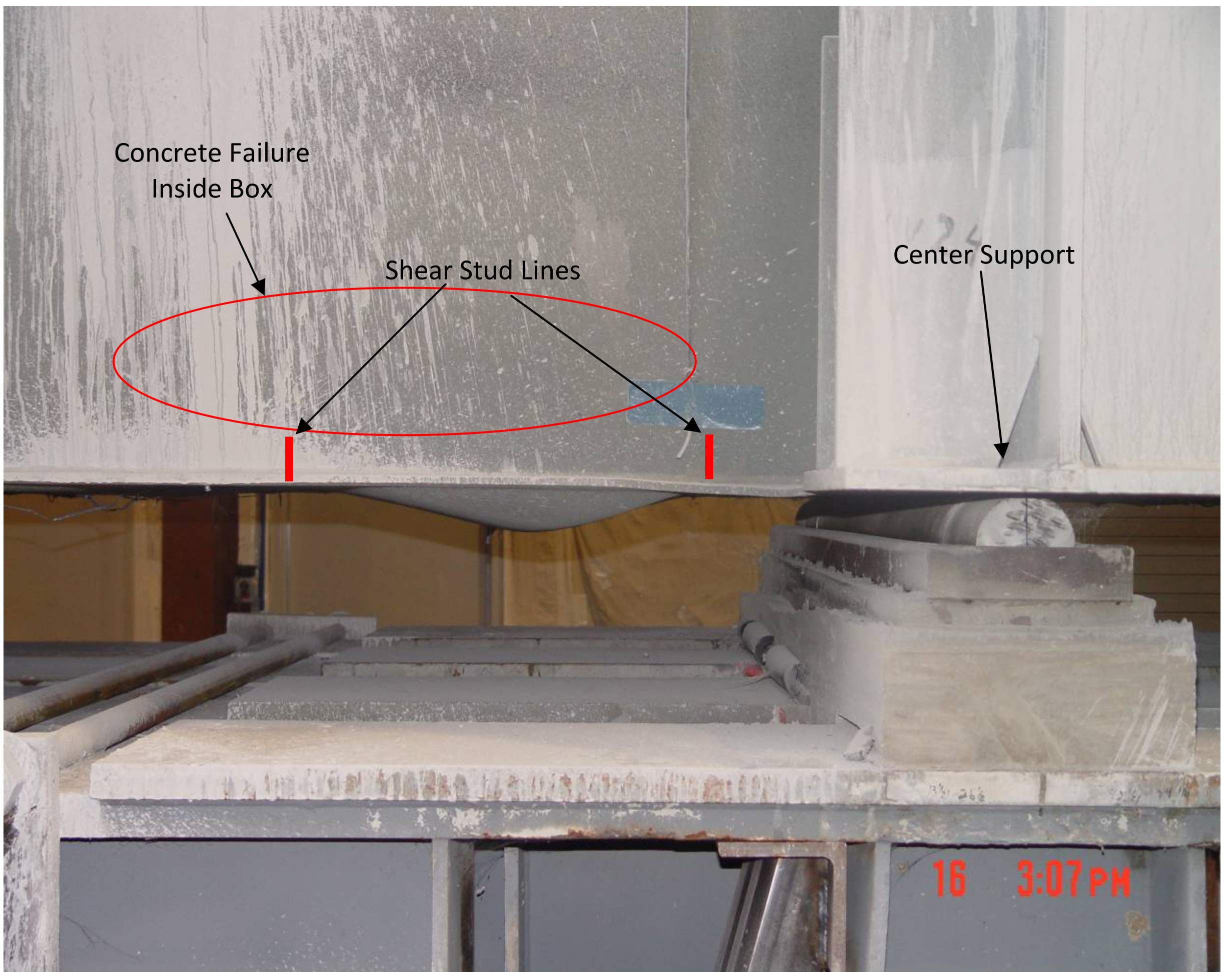


Figure 3: Location of Failure

After analyzing, the data indicates plate buckling occurred in the early stages of loading. A load versus deflection curve for the 1st and 2nd service load case, 1st cycle, is given in Figure 4 for two displacement gages, LV 23 and LV 24, that were located in the region where buckling occurred. This load-deflection curve should theoretically be linear with positive slope. However, there is a noticeable slope change at approximately 130 kips which is indicative of buckling. It is further magnified at gage LV23 above 300 kips on the 1st cycle then around 200 kips on subsequent cycles. A transverse strain gage at the location of failure also suggests that out of plane bending occurred at low loads. Figure 5 is a load versus micro-strain graph showing nonlinearity which is apparent around 150 kips.

Calculations based on *Roark's Formulas for Stress and Strain* were used to study the critical buckling stress in the bottom steel flange. The formula considers a rectangular plate under uniform compression on two opposite edges. It was assumed that all edges were simply supported. The values used in the equation are as follows: ⅜"plate thickness, 23" buckling length, and 72" width between webs. Based on the given setup and equation the critical stress for this location was 8.75 ksi. The calculations are given in the Appendix. The low critical stress level explains the early buckling of the bottom flange. The applied load needed to achieve this stress in the bottom flange at the critical location was 152 kips, based on a composite section. The value of the critical stress or load could vary a small amount due to the exactness of the boundary conditions and should be taken as the lower bound. This early buckling condition eliminated the added benefit of using high-performance steel in the bottom flange (HPS 70).

The behavior of the test specimen during the initial loading stage of this test was complex with the slab and bottom flange not acting completely integral. Due to shrinkage there are minute cracks and gaps at the diaphragms that prevent the concrete from being loaded immediately. This in turn can accentuate the amount of the initial loading resisted by the steel in the bottom flange. This would lower the required load, 152 kips, to produce the critical buckling stress. Once buckling of the bottom flange has occurred the bottom slab concrete would resist a majority of the additional load. Higher stresses would result in the concrete due to the lack of composite action.

At the time in the test when the load was being held, at 894 kips, the concrete capacity was exceeded, resulting in a sudden brittle failure. The concrete cylinder strength was 8700 psi at the time of testing. The concrete failure is visible in the top portion of the bottom slab, see Figures 6 and 7. This region has little confinement with the exposed face and shear studs only extending 4 inches into the 7 inch slab, at a spacing of 23 inches. Two strain gages, SG 109 and 111, located on the top of the bottom slab at 4'-10⅛" from the diaphragm on the hold down side revealed that the concrete in the bottom slab was under distress during the load hold. Figure 8 is a plot of load versus micro-strain, using the average of gages SG 109 and 111, and depicts increasing strain while the load was held constant at 894 kips. By averaging the strain gages along the depth of the box at 4'-10⅛" from the diaphragm on the hold down side and using linear extrapolation the approximate strain level at 11 inches from the diaphragm was 2148 micro-strain in the bottom fiber of the bottom slab and 1513 micro-strain in the top fiber of the bottom slab in compression. The average measured strain gradient along the depth of the box, at failure, is shown in Figure 9. This data includes the average for gages in the top flange, web and bottom slab. The stress-strain curves for three cylinders of the bottom slab concrete are given in Figure 10. The average maximum failure strain for the three cylinders is 2230 micro-strain. The situation for the Double Composite is similar to a cylinder test in that due to the position of the neutral axis there is a small strain gradient across the depth of the bottom slab, however, the cylinders were tested at the ASTM prescribed load rate, as opposed to a held load in the double composite test. Concrete fails at lower stresses under sustained load.

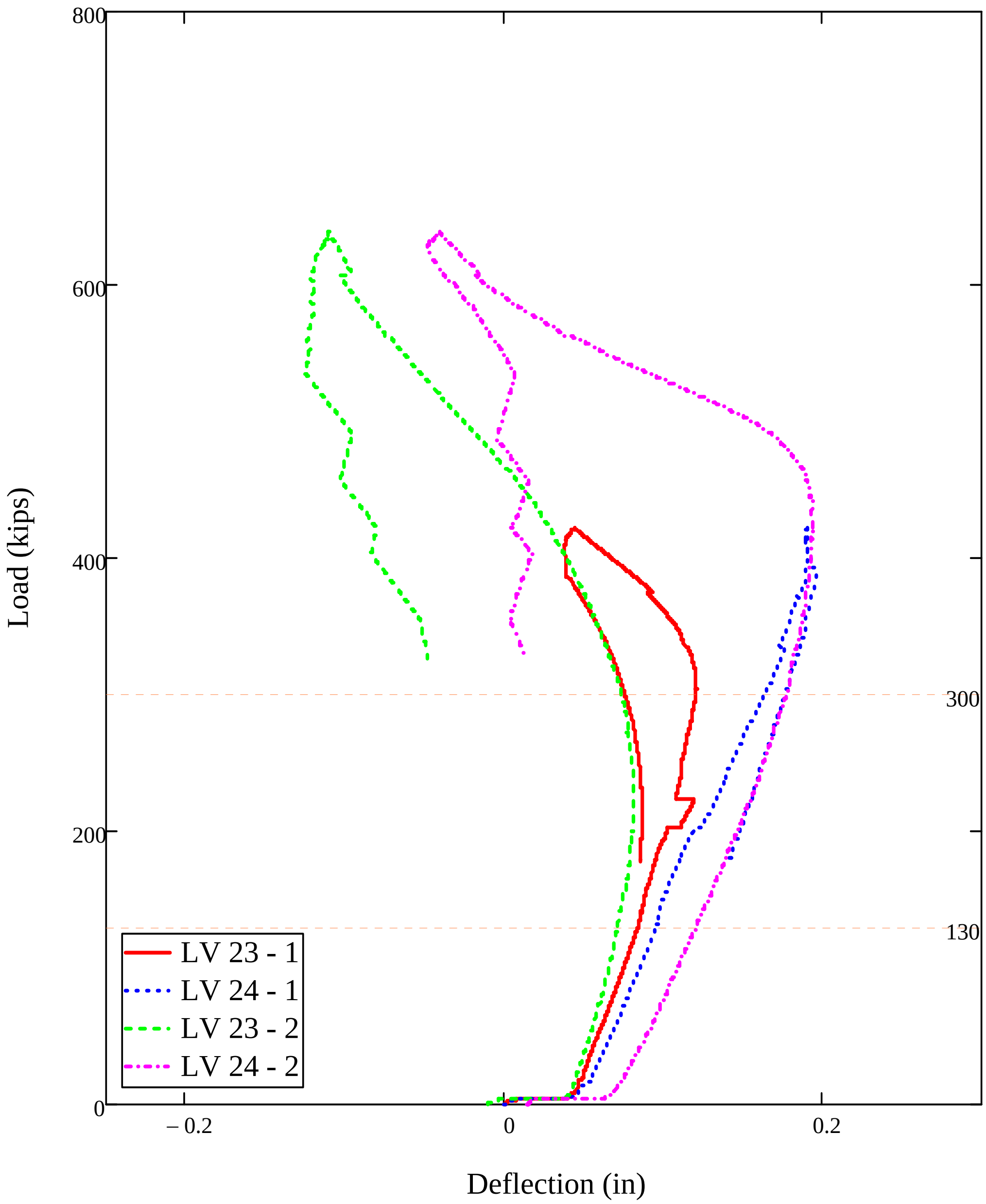


Figure 4: Load versus Deflection (LV 23-24)

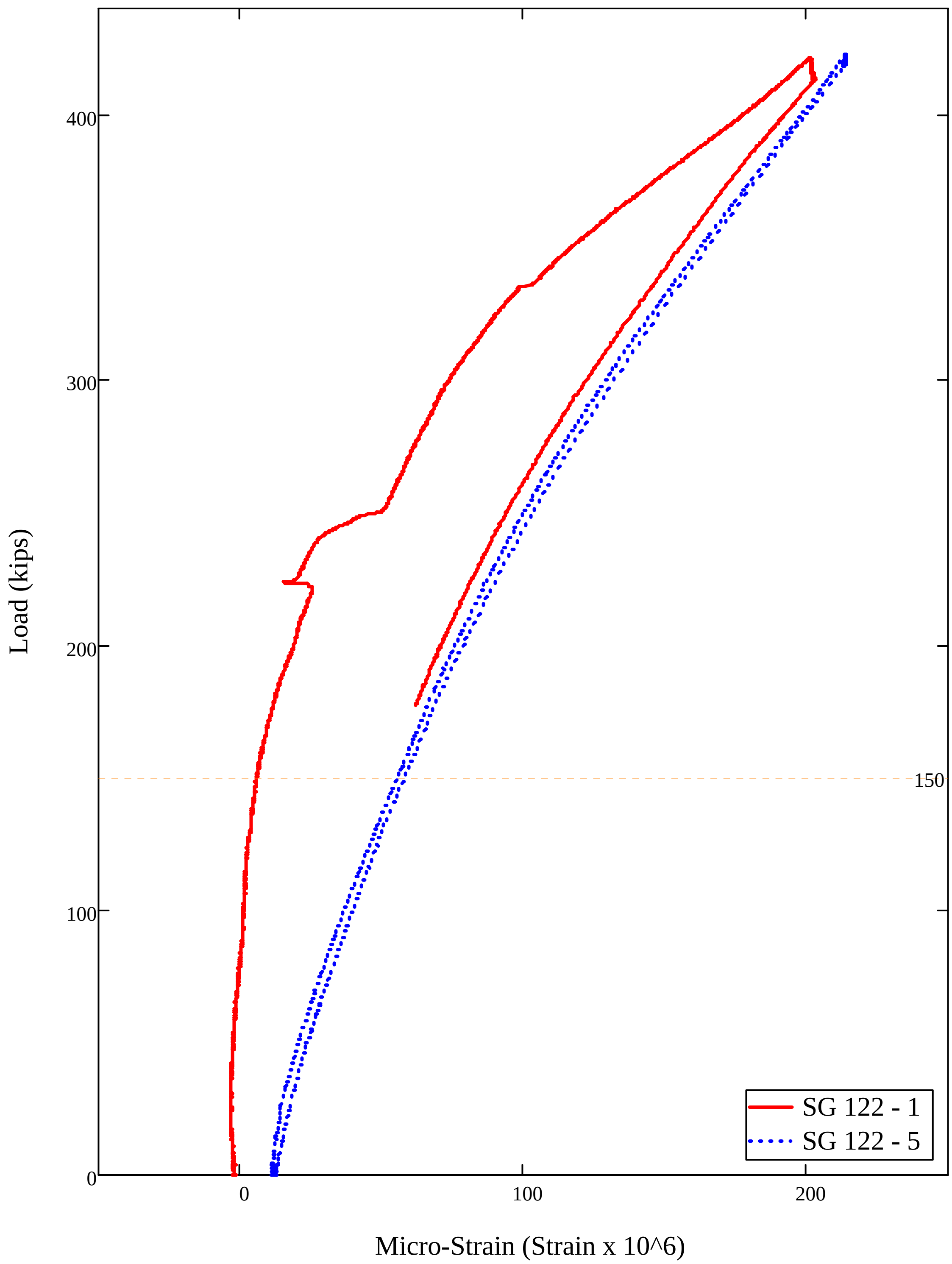


Figure 5: Load versus Micro-strain (SG 122 – Transverse)

Figure 6: Concrete Failure

Figure 7: Concrete Failure (Removal of Loose Pieces)

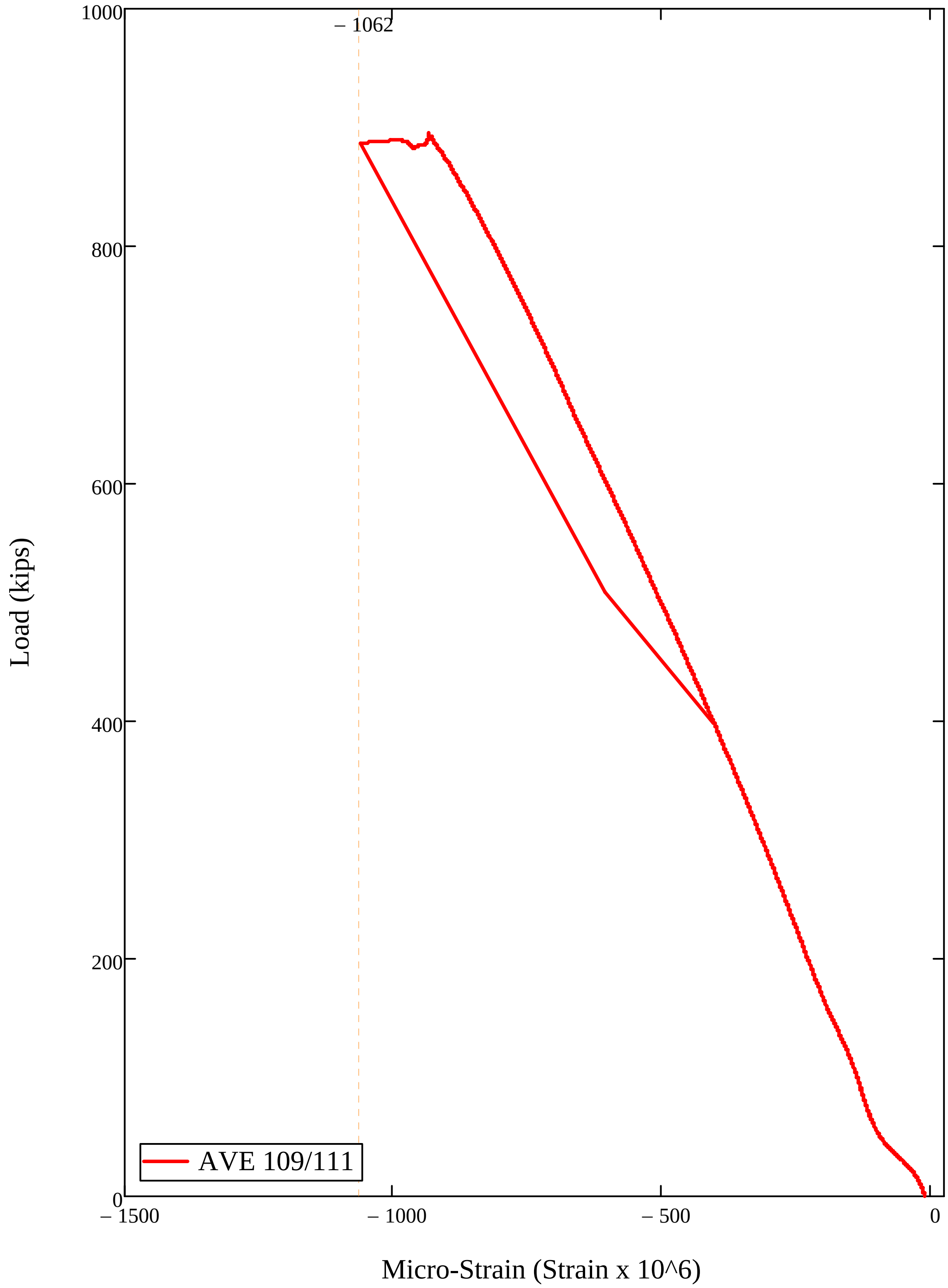


Figure 8: Load versus Micro-Strain (Bottom Slab Strain)

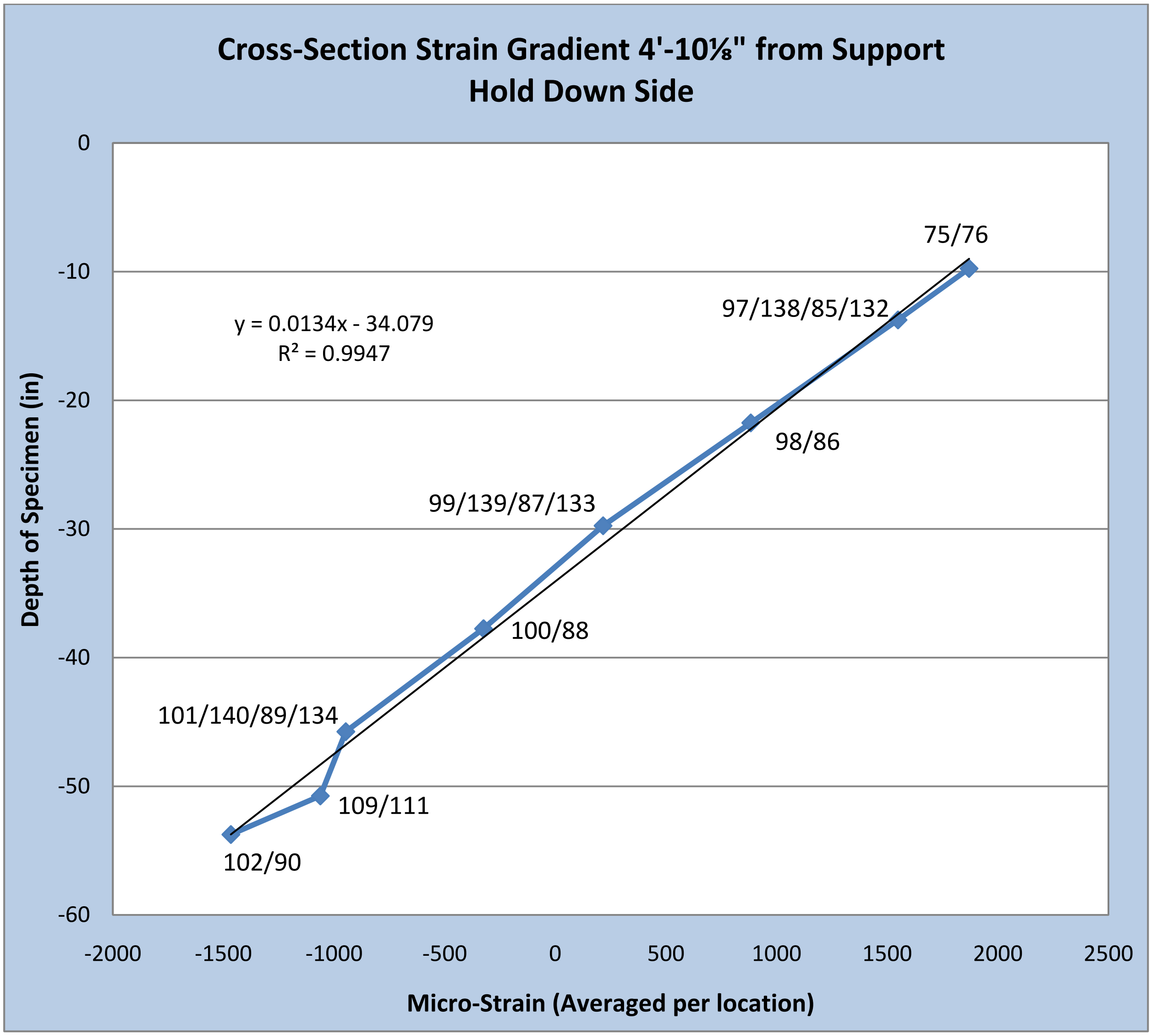


Figure 9: Strain Gradient at 4'-10⅛" from Support – Hold Down Side

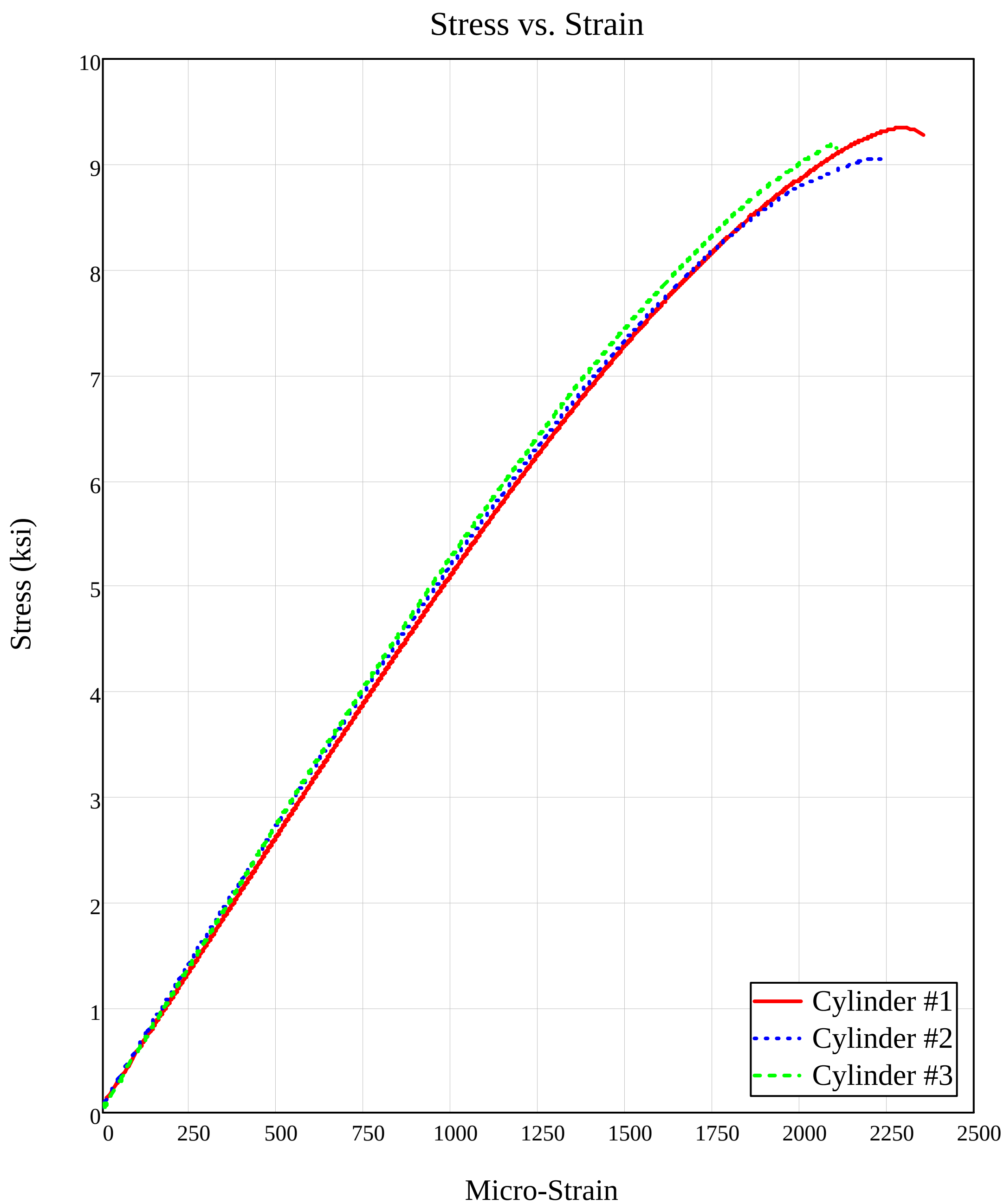


Figure 10: Stress versus Strain from Bottom Slab Concrete Cylinders

Conclusion

The failure mechanism for the given setup was a sudden brittle concrete failure that occurred after elastic buckling of the steel bottom flange at low load levels. The bottom flange buckling could potentially be resolved by using a tighter spacing of studs closer to the support which would reduce the buckling length. This also could provide additional confinement to the concrete. A higher capacity could be obtained; however, this would still entail a sudden concrete failure if the entire section is required to achieve plasticity. For designs of this type the bottom concrete slab and bottom steel flange are composite requiring that the strain levels in the materials match. The concept of achieving the full plastic moment capacity is not possible due to the concrete bottom slab's inability to withstand strains equal to the yield strain of the steel bottom flange. In this particular case, the bottom steel flange yielded at 2750 micro-strain. The concrete failed at approximately 2230 micro-strain in compression. The double-composite design should be limited in design, in negative moment regions, to achieving full plasticity in the top flange only.

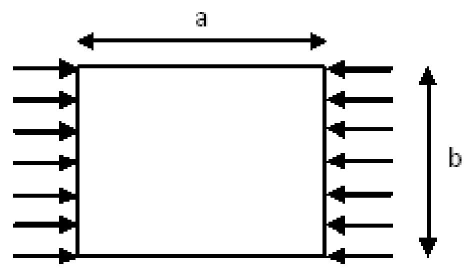
-Appendix-

Roark Formulas *- Elastic Stability of Plates - Rectangular Plate under equal uniform compression on two opposite edges b. Assuming all edges simply supported. Table 15.2.1a (p. 703)*

$a := 23in$ $b := 72in$ $E_s := 29000ksi$ $\nu := 0.3$

$$ab := \begin{pmatrix} 0.2 \\ 0.3 \\ 0.4 \\ 0.6 \\ 0.8 \\ 1.0 \\ 1.2 \\ 1.4 \\ 1.6 \\ 1.8 \\ 2.0 \\ 2.2 \\ 2.4 \\ 2.7 \\ 3.0 \end{pmatrix} \qquad K := \begin{pmatrix} 22.2 \\ 10.9 \\ 6.92 \\ 4.23 \\ 3.45 \\ 3.29 \\ 3.40 \\ 3.68 \\ 3.45 \\ 3.32 \\ 3.29 \\ 3.32 \\ 3.40 \\ 3.32 \\ 3.29 \end{pmatrix}$$

$$\frac{a}{b} = 0.319$$



$$K' := \mathrm{linterp}\left(ab, K, \frac{a}{b}\right) \qquad K' = 10.126$$

$$\sigma' := K' \cdot \frac{E_s}{1 - \nu^2} \cdot \left(\frac{t}{b}\right)^2 \qquad \sigma' = 8.754\,ksi$$

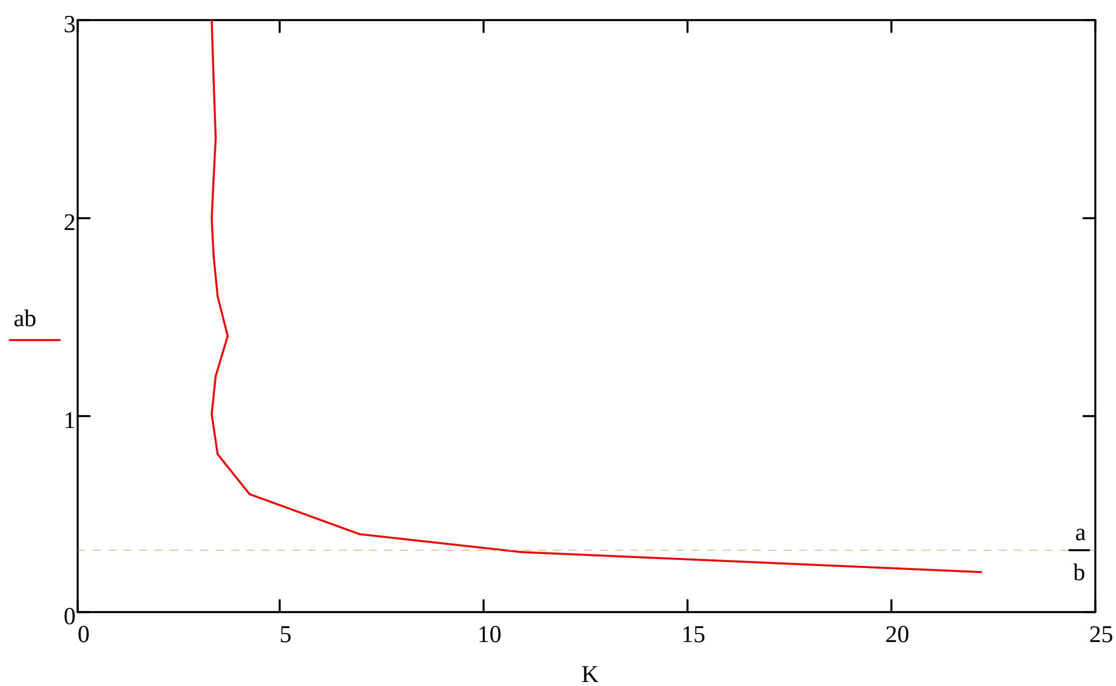


Plate Analysis

$f'_c := 8700\,psi$ $\quad E_c := (0.9)\cdot 57000\sqrt{\frac{f'_c}{psi}}\,psi$ $\quad E_c = 4784.945\,ksi$

$n := \text{round}\left(\frac{E_s}{E_c}, 1\right)$ $\quad n = 6.1$

Effective Width $\quad b := 72\,in$ $\quad \frac{b}{n} = 11.803\,in$

Slab Thickness $\quad t_{bs} := 7\,in$

Section Properties - Total Properties

$A_b := 254.126\,in^2$ $\quad I_b := 112671.273\,in^4$ $\quad y_t := 33.291\,in^3$ $\quad y_b := 22.521\,in$

$S_b := 5002.875\,in^3$ $\quad S_t := 3384.417\,in^3$ $\quad d := 55.813\,in$

*Moment at **11 inches** from support on "Hold Down" end, i.e. north end*

$$M_{appl} = \left(\frac{25}{23}\right)(22.0833\,ft)\cdot P_{appl}$$

Back out moment/load needed to produce the critical stress found in Roark's Formulas

$M_{back} := \sigma'\cdot S_b$ $\quad M_{back} = 3649.516\,kip\cdot ft$

$P_{back} := \frac{23}{25}\cdot\frac{M_{back}}{22.0833\,ft}$ $\quad P_{back} = 152.04\,kip$

***Theoretical** Computed Bottom Flange Steel Stresses with applied load, assuming elastic section throughout loading.*

$$P_{appl} := \begin{pmatrix} 0 \\ 421 \\ 638 \\ 894 \\ 1441 \end{pmatrix} kip \qquad \sigma_{appl} := \frac{\left(\frac{25}{23}\right)\cdot(22.0833\,ft)\cdot P_{appl}}{S_b} \qquad \sigma_{appl} = \begin{pmatrix} 0 \\ 24.239 \\ 36.733 \\ 51.472 \\ 82.966 \end{pmatrix}\cdot ksi$$