Florida Department of
TRANSPORTATION

Solid Concrete Slab Bridges

Effective Width Recommendations

FDOT Office
M. H. Ansley Structures Research Center

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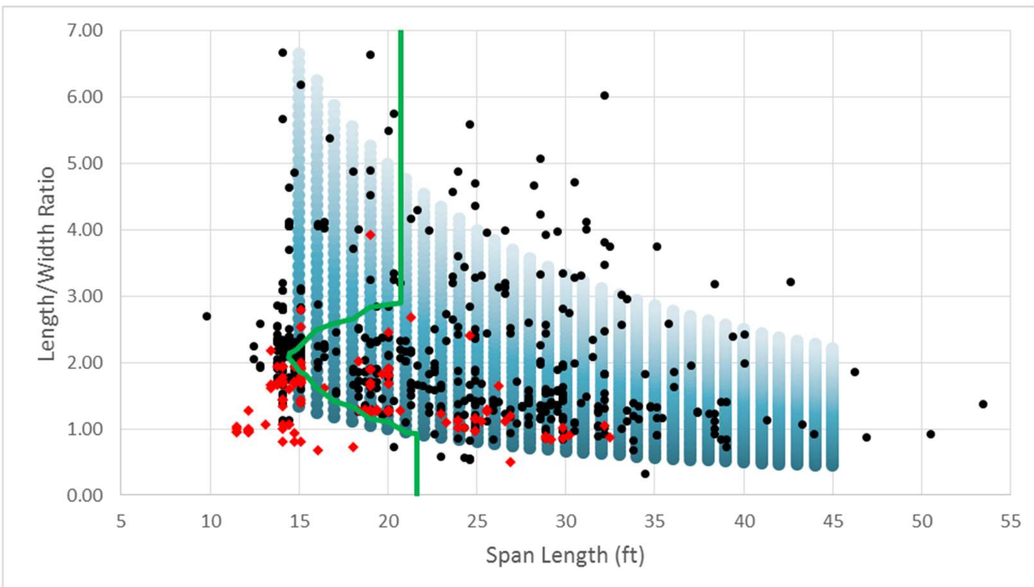
Introduction

The FDOT Structures Maintenance Office requested that the FDOT Structures Research Center investigate the effective width for reinforced concrete cast-in-place flat slabs to improve load ratings for various bridges throughout the state. The effective width equations in the AASHTO Standard Specifications and AASHTO LRFD Bridge Design Specifications are perceived to be overly conservative compared to actual bridge behavior. Although conservative, they remain appropriate for design of new bridges to maintain a reliable margin of safety. Existing bridge load ratings are permitted to have a lower reliability index than allowed for design, to maintain mobility of goods and services while loads increase and design provisions become more stringent during the life of a structure. The information contained in this document is appropriate for load rating purposes only, not design, as the reliability index may be less than typical for design of new bridges.

Current FDOT Inventory

The FDOT simply supported flat reinforced slab inventory, as of March 2016, covers roughly 980 bridges built from 1922 to 2015 of which nearly 170 are posted for load restriction. Nearly half of all flat slab bridges are located in districts 1 and 2 alone. As they were relatively simple to construct and would adequately serve lower traffic areas and smaller span lengths they were often used to cross small creeks and drainage canals across the state. A long stretch of highway such as US-41 will have dozens of bridges built within a short period of each other. The older bridges appear to follow a standard design and are typically shorter span lengths. In theory all of these should have very close load ratings if conditioning is not taken into account. The following paragraphs detail some of the overall characteristics of the flat slab inventory as well as attempts to find reasons for varying ratings or effective slab widths.



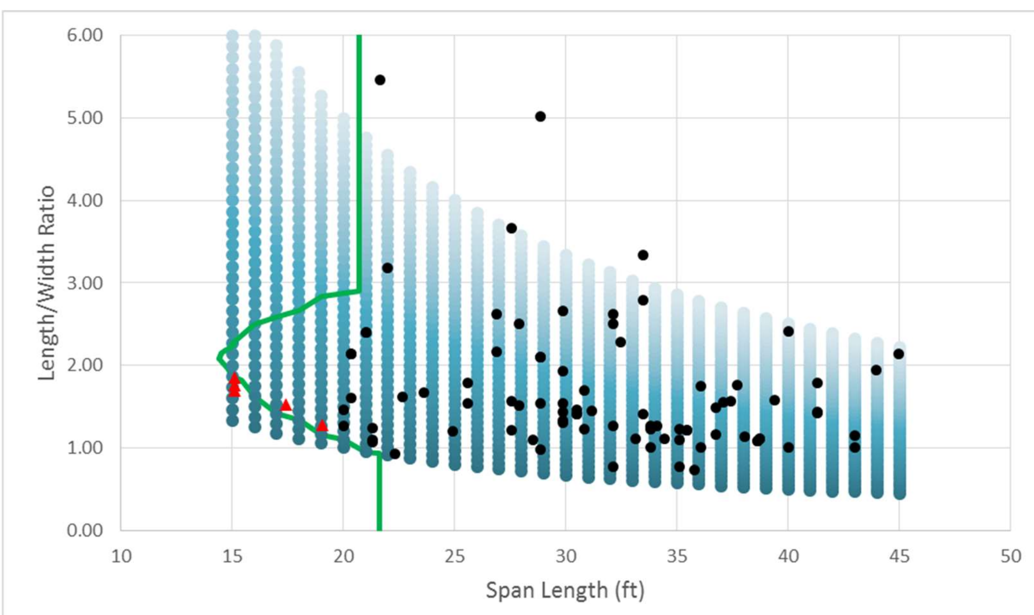


Figure 1: LFR (top) and LRFR (bottom) bridges are compared to the single and multi-lane

Shown above are graphs depicting the aspect ratio of all flat slab bridges coded as having been rated with LFR and LRFR along with a range of aspect ratios. The black dots represent non-posted bridges, red dots represent load restricted bridges, and the blue dots represent a range of aspect ratios for lengths most commonly found in the flat slab inventory. All values to the left of the green line are bridges that would that would typically have the single lane AASHTO LRFR effective width control the load rating. Bridges to the right would have multiple lanes control. Not shown in the graphs are how the edge strip width would alter the rating. This relationship varies based on curb and parapet widths therefore each curb width would have an individual graph depicting which of the three effective width cases would control in a particular bridge. What is important to note from this graphic, however, is that in general the LRFR bridges do not have nearly as many posting and that the majority of the postings occur in bridges with aspect ratios of less than 2.

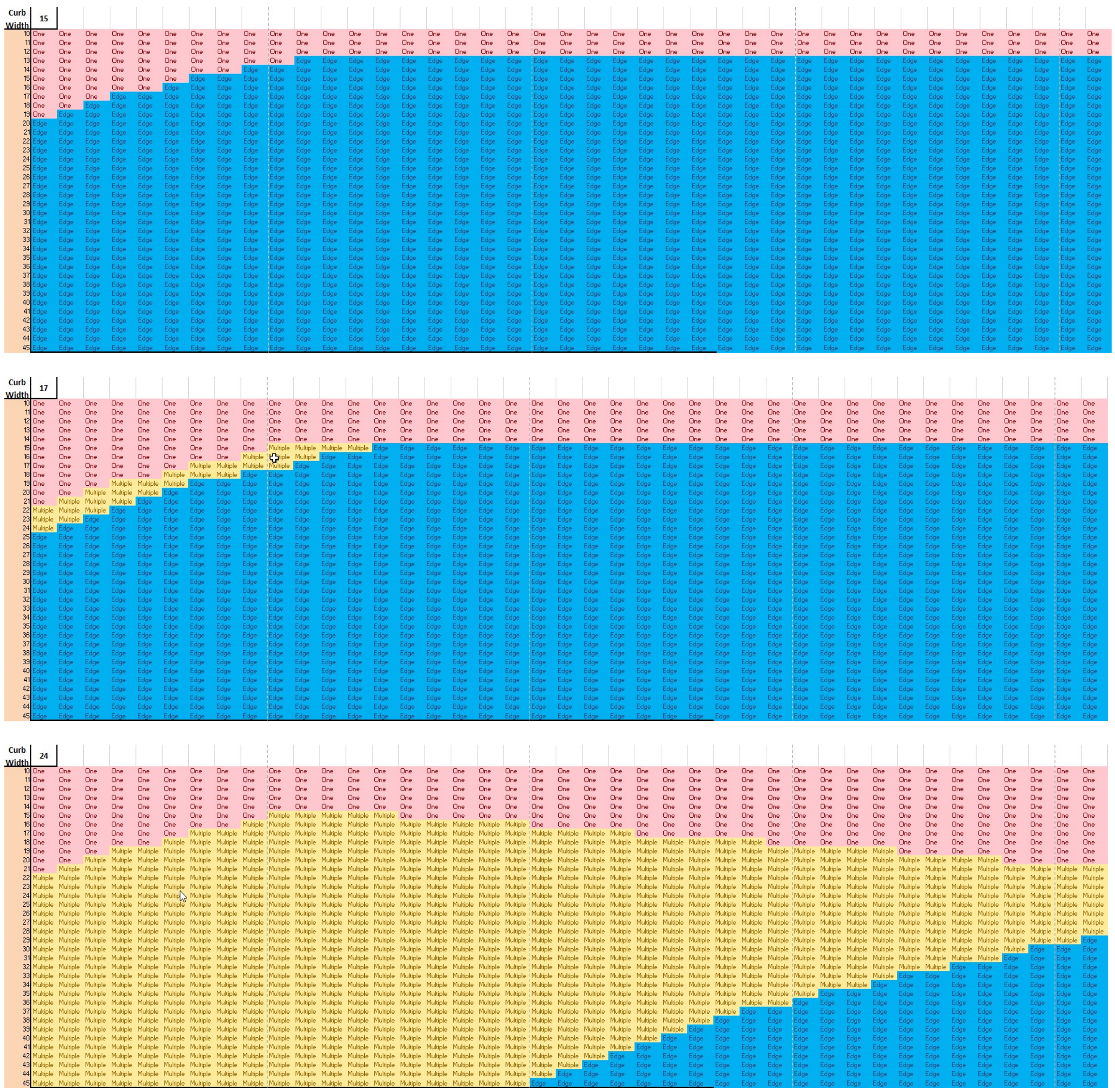


Figure 2: Progression of Edge Strip Control with Increase in Parapet Width

Although difficult to determine when a bridge will be governed by the edge strip, general relationships can be shown. Using the effective width equation in AASHTO the edge strip width will effectively always control if there is less than 8in of total parapet width on the bridge. As the parapet width is increased the single lane begins to control shorter, narrower bridges. Beginning at 17in total parapet width the multiple lane controls certain aspect ratios and the division shown in the earlier graphs between single and multiple lanes starts becoming more obvious. Above 24in any change in curb or parapet width will not change the controlling AASHTO equation; 199/773 LFR bridges have a curb width of 24in or less. For bridges with parapets less than 10ft wide the posting percentage is much higher than those above 10ft: 20.9% compared to 4.5%. Of the 141 LFR bridges posted for load restriction only 6 of them have total parapet widths of 10ft or more.

In addition to the edge strip width potentially altering the load rating, the outside strip of flat slab bridge also contains the barrier which vary greatly throughout the inventory. Almost 200 bridges have a steel guardrail; 76 of these are load restricted. Of the 134 that still have a concrete post and rail 21 bridges are posted. Compared to the larger but stiffer Jersey type barrier with only 5 postings out of 222 bridges it appears that barrier type may be related to the load rating.

Using the AASHTO equation to back calculate the expected distribution factors it becomes apparent that even for the simple LFR effective width there is little agreement between expected distribution factors and what is in PONTIS. Nearly a quarter of the flat slab bridges in PONTIS have been removed from this analysis due to an implausible distribution factor. For LFR bridges 46.1% match within ±2.0% while 33.3% fall within ±0.5% of the calculated values. LRFR bridges are 15.6% and 6.7%. The updated AASHTO LRFR equation has much more variation and can change somewhat drastically based on the aspect ratio of a bridge. In other words for a given span length there can be multiple effective widths even if the curb width is held the same, which is true for the values shown.

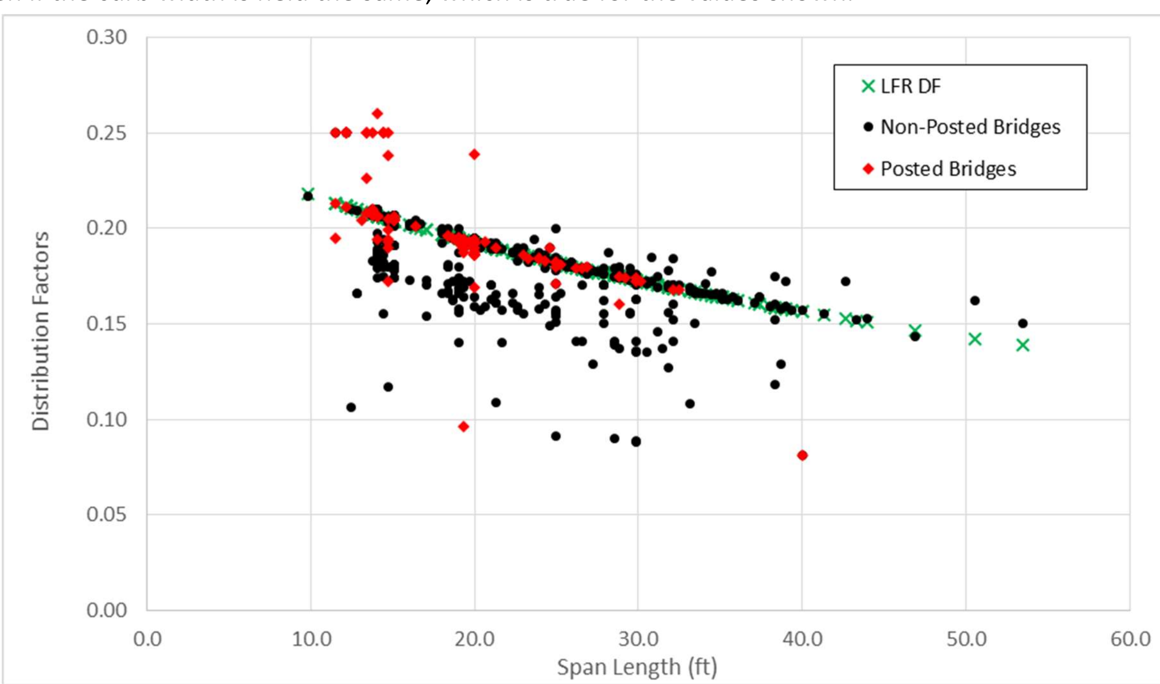


Figure 3: LFR Rated Bridges with Expected LFR Distribution Factor

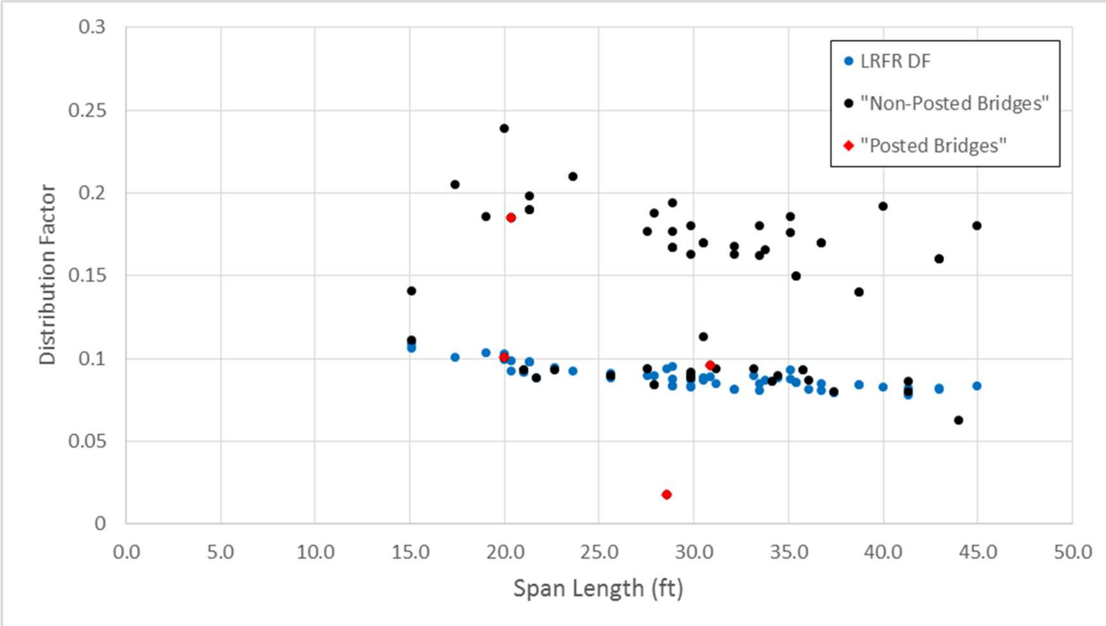


Figure 4: LRFR Rated Bridges with Expected LRFR Distribution Factor

The average increase of distribution factor if load ratings are upgraded to LRFR would be about 12%. This would decrease the average load rating by about 10.7%. Since the majority of posted flat slab bridges are in LFR then upgrading may actually lower already poorly rated bridges. The following graph shows how the calculated distribution factors change when the load rating method is upgraded.

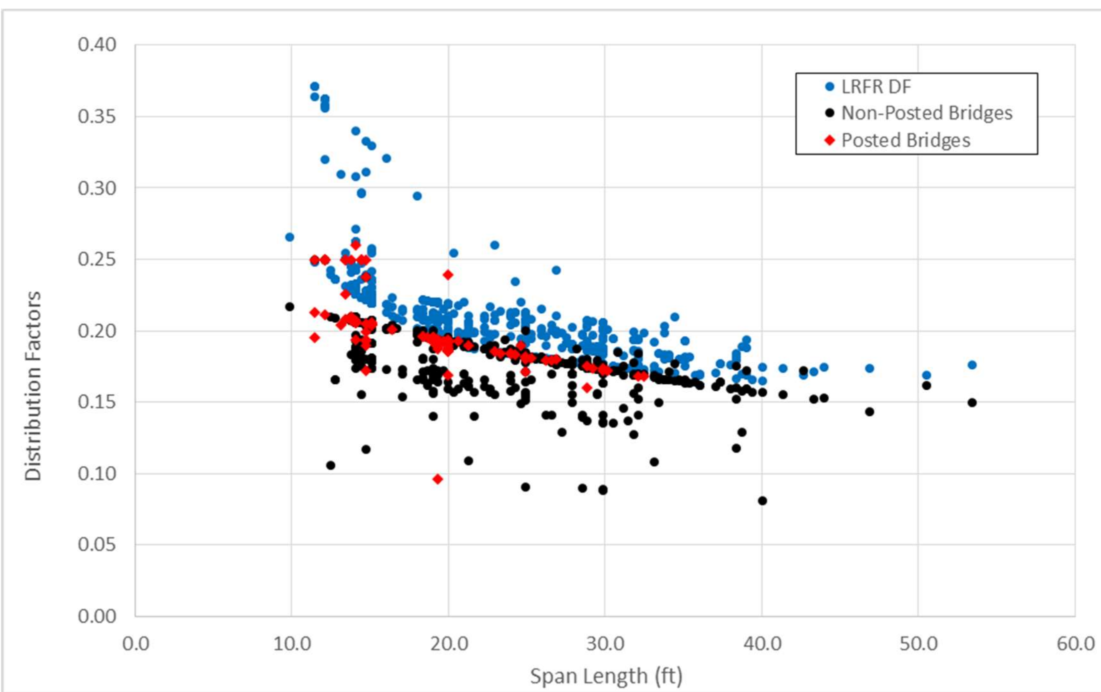


Figure 5: LFR Rated Bridges with Expected LRFR Distribution Factor

The overwhelming method chosen by load rating engineers to determine effective widths is the AASHTO equation followed by SALOD. Of these two methods the AASHTO LFR seems to produce load restricted bridges more than SALOD, 18.4% and 12.1% respectively.

Although many of the bridges match very closely when compared to the expected LFR and LRFR distribution factors, an unexpectedly high percentage of LFR bridges appear to have distribution factors much higher than either AASHTO equation can produce. For the purposes of this analysis a distribution factor of 0.40 or above was considered to be high. Several selected bridges from the group of bridges with a high distribution factor were found to be mis-coded in the bridge inventory. They are precast slab bridges and therefore not the target of this study. As there may be more mis-coded bridges, those with a distribution factor above 0.40 are excluded from this study.

Since simply upgrading load rating codes will not provide a substantial increase to the overall load ratings of bridges, a look was taken at how SALOD would affect the borderline postings. Comparing the distribution factors coded as SALOD to the AASHTO LFR distribution factors the average bridge would have an increase in load rating of almost 13% due to a decrease of roughly 11% in distribution factor. Using the definitions from pg. 108 of the BMS coding guide to further breakdown the inventory based on the level of posting it is possible to determine whether this increase can possibly provide immediate improvement to posted bridges.

CODE	to Legal Load Stress (Operating Factor)		Posting
5	AT/ABOVE LEGAL LOADS	1.000 up	Not Required
4	0.1 TO 9.9 % BELOW	0.901 - 0.999	Required
3	10.0 TO 19.9% BELOW	0.801 - 0.900	Required
2	20.0 TO 29.9% BELOW	0.701 - 0.800	Required
1	30.0 TO 39.9% BELOW	0.601 - 0.700	Required
0	> 39.9% BELOW	0.000 - 0.600	Required

Figure 6: Definition of Load Posting Levels

Table 1: Breakdown of Posting Levels for LFR Bridges

Posting Level	0	1	2	4	5
Number of Bridges	25	12	23	43	37

For bridges that are coded as having a level 4 load rating (up to 9.9% below the acceptable operating load rating) this small increase could prove beneficial. If the rating exceeds 9.9% the posting level changes from 4 to 3 although there is a possibility that level 3 posting may have borderline ratings that may still benefit from the increase SALOD may provide.

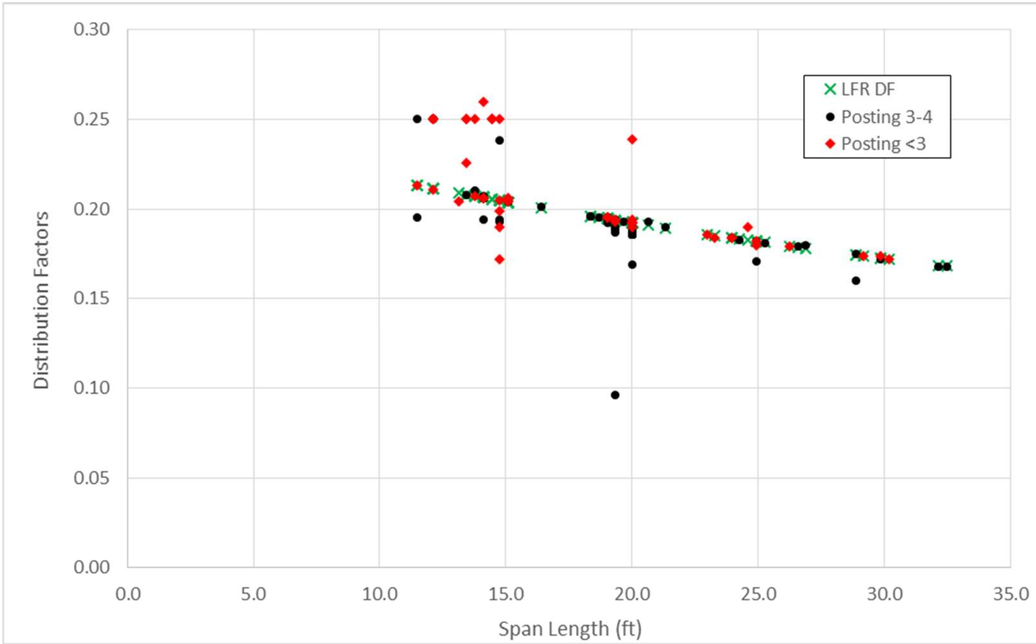


Figure 7: Posted LFR Bridges Compared to LFR Calculated Distribution Factor

Literature Review

This section provides a summary of current code language for live load resisting effective bridge widths for flat slab bridges. Development of the current equations is discussed along with work done by others to evaluate, compare or improve code language. The current code equations for effective width were derived with the conservative assumption that no curb or parapet exists. The effective width could be increased by considering the parapet in two ways. Including the parapet width in the analytical model increases the transverse distance between the truck and edge of the slab and increases the effective width resisting load. Solid barriers or curbs also provide increased stiffness for slab beam bridges but are not generally considered structural members. Potential increases of the effective widths due to consideration of the curb or parapet would increase load ratings, but work done to-date is limited and has not been adopted by AASHTO.

Simple Equations for Distribution

Effective widths for cast-in-place reinforced concrete slab superstructures can be calculated using AASHTO's two available methods, Load Factor Design (LFD) and Load and Resistance Factor Design (LRFD). Load Factor Design is based on the AASHTO Standard Specifications, discontinued in 2005, while LRFD is based on the more current AASHTO LRFD Bridge Design Specifications.

The LFD effective width is specified in section 3.24 of the AASHTO Standard Specifications, for main reinforcement parallel to traffic. The wheel line distribution width is Equation 1, with lane loads distributed over a width of 2E. "E" is defined as the transverse distance over which a wheel line is distributed, in feet. "S" is defined as the span length, in feet. The AASHTO Standard Specifications additionally requires the edge beam of a simple span to be designed to resist a live load moment, Equation 2. "P" is equal to the wheel load, in lbs. The equations date back to before 1970 (AASHTO, 2002) (AASHO, 1969). The distribution factor formulas in AASHTO (2002) are simple, but not particularly accurate. In some cases, highly un-conservative results are produced, in other cases the results are overly conservative (AASHTO, 1994).

Equation 1: Effective Width (AASHTO, 2002)

$$E = 4 + 0.06\,S \ \leq 7 \quad \text{(feet)}$$

Equation 2: Edge Beam Live Load Moment (AASHTO, 2002)

$$M = 0.1\,P\,S \quad (lbs.-ft.)$$

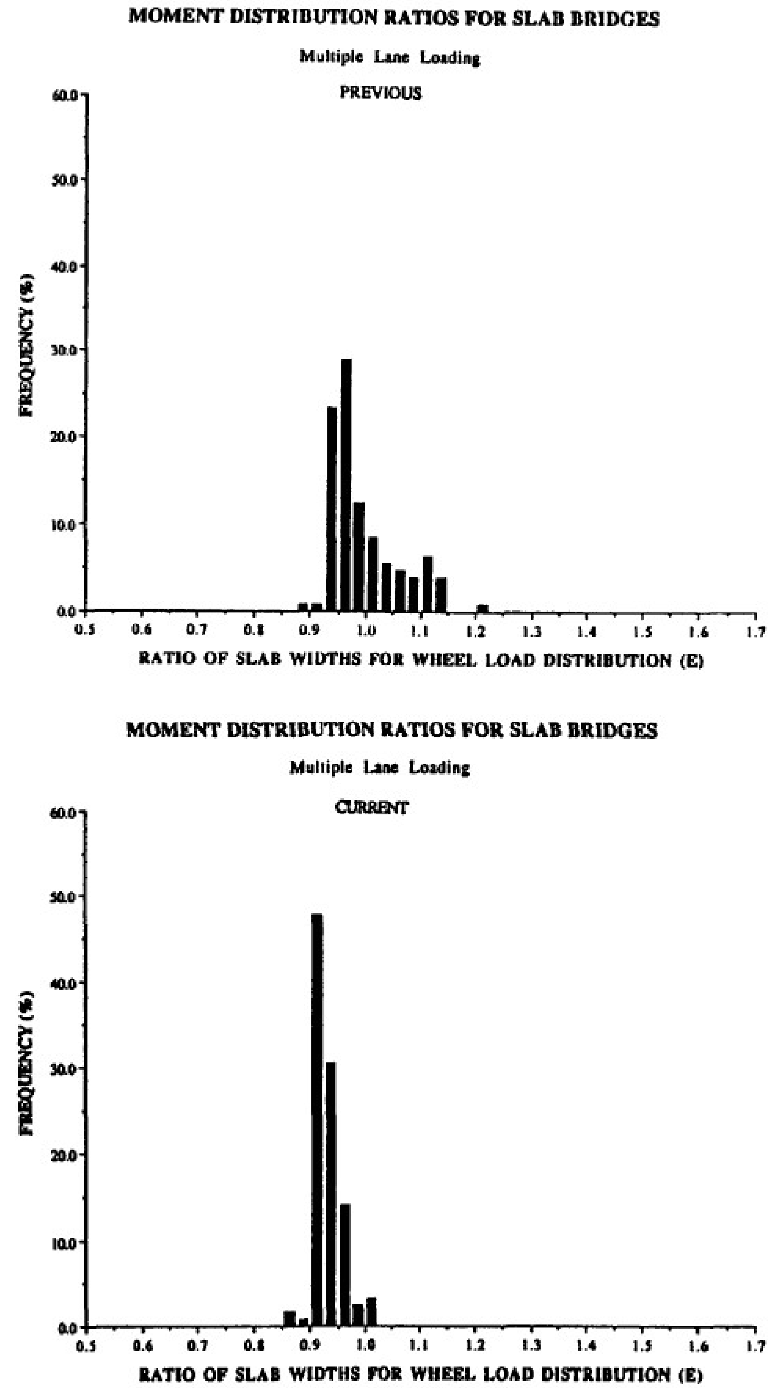
Completed in 1992, Zokaie (1992) documents the findings of NCHRP project 12-26. That research project entailed deriving simple formulas for load distribution using an exponential function. The derived equations for distribution were compared to a more accurate analysis method, such as FEA. The research project developed Equation 3 and Equation 4 for effective width of concrete slab superstructures. Correction for skew effects were also specified, but are not included here. "L_1" is defined as the span length and "W_1" is defined as the edge to edge bridge width. Both values are measured in feet and are not to exceed 60 feet.

Equation 3: Effective Width for Single Lane Loading per NCHRP 12-26

$$E = [2 + (L_1 W_1)^{0.5}]/4 \quad \text{(feet)}$$

Equation 4: Effective Width for Multi-Lane Loading per NCHRP 12-26

$$E = 3.5 + 0.06\,(L_1 W_1)^{0.5} \quad \text{(feet)}$$



Comparison of Accuracy of the Previous and Current AASHTO Formulas for Slab Bridges

Figure 8: Excerpt from (AASHTO, 1994)

The AASHTO Guide Specifications for Distribution of Loads for Highway Bridges adopted Equation 3 and Equation 4 from NCHRP project 12-26 with one revision. The maximum width used for single lane loading is reduced to 30 feet, from 60 feet. The equation is in section 3.24.3.2 (AASHTO, 1994). In comparison to the AASHTO Standard Specifications, the formulas for effective width developed by NCHRP 12-26 and revised in the Guide Specifications are very accurate. The Figure 8 histogram plot shows the accuracy of previous and current AASHTO formulas. The previous formula is Equation 1 from the AASHTO Standard Specifications and the current formula is Equation 4.

When adopted into the AASHTO LRFD Bridge Design Specifications code, Equation 3, effective width for single lane loading, was divided by 1.2 to account for the multiple presence factor. The effective width was doubled to an effective width for the entire lane and not simply a wheel line, a change consistent for all distribution factors in the code. A practical upper bound was added to the effective width for multi-lane loading, representing the total bridge width divided by the number of lanes. Both equations were also converted to inch units. The result is Equation 5 and Equation 6. “W” is the physical edge-to-edge width of bridge, in feet. Other variables remained the same as previously defined (AASHTO, 2017).

Equation 5: LRFD Effective Width for Single Lane Loading

$$E = 10.0 + 5.0\sqrt{L_1 W_1}$$

Equation 6: LRFD Effective Width for Multi-Lane Loading

$$E = 84.0 + 1.44\sqrt{L_1 W_1} \leq \frac{12.0W}{N_L}$$

Published after development of the codified equations, Amer, et al. (1999) documents the development of an effective width equation (Equation 7) for solid slab bridges based on grillage analysis of 27 cases. Bridges with an aspect ratio (length: width) between 0.5 and 1.6 were investigated. The main parameters affecting equivalent width are also identified. Span is an important parameter in load distribution, in agreement with both the AASHTO LFD and LRFD equations for effective width. Also in agreement with the AASHTO equations, slab thickness is not included as a variable in the effective width equation. Based on that set, bridge width insignificantly affects effective width. In contrast, the AASHTO LRFD effective width equation includes bridge width. Ignored for derivation of the AASHTO equations, this research determined that effective width is significantly affected by the edge beam depth. A second equation is provided (Equation 8), which adjusts the effective width based on the edge beam depth above the slab thickness.

The grillage analysis was compared to real world measurements taken during three field tests. For one bridge, the results were very close. For the other two bridges, the grillage analysis was significantly conservative (40%) in comparison to field tests. Equation 7 and Equation 8 have been converted from as-published SI units to US units for presentation here. “E” is the effective width, in feet, “L” is the span length, in feet, and “d” is the edge beam depth above the slab thickness, in feet. Neither equation has been adopted by AASHTO.

Equation 7: Effective Width (Amer, et al., 1999)

$$E = 6.9 + 0.23\,L$$

Equation 8: Effective Width Factor for Edge Beam Height (Amer, et al., 1999)

$$C = 1.0 + 0.1524\,(d - 0.5)\ \geq 1.0$$

The University of Delaware Center for Innovative Bridge Engineering (CIBrE) developed another formula for calculating effective width with funding from the Delaware Department of Transportation. The scope of work included diagnostic testing on six slab bridges, analysis of the load test data to produce an effective width and development of new formulas for estimating the slab effective width. Their work and conclusions are detailed in Jones and Shenton (2012).

The six bridges selected for testing are representative of approximately 250 concrete slab bridges in Delaware that needed load rating evaluation. Previous research indicated the code effective width may be conservative. The research was intended to provide a more accurate and less conservative effective width equation, based on the six bridges tested, which could be applied to the larger group of 250 bridges. The six bridges tested were selected because they had a varying range of parameters providing an accurate respresentation of the larger set of bridges. For the bridges tested, spans ranged from 8 ft to 20 ft, widths ranged from 26 to 47 ft, aspect ratios from 0.17 to 0.68 and slab thickness from 10 to 18 inches. The authors note that "caution should be used when applying the new equations to bridges that fall outside of these ranges" (Jones & Shenton, III, 2012).

For some characteristics, FDOT flat slab bridges fall within the range of bridges tested by the CIBrE and in other characteristics, FDOT bridges are substantially different. For comparing FDOT bridges to those tested by CIBrE, the bridge inventory database, as of March 2016, was examined. A majority, 72%, of Florida flat slab bridges fall within the range of aspect ratios tested by CIBrE, 0.2 to 0.7. 90% lie within a slightly wider range of 0.2 to 0.9. Of the range of widths tested, 68% of Florida flat slab bridges have widths between 26 and 47 feet, while the overall range is substantially larger, 12 feet to 336 feet. In the characteristic of span length, the bridges differ more substantially. Only 53% of FDOT flat slab bridges have spans in the range tested by CIBrE, 8 to 20 feet. A large percentage of slabs have longer spans of up to 40 feet. Slab thickness is not recorded in the FDOT bridge database, so that information is not available for comparison.

All of the bridges tested by CIBrE are single span bridges with end bents consisting of solid walls. In photographs provided by Jones & Shenton (2012), all six bridges appear to be box culverts, which typically have a moment connection between the slab superstructure and end bent walls. Due to the moment connection, the structure behaves as a frame and restraint from the substructure affects superstructure behavior. Bridge plans are not included in the report, but Jones and Shenton (2012) indicate "construction of the bridges is similar to that of a frame," confirming that the bridge is a box culvert but not providing details on the extent of end span restraint. Frame-like behavior may account for some of the inaccuracies between the measured behavior and AASHTO equation for effective width.

In comparison, many of the Florida flat slab bridges targeted for load rating improvement by the work detailed in this report are true slab bridges and not box culverts. Many are multiple span and inherently would not benefit from frame action. A typical detail from construction plans for Bridge 260038 is shown in Figure 9. At expansion bearing locations, there is no connection between the slab and bent cap. At fixed bearing locations, a dowel connection is provided, but the length of number of dowel bars would not be sufficient to transfer enough moment to significantly affect slab behavior.

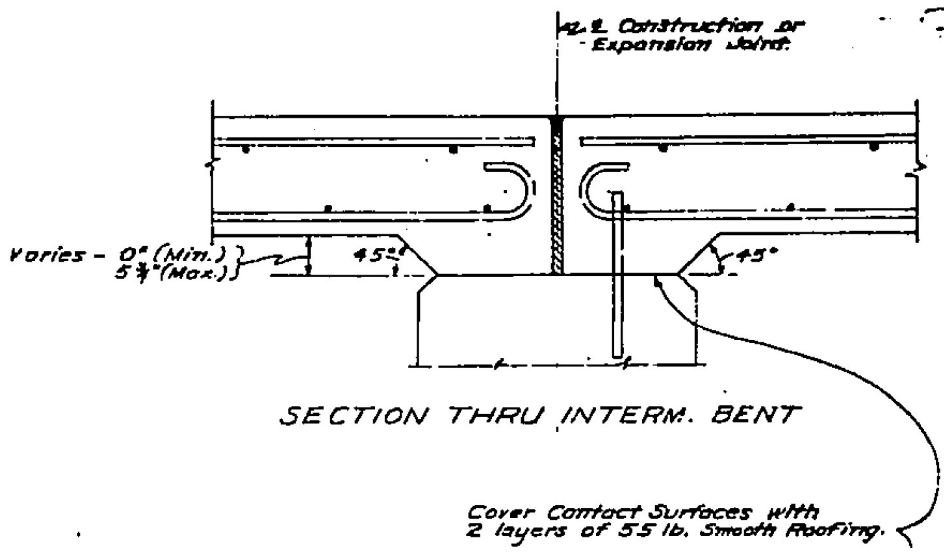


Figure 9: Bridge 260038 Section Thru Intermediate Bent

Florida bridges are very similar to the CIBrE tested bridges in aspect ratio, but more than 30% of Florida bridges fall outside the bridge width range and almost 50% fall outside the bridge span range. With consideration of all parameters simultaneously, only 34% of Florida flat slab bridges fall within the range of parameters tested by CIBrE. Limiting use of the formulas to bridges that fall within the tested parameters would leave a significant number of bridge load ratings un-improved. In addition, bridges tested by CIBrE had some moment restraint at the end of each span while typical Florida slab beam bridges do not. If the formula is used for rating slab slab bridges in Florida, verification that the formulas are appropriate would be necessary.

At lower span ranges (15 feet) for multi-lane loading, the four methods of calculating effective width, LFD, LRFD, Amer, et al. (1999) and Jones and Shenton (2012) are in close agreement. As span length increases, the three methods diverge slightly. For single lane loading, LFD is the most conservative method except for narrow bridge widths, while for multi-lane loading, LRFD is most conservative. The methods presented in Amer, et al. (1999) and Jones and Shenton (2012) are least conservative. The Amer, et al. method is more unconservative for multi-lane loading. The comparisons are presented graphically in Figure 10 and Figure 11. For the LFD and Amer, et al. (1999) equations, the effective width is the same for single and multi-lane loadings. For the LRFD equation, between spans of approximately 15 feet to 22 feet, the effective width for single lane loading may control the bridge rating analysis, depending on bridge width, while for typical spans above that range, the effective width is controlled by multi-lane loading. For the Jones & Shenton method, multi-lane loading controls the effective width.

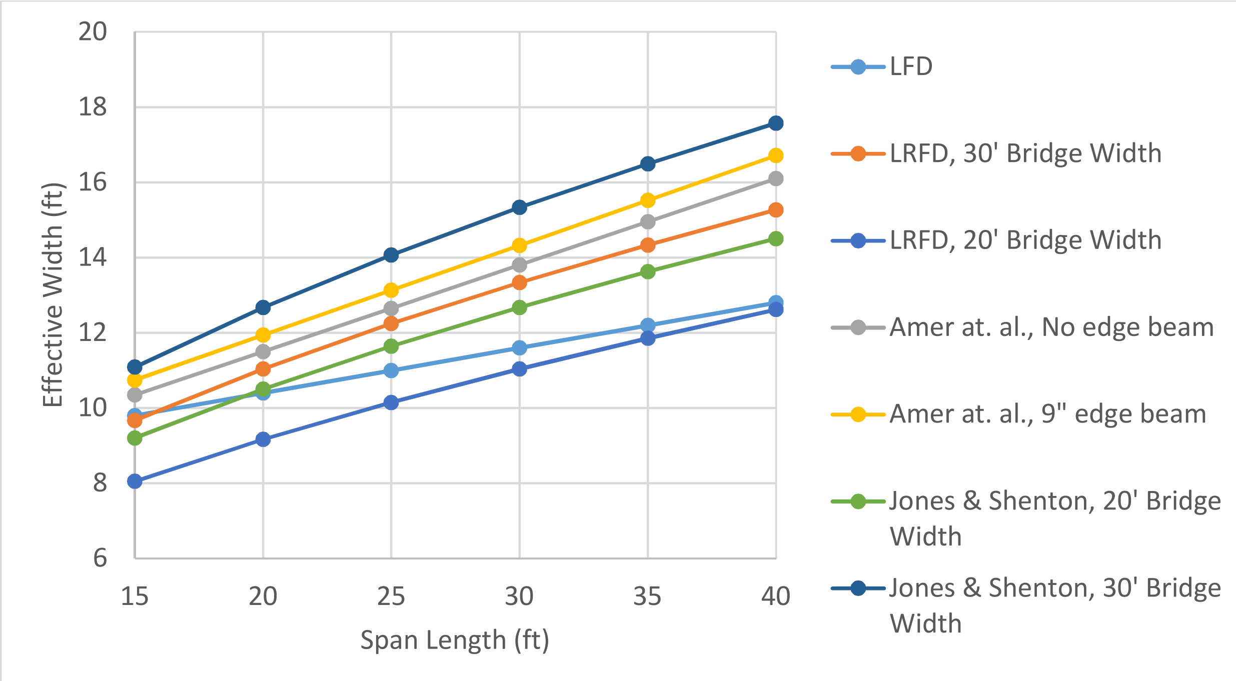


Figure 10: Effective Width for Single Lane Loading

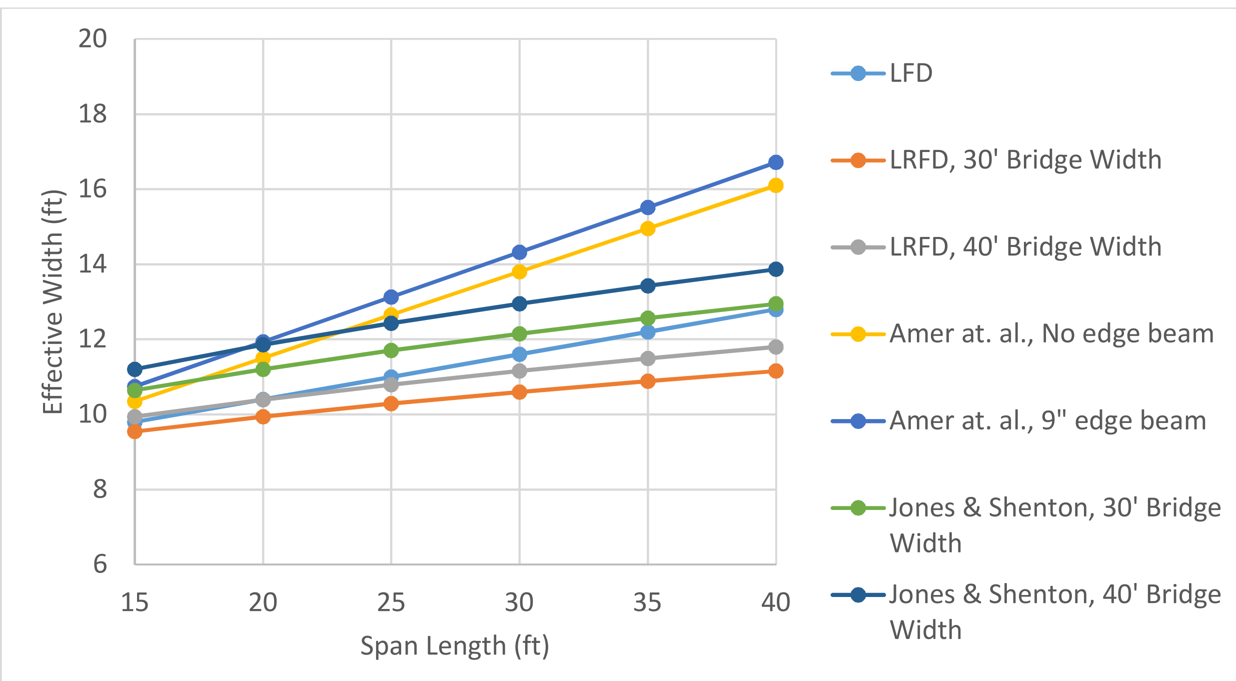


Figure 11: Effective Width for Multi-Lane Loading

A comparison between the AASHTO LFD and LRFD effective widths was created using current FDOT inventory as the subject population. The data is graphed as a percent change in distribution factor, where the distribution factor is equal to 1 foot divided by effective width. The percent change between the methods is shown in Figure 12. For all bridges in the FDOT inventory, the LRFD effective width or distribution factor is conservative compared to the LFD distribution factor. In a select number of cases with a span close to 20 feet, the distribution factor is approximately equal for the two methods. For those bridges, the LFR rating could be improved to an LRFR rating without an effective change to the bridge load rating.

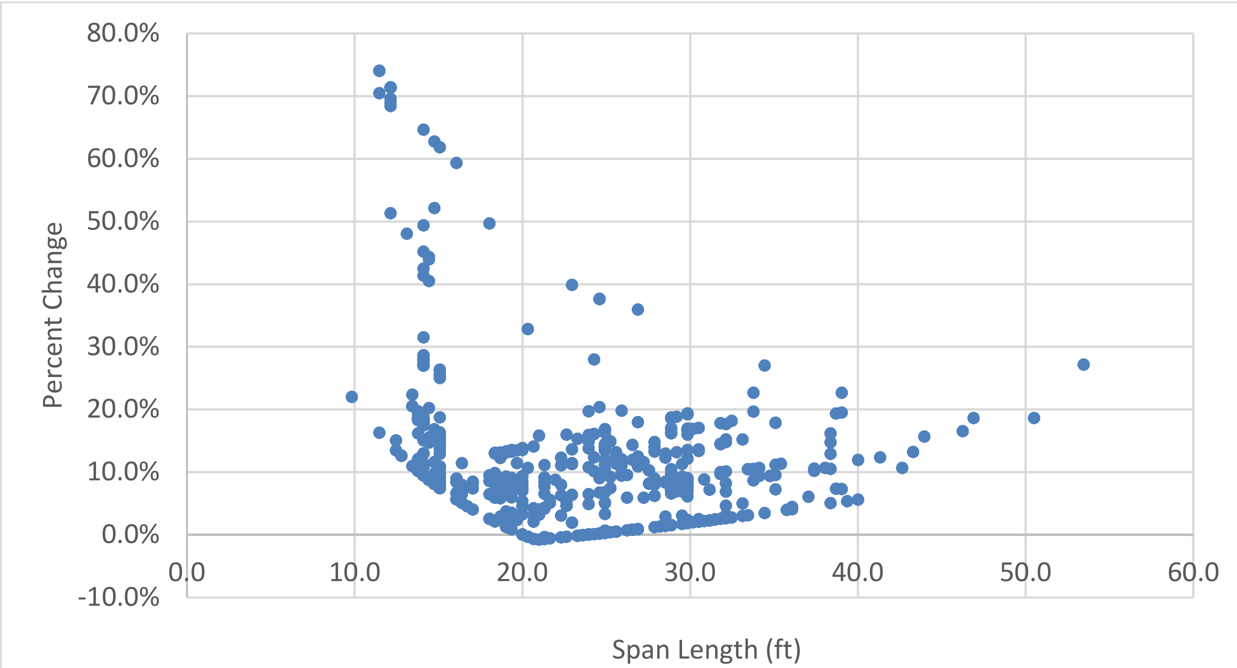


Figure 12: Percent Change in Calculated Distribution Factor from LFR to LRFR

The equations for distribution factor published in AASHTO were evaluated by Mabsout, et al. (2004). The research compared both AASHTO LFD and LRFD equations to a more detailed finite element analysis. 112 case studies were considered, all single span simply supported concrete slabs. Four different clear roadway widths were investigated – 14 ft., 24 ft., 36 ft. and 48 ft. Cases with and without additional 4 foot shoulders were considered. Four different span lengths were included – 24 ft., 36 ft., 46 ft. and 54 ft. with corresponding slab thicknesses of 18 in., 21 in., 24 in. and 27 in., respectively. Corresponding aspect ratios (length: width) for the bridges considered range from 0.43 to 3.85, typical for Florida inventory. However, the research did not address very short span bridges – 15 ft. to 20 ft., which are common in Florida. Truck loading positions considered include two trucks centered on the bridge and two placed close to one edge of the slab (Mabsout, et al., 2004). Bridges with shoulders had an additional loading position, including a disabled truck near the edge and design trucks in each lane, resulting in three loaded trucks on the bridge. The three truck loading case is referred to as "edge+truck."

Findings from Mabsout, et al. (2004) related to the AASHTO effective width equations are summarized in the following paragraph. Sentences one through five relate to AASHTO LFD, while the last two sentences relate to AASHTO LRFD. The conclusions are limited because 24 feet was the shortest span considered. The research includes longer spans of up to 55 feet, but 95% of flab slab concrete bridges in Florida have a span length less than 38 feet. In addition, the findings may be conservative because the maximum live load effects occur due to "edge+truck" loading. It is not specified whether or not the multiple presence factor of 0.85 per Table 3.6.1.1.2-1 in AASHTO (2017) was considered for the "edge+truck" loading case with three trucks placed on the bridge.

> *"For slabs without shoulders, where the edge load condition is critical, and for one-lane bridges, AASHTO…overestimates the FEA moments (30%) for short spans (up to 7.5 m or 25 ft) and agrees with the FEA for longer spans. For more than one lane, AASHTO agrees with the FEA for short spans (less than 10.5 m or 35 ft) and underestimates FEA (15 to 30%) for longer spans. Reinforced concrete slab bridges with shoulders on both edges tend to increase in load carrying capacity. Therefore, the edge+truck load condition was found to be critical for bridges with shoulders on both free edges where AASHTO agrees with the FEA for short spans (up to 7.5 m or 25 ft) and underestimates the FEA by 25% for longer spans, regardless of the number of lanes. Therefore, a suggested 20% reduction factor applied to the FEA moments for span lengths greater than 10.5 m (35 ft), in combination with at least two lanes, will tend to give results similar to those of AASHTO… The AASHTO LRFD procedure gives higher bending moments than AASHTO standard specifications as well as the FEA results. The AASHTO LRFD procedure gives design bending moments closer to the FEA results subject to edge+truck load conditions." (Mabsout, et al., 2004)*

Effect of Transverse Truck Placement

The work detailed in Mabsout, et al. (2004) included multiple transverse truck positions. The truck closest to the edge was located either 1 foot or 3 feet from the edge of the slab. The one foot spacing is typically only used for the design of deck overhangs, as specified by AASHTO (2017), section 3.6.1.3.1. The 3 foot spacing is more appropriate for overall longitudinal bending and therefore effective width equations. The 3 foot width includes a 1 foot parapet with the center of the design truck wheel load placed 2 feet from the edge of the design lane or parapet. The maximum moment closest to the edge of the bridge was not considered; it is resisted by the edge beam. The second peak of the bending moment diagram, used for effective width comparisons, is approximately equal (within 5%) for the 1 or 3 foot wheel placement.

The effect of transverse truck placement was also evaluated in Hays & Hachey (1984), which documents development of the load distribution software SALOD. Flat slab bridges considered had a constant width of 30 feet and a span length ranging from 10 feet to 60 feet, for an aspect ratio (length: width) ranging from 0.3 to 2. The results agree well with the AASHTO LFD effective width formula for spans up to 40 feet. Above spans of 50 feet, the SALOD program is about 10% more conservative than AASHTO LFD. The most conservative effective width usually occurs for a truck located at the exterior edge of the slab. Similar to Mabsout, et al. (2004), the truck wheel line location was as close as 1 foot from the edge of the slab. By increasing the parapet width input into SALOD, the location of the truck wheel line is moved further from the edge of the slab. A series of SALOD analyses for a 40 foot span bridge, with parapet widths varying from zero to 3 feet were completed (Hays & Hachey, 1984). The minimum effective width increased by 13% for a parapet width increase from zero to 1 foot. Each subsequent one foot increase in parapet width increases the effective width by about 6%.

The effect of moving the wheel line transversely was found to be more significant by Hays & Hachey (1984) than by Mabsout, et al. (2004). Several differences in the research approach may account for the difference in results. The models used for analysis had different finite element mesh sizes. For Mabsout, et al. (2004), the mesh size was 1 foot by 1 foot, while for Hays & Hachey (1984), the mesh size for the specific cases considered ranged from 2.8 foot by 4 feet to 3.4 feet to 4 feet. Smaller mesh sizes produce higher moments for a thin flat slab model and may be unnecessarily conservative. Another difference may be due to the bridge dimensions considered for the analysis. For Mabsout, et al. (2004), the bridge aspect ratios ranged from 0.43 to 3.85, while for Hays & Hachey (1984), the aspect ratios for the specific bridges used for the transverse truck placement investigation ranged from 1.2 to 1.4.

Both research reports discussed here prove that effective width increases as the transverse distance between the edge of the slab and wheel location increases. Improvements to the AASHTO LRFD effective width, Equation 5 and Equation 6, could be feasible due to the conservative assumptions made to develop those equations. Although the transverse truck placement used for derivation of the AASHTO LRFD effective width equations is not documented in Zokaie (1992), the author was contacted. He stated the curb width was assumed to be zero and the center of the closest wheel line was placed a minimum of 2 feet from the edge of the curb. The edge of the curb is also the edge of the slab. The 2 foot distance is in agreement with the requirements of AASHTO (2017) for the analysis of longitudinal bending. The use of a zero value for curb width is conservative (Mabsout, et al., 2004) (Hays & Hachey, 1984). However, the degree of conservatism is in dispute and may range from 5% to 13%. The effective width equations could be improved and made less conservative with consideration of the actual curb width.

Effect of Edge Stiffening

For load rating purposes, structural resistance of a concrete barrier or parapet is not considered for typical, simple analyses. For flat slab bridges, neglecting the barrier stiffness can be a huge penalty due to the relatively high stiffness of the barrier compared with the relatively low stiffness of the reinforced concrete slab. In several research reports, the presence of a curb or barrier has been noted to have a significant effect on the behavior of a flat slab concrete bridge, particularly at the edge (Azizinamini, et al., 1994) (Sessions, 1985) (Hays & Foley, 1985). As the edge beam moment of inertia increases, the slab moment decreases and the effective width increases (Arockiasamy & Amer, 1995). Based on grillage analogy of a 21 foot long, 30 foot wide bridge, a 9-fold increase in the edge beam section modulus results in a 10% decrease in the maximum moment and effective width.

The effect of edge beam stiffness was also investigated in Hays & Hachey (1984). Three different cases were considered – no barrier, a 36 inch tall barrier and a 36 inch tall barrier with a construction joint at centerline. The stiffened slabs showed a much higher moment at the edge of the slab, for a load placed at that location. For a load placed one-fifth of the slab width from the edge of the slab, the slabs with a barrier had a lower moment than the slab without a barrier, although the difference was minimal. The presence of an edge beam such as a curb or barrier has significant effects at the edge of the slab, less so towards the middle of the slab. And, while edge stiffening decreases moment demand for the slab, the edge beam moment increases.

LRFD criteria requires edge beams to support one line of wheels and consists of a reduced deck strip width. Any local thickening or protrusion within the edge beam width may be included in the stiffness of the edge beam (AASHTO, 2017). LFD criteria has a different moment demand: 0.1 times the wheel load and span length. The edge beam definition is also more prescriptive. “The beam may consist of a slab section additionally reinforced, a beam integral with and deeper than the slab, or an integral reinforced section of slab and curb" (AASHTO, 2002). Allowing the barrier or curb to be part of the structural system for bending resistance is a departure from other sections of the code. For instance, sections 4.5.1 and 4.6.3, relating to mathematical modeling and refined methods of analysis, allows a composite barrier to only be considered for the service and fatigue limit states (AASHTO, 2014).

Load rating processes typically follow the requirements of AASHTO LRFD or LFD. FDOT makes an exception to the edge beam evaluation requirements of AASHTO. Section 6A.5.7 allows flat slab longitudinal edge beams to be neglected, provided that curbs or barriers are present, concrete and continuous (no open joints) and the exterior strength per foot meets or exceeds the interior strength per foot (FDOT, 2017). Concrete, continuous barriers on flat slab bridges need not be checked for ultimate moment capacity. Despite the load shedding to the stiffer edge beam, there is no specified demand reductions for interior sections of the slab bridge.

Comparison to Previous Bridge Testing

Physical load testing can be used as a method to improve an analytical load rating and demonstrate ways in which the actual behavior of the bridge differs from the assumed behavior. Several bridge tests on reinforced cast-in-place flat slabs were completed before the start of this work, in the 1980s and 1990s. This first to be completed included the physical testing of three bridges, selected as representative of 12 bridges which required load restrictions based on analytical evaluations (Sessions, 1985). Two of the three bridges were flat slab bridges and the third had multiple span types, both flat slab and T-beam. The bridges had a clear roadway width of 26 feet or 28 feet and an overall width of 30.5 feet or 33.25 feet. Span lengths for the flat slabs ranged from 14 feet to 30 feet. The testing regimen included material sampling and testing and incremental load application with one or two trucks. Measurements were collected using strain gages attached directly to the bridge reinforcing steel. Several conclusions were made based on the load tests of the three bridges, with the intent for the results to be applied to a larger group of similar bridges. The report recommends two exceptions to the AASHTO procedures for narrow cast-in-place structures. The first assumption is that the edge beam will support the entire weight of the curb and railing; distribution of the dead load to the entire width of the bridge is not necessary. Second, distribution of live load is estimated to be uniform throughout the bridge width. Many bridges in the current FDOT inventory are wider than the bridges investigated in this report, but there are still a significant number which could benefit from the recommendations. Of flat slab FDOT bridges, 24% rated according to LRFR and 46% rated according to LFR have a width less than or equal to 28 feet. Also, the report recommends the use of SALOD for conservative ratings of monolithically cast bridges.

Load testing of three solid flat slab bridges in Florida were documented (Arockiasamy & Amer, 1995). A combination of load tests and grillage analyses were used to develop the effective width equation previously discussed. Both the load testing and analysis indicate the AASHTO LRFD effective width is conservative. However, the report notes that effective width based on measured strains are inaccurate for a tested slab bridge which had pre-existing cracks.

Although most bridge tests are non-destructive, intended to improve the rating of an in-service bridge, Azizinamini, et al. (1994) documents the service level test of six three-span continuous bridges and the destructive test of a five span reinforced concrete slab bridge. The bridge selected for the destructive test was constructed in 1938 and decommissioned in 1972, with severe deterioration present at the time of testing. The bridge consisted of two end spans and a three-span continuous section, with lengths of 20 ft. – 31 ft. – 37 ft. – 31 ft. – 20 ft. One end span and one end of the continuous span was tested. The loading pattern simulated two trucks side by side. The effect of multiple truck passages and damage accumulation was investigated by subjecting the spans to several cycles of increasing load before loading to failure. Results of the testing indicated the slab bridge had a high reserve capacity not predicted by analytical load rating. The load rating indicated the bridge could carry 67% of the HS20 truck load, but exhibited linear behavior up to three times the HS20 truck load, and failed at seven times the truck load.

SALOD

Potential Load Rating Improvements

To determine which bridge ratings can be increased, and roughly to what extent they can be improved, the bridge rating list was reduced to only level 3 and 4 postings. Bridge load rating levels relate to how the rating compares to 1.00; the bridges below are either within 9.9% (Level 4) or between 10-19.9% (Level 3) of a 1.00 rating. The following graph was produced to determine what decrease in distribution factor would be required to affect the posting level of any given bridge. The minimum rating for levels 3 and 4 are graphed for LFR bridges and for comparison these ratings are converted to LRFR to show that further distribution factor changes required for an updated load rating to be beneficial.

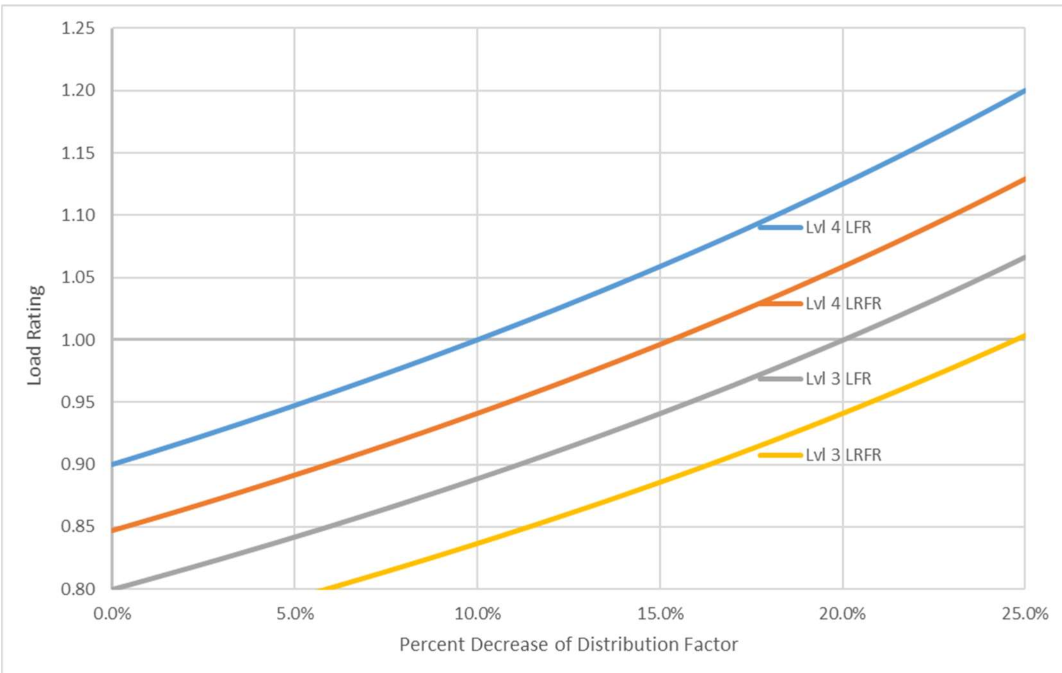


Figure 13: Change in Load Rating as a function of decrease in distribution factor

For the lowest rated level 4 bridge (RF=0.90) a decrease in distribution factor of 10% would bring the load rating to 1.0. Updating this rating factor to LRFR would require a 15.5% distribution factor decrease to prevent a load restriction. Similarly, for a level 3 bridge with RF=0.80 maintaining an LFR rating would require a 20% distribution factor decrease and updating to LRFR would require 24.5% distribution factor increase.

Previous analysis of the bridge inventory shows SALOD appears to have the quickest potential to increase load ratings. Before gaging how much these bridges could benefit from changing the distribution factor calculation method from AASHTO effective width to SALOD the remaining flat slab inventory was further whittled down to include only those coded as having used AASHTO and further filtered to remove the following undesirable characteristics:

- Incorrect distribution factors
- Steel barriers
- Wood barriers
- Dirt Road
- Panel bridges

With a more manageable data set the bridges were then reviewed in more detail and verified on Google Maps Street View to ensure the coded entries matched bridge conditions. During this step, it was discovered that 10 precast panel bridges were included as well as 2 dirt roads in the reduced LFR Level 3-4 rating list. Those bridges were removed from the dataset. Each bridge was also examined in Street View to determine what type of barrier and edge condition each remaining bridge possessed. All appear to have thickened edge beams with either solid barriers or post-type barriers that may contribute to the overall stiffness. These barrier types and edge conditions provided the comfort to increase a load rating based on the stiffness it provides.

The list of bridges was ultimately narrowed down to 23 posted LFR bridges with stiffened edge conditions that may improve with updating the effective width to that provided from SALOD. A single LRFR bridge qualified for this phase as well as 5 ASR (Allowable Stress Rating) bridges with distribution factors in the plausible range.

For the 23 bridges in the data set, bridge plans, inspection reports, and load rating calculations from EDMS were obtained to ensure current conditions would merit a load rating increase and to verify the distribution factors coded in the bridge management system were plausible. The SALOD program places the exterior wheel line 1 foot from the edge of the parapet. Current AASHTO code allows for the wheel to be placed 2 feet from the edge of the parapet. To match current code requirements while using the current SALOD program, a "dummy" width was added into the actual parapet entry in the bridge geometry. This value was kept to 1ft, bringing the SALOD parapet input value to 2ft. Due to how SALOD interpolates between values it would calculate that the truck wheels were off the bridge deck if the "dummy" width is too large. The dummy width was decreased until the errors were no longer present for certain bridge conditions. A simple schematic of the additional parapet width is shown below.

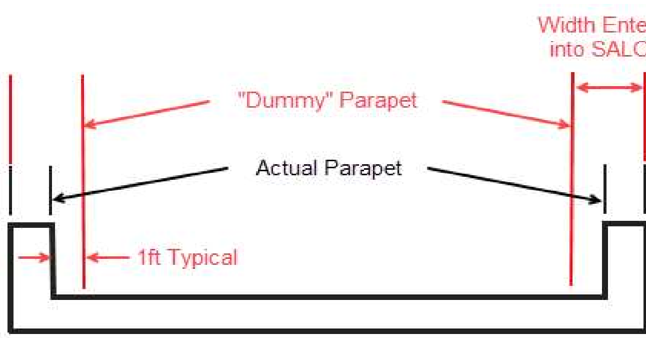


Figure 14 : Schematic of added "dummy" width in parapet entry of SALOD input

To reduce computational effort only the SU4 truck was used due to that being the controlling truck for all 23 bridges. Although SALOD allows for multiple presence reductions to be manually entered, 1% being the minimum and no upper bounds, no reductions were used. In all cases the bridge load rating would be improved using SALOD. For LFR bridges the load rating would be increased by at least 10% with an average increase of 18%. Additionally, 13 of the 23 selected bridges would no longer need to be posted for load restriction with 2 more re-rating at 0.99. Similarly, the 5 ASR bridges increased 9.3% and 3 no longer require posting. The single LRFR bridge increased 16.3% to a 1.14 rating. These increased load ratings result from modifying the effective width only and no other parameter in the load rating. Although the added parapet width benefited the load rating there are other edge conditions that could provide greater benefits such as added sidewalks and thick edge beams. These conditions are not well captured by SALOD as they were not incorporated into the original creation of the software.

580064		LRFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	28.7	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	26	123.336	10.278	0.097295	SU4	6.143	12.286	0.0814	SU4	0.95	16.3%	1.14

280051		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	15	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	29	58.25243	4.854369	0.206	SU4	5.914	11.828	0.169	SU4	0.81	17.9%	0.98

280052		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	15	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	29	58.25243	4.854369	0.206	SU4	5.914	11.828	0.169	SU4	0.81	17.9%	0.99

090015		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	15	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	29	58.25243	4.854369	0.206	SU4	5.914	11.828	0.169	SU4	0.92	17.9%	1.12

070037		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	15	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	29	58.25243	4.854369	0.206	SU4	5.914	11.828	0.169	SU4	0.92	17.9%	1.12

070035		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	15	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	29	58.25243	4.854369	0.206	SU4	5.914	11.828	0.169	C3	0.92	17.9%	1.12

300014		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	33	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	29	142.8571	11.90476	0.168	SU4	6.665	13.33	0.150	SU4	0.85	10.7%	0.95

070011		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	27	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	30	133.3333	11.11111	0.180	SU4	6.605	13.21	0.151	SU4	0.83	15.9%	0.99

590028		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	28	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	33.5	134.0782	11.17318	0.179	SU4	6.742	13.484	0.148	SU4	0.97	17.1%	1.17

390023		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	20	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	25.5	122.449	10.20408	0.196	SU4	5.71	11.42	0.175	SU4	0.99	10.6%	1.10

340012		LFR DF				Controlling SALOD Eff. Width				SALOD Change		
Span Len	20	Truck in	Truck ft	DF	Control	Wheel	Truck	DF		SU4 RF	%inc	New RF
Span wid	25.5	122.449	10.20408	0.196	SU4	5.71	11.42	0.175	SU4	0.92	10.6%	1.03

Figure 15: Sample of SALOD Analysis of Selected Bridges

A current level 4 rating is likely to improve enough with only updating to SALOD that a posting is not required. If the above criteria are relaxed to also include steel barriers then it is likely that 31 out of the 160 posted flat slab bridges may be immediately impacted by using SALOD alone. For LFR bridges the greater increases from SALOD suggests that an additional 14 level 3 bridges could also be removed from load restriction. This would bring the potential improvement number to 45 out of 160, or 28% of the posted flat slab brings.

SALOD Methodology

SALOD calculates effective widths based on seven finite element analysis runs of flat slab bridges with different aspect ratios (width:length) ranging from 0.5 to 3.0. Results from those finite element analysis runs are stored as influence surface arrays within the program. The only influence surfaces included are for moment at midspan and therefore the SALOD program only considers moment at midspan for the effective width calculation.

For each different aspect ratio considered in the program development, 0.5, 0.75, 1.0, 1.5, 2.0, 2.5 and 3.0, six arrays are stored. Each array has 66 values for width locations at increments equal to 1/10 of the width and length locations at increments equal to 1/10 of the span. The entire width of the bridge is represented but only half of the span length is represented. Each array contains coefficients for the moment at width location 1, 2, 3, 4, 5 or 6 for a load placed at any location in the grid, denoted by width location 1-1' and length location 1-6. Length location 6 corresponds with midspan of the bridge. The bridge and loading are assumed to be symmetrical, so only locations 1-6 along the width at midspan need to be considered and locations beyond 6 can be ignored for effective width calculation. Location 1 is at the edge of the bridge and location 6 is at the middle. Graphical representation of the influence surface arrays is shown in Figure 16. The circle denotes the location for which an array is stored. The array contains a coefficient for moment at that location due to load at each location marked with an arrow.

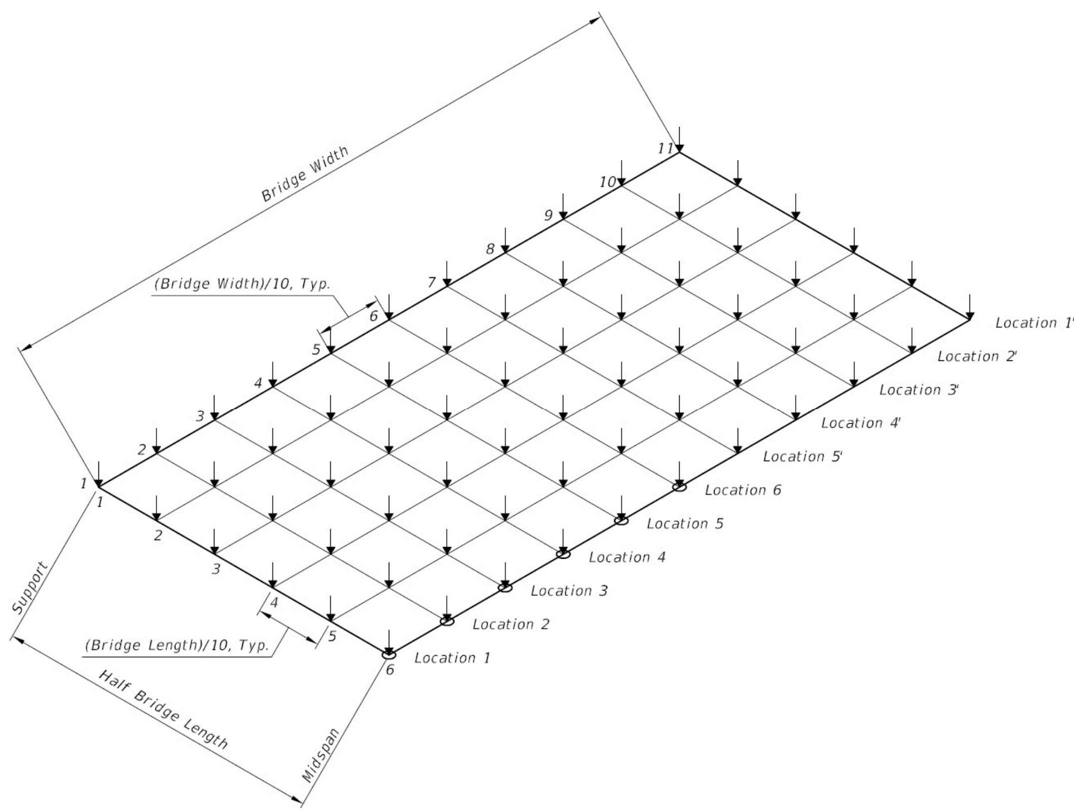


Figure 16: SALOD Influence Surface

An example SALOD influence surface array is presented in Table 2 for the 0.50 aspect ratio. The SALOD program interpolates from the stored influence surface arrays to determine arrays that are specific to the aspect ratio for the subject bridge. Then, an effective width is calculated for locations 1-6 with the truck placed longitudinally to produce maximum moment and transversely at increments of 1 foot.

Table 2: SALOD Data FSLAB050 (0.50 Aspect Ratio)

Location 1	1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	0	0	0	0	0	0	0
2	-0.104	-0.1044	-0.1048	-0.1049	-0.1046	-0.1035	-0.1017	-0.0992	-0.0963	-0.0931	-0.0899
3	-0.2128	-0.2132	-0.2138	-0.2138	-0.2118	-0.2078	-0.2022	-0.1956	-0.1885	-0.1814	-0.1745
4	-0.3352	-0.3347	-0.3369	-0.3332	-0.3238	-0.311	-0.297	-0.2831	-0.27	-0.2578	-0.2466
5	-0.4699	-0.515	-0.503	-0.4693	-0.4332	-0.4007	-0.3727	-0.349	-0.3289	-0.3117	-0.2966
6	-1.0903	-0.8104	-0.648	-0.5516	-0.4872	-0.4399	-0.4034	-0.3744	-0.3509	-0.3314	-0.3147

Location 2	1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	0	0	0	0	0	0	0
2	-0.1044	-0.104	-0.1036	-0.1031	-0.1026	-0.1018	-0.1007	-0.0991	-0.0971	-0.0947	-0.0923
3	-0.2137	-0.2123	-0.2105	-0.2085	-0.2068	-0.2043	-0.2007	-0.1961	-0.1908	-0.1853	-0.1798
4	-0.3371	-0.3336	-0.3248	-0.3211	-0.3157	-0.3073	-0.2969	-0.2858	-0.275	-0.2648	-0.2554
5	-0.5113	-0.4672	-0.4716	-0.4567	-0.43	-0.4022	-0.3771	-0.3556	-0.3374	-0.322	-0.3087
6	-0.7906	-0.8037	-0.664	-0.5616	-0.4942	-0.4468	-0.411	-0.3832	-0.3611	-0.3432	-0.3282

Location 3	1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	0	0	0	0	0	0	0
2	-0.1045	-0.1032	-0.1021	-0.1012	-0.1007	-0.1003	-0.0998	-0.0991	-0.0979	-0.0965	-0.0949
3	-0.213	-0.2097	-0.2064	-0.2041	-0.2022	-0.2009	-0.1992	-0.1967	-0.1934	-0.1896	-0.1857
4	-0.3342	-0.3236	-0.3191	-0.3099	-0.3066	-0.3029	-0.2969	-0.2893	-0.2811	-0.2731	-0.2656
5	-0.4952	-0.4684	-0.4255	-0.4337	-0.4254	-0.4057	-0.3843	-0.365	-0.3486	-0.3348	-0.3233
6	-0.6326	-0.6592	-0.7123	-0.6015	-0.5174	-0.4627	-0.4247	-0.3966	-0.3751	-0.3583	-0.3448

Location 4	1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	0	0	0	0	0	0	0
2	-0.1041	-0.1024	-0.1009	-0.0998	-0.0992	-0.099	-0.0989	-0.0989	-0.0987	-0.0982	-0.0975
3	-0.2116	-0.2069	-0.2034	-0.2005	-0.1988	-0.1977	-0.1974	-0.197	-0.1959	-0.1941	-0.1919
4	-0.3279	-0.3182	-0.3086	-0.3052	-0.2977	-0.2968	-0.2958	-0.2927	-0.288	-0.2826	-0.2773
5	-0.4595	-0.4516	-0.4317	-0.3949	-0.4096	-0.4076	-0.3937	-0.3777	-0.3632	-0.3512	-0.3414
6	-0.5384	-0.5549	-0.5991	-0.6683	-0.5697	-0.4951	-0.4482	-0.4169	-0.3946	-0.3784	-0.366

Location 5	1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	0	0	0	0	0	0	0
2	-0.1033	-0.1016	-0.1001	-0.099	-0.0983	-0.0981	-0.0983	-0.0987	-0.0992	-0.0996	-0.0999
3	-0.2085	-0.2046	-0.2009	-0.1983	-0.1964	-0.1957	-0.1958	-0.1969	-0.1979	-0.1982	-0.1981

4	-0.3172	-0.3119	-0.3045	-0.297	-0.2959	-0.2911	-0.2931	-0.295	-0.2948	-0.2929	-0.2903
5	-0.423	-0.4239	-0.4224	-0.4085	-0.3777	-0.3981	-0.4015	-0.3927	-0.3815	-0.3715	-0.3637
6	-0.4749	-0.487	-0.5139	-0.5685	-0.6465	-0.5556	-0.4877	-0.4471	-0.4215	-0.4048	-0.3933

Location 6	1	2	3	4	5	6	7	8	9	10	11
1	0	0	0	0	0	0	0	0	0	0	0
2	-0.1018	-0.1007	-0.0996	-0.0987	-0.098	-0.0978	-0.098	-0.0987	-0.0996	-0.1007	-0.1018
3	-0.2038	-0.2018	-0.1994	-0.197	-0.1956	-0.1949	-0.1956	-0.197	-0.1994	-0.2018	-0.2038
4	-0.304	-0.3031	-0.3006	-0.2957	-0.2909	-0.2927	-0.2909	-0.2957	-0.3006	-0.3031	-0.304
5	-0.3907	-0.396	-0.4024	-0.4062	-0.3977	-0.3722	-0.3977	-0.4062	-0.4024	-0.396	-0.3907
6	-0.4286	-0.4397	-0.459	-0.4935	-0.5552	-0.6399	-0.5552	-0.4935	-0.459	-0.4397	-0.4286

Current Testing Program

Overview

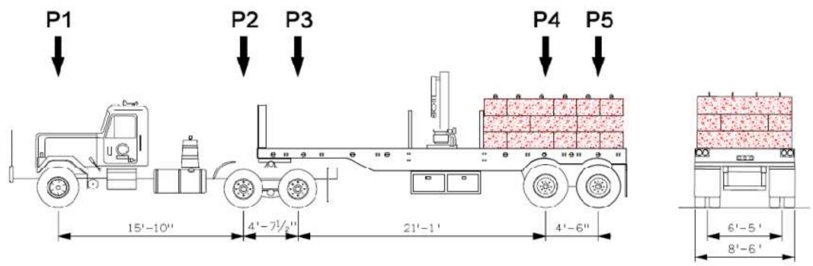
Previously completed research and bridge testing proved the AASHTO LRFD and Standard Specifications equations for effective width are conservative. To verify and quantify the level of conservatism, several bridge tests were completed for this study. Physical structural testing is more accurate than analytical predictions and was warranted for this study because the findings will apply to many bridges throughout Florida. Five bridge tests were completed to measure actual bridge behavior. Results were compared to predictions using codified effective widths, SALOD effective widths and analyses assuming the whole bridge width is effectively resisting loads. Conclusions varied for the five bridges, indicating the same solution may not provide reliable predictions for all bridges.

All five of the bridges tested are cast-in-place concrete flat slab bridges. Two were tested before this study was initiated due to low load ratings, but the test results are related and valid to this study. Two bridges were tested twice because the data collected during the original test was outdated and not available for this study. The bridges have varying features, representing many different types of bridges in the Florida inventory. Differing characteristics of the bridges include barrier type, aspect ratio, length, thickness, age and past widening. Three bridges are located in FDOT district 2 and one each is located in FDOT district 3 and 5. The table below contains information about each of the bridges tested.

Bridge	District	Dates of Interest			Span Length (ft)	Thickness (in.)	Aspect Ratio (width : length)
		Construction	Widening	Test			
730043/ 730044	5	1957	N/A	4/24/2013	20	12	1.69
280023	2	1954	1999	3/3/2005	15	10.5	3.70
470030	3	1948	N/A	6/21/1990 and 9/24/2015	15	11.5	1.92
260038	2	1957	1984	1/15/1992 and 5/4/2016	15	10.5	4.8
340048	2	1962	2003	6/29/2016	33	18	1.18

Each of the bridges was instrumented to collect data concerning the bridge behavior during testing. Typically, instrumentation consisted of both strain and deflection gages. Strain gages were oriented to capture longitudinal strain and placed incrementally across the width of the bridge at midspan. Two to three additional gages were placed transversely along the width of the bridge to capture transverse strain at select locations. For bridges which had been widened, strain was compared on either side of the widening joint. Strain gages were also installed on the barrier to capture longitudinal strain at discrete locations in the height of the barrier, therefore capturing longitudinal moment resistance contribution from the barrier. Deflection was measured at approximately the same location as strain gages and also at the support locations. The transverse gage spacing for longitudinal strain and deflection was controlled by data acquisition limits on the number of gages which could be recorded at any one time. Therefore, narrow bridges had a higher gage concentration (lower spacing) than wider bridges.

Loading of the bridges was done using the two FDOT Structures Research Center load trucks. The trucks are designed to be loaded with standard blocks which weigh approximately 1 ton each, allowing for incrementally loading the bridge during the test. The test generally started with a minimum load of 6 to 18 blocks and concluded with a maximum load of 30 to 42 blocks. Figure 17 indicates the axle loads corresponding to the number of blocks. For some load tests, the maximum weight was sufficient to consider the test a proof test. For other tests, a lower maximum weight means the test is considered a diagnostic test (AASHTO, 2018). Longitudinally, the trucks were placed to maximize moment at midspan of the bridge. Transversely, the trucks were placed at multiple locations, to maximize load effects at the barrier, the middle of the bridge, or the widening joint. Either one or both trucks were placed on the bridge for each measurement.



AXLE WEIGHTS OF LOAD TESTING TRUCKS

Rear Truck Loading – Steel Blocks

Description of Loads	Front Axle	Front Tandem		Rear Tandem	
	P1 (kips)	P2 (kips)	P3 (kips)	P4 (kips)	P5 (kips)
Empty	10.60	9.59	9.59	9.06	9.06
6 Blocks	10.24	10.56	10.56	14.1	14.1
12 Blocks	10.34	10.72	10.72	19.64	19.64
18 Blocks	10.08	10.47	10.47	26.28	26.28
24 Blocks	10.12	10.77	10.77	32.36	32.36
30 Blocks	9.98	11.66	11.66	37.48	37.48
36 Blocks	10.04	11.60	11.60	43.81	43.81
42 Blocks	10.10	12.52	12.52	48.45	48.45
48 Blocks	10.36	12.25	12.25	54.21	54.21

Figure 17: Axle Weights of Load Testing Trucks

Bridge tests were typically conducted at night with a full or partial closure lasting 4-6 hours. The typical operation consisted placing one or both trucks on the bridge and then recording gage data for a period of 30 to 60 seconds at a rate of 1 Hz. Then the truck(s) were moved and another recording was taken. Zero readings taken with no load on the bridge were collected before and intermittently during the test.

Each data recording consisted of 30-60 gage readings, which were averaged to obtain a single reading for each gage and each truck position. It was then adjusted with the zero to remove gage drift or error. The data was primarily analyzed visually, using graphs with parameters such as applied moment, strain and deflection and similar gages grouped together.

Data collected from the bridge tests was also compared with an analytical rating of each bridge. The analytical rating used an effective width from the applicable bridge code or SALOD, or the full bridge width was used. An individual report and load rating was prepared for each bridge and sent to the FDOT Maintenance Office. In general, the effective widths calculated per either AASHTO code were determined to be conservative compared to the measured bridge behavior. (AASHTO, 2002) (AASHTO, 2017) One exception was bridge 260038, which is particularly wide in comparison to the other bridges tested. Several bridges would have required posting if the codified effective width was used, but load testing allowed the use of a wider effective width and therefore posting was avoided. Readings from several bridges, 730043 and 470030, indicated that the entire bridge width was effective in carrying load. A comparison of findings from the bridges is provided in more detail in the following section.

Load Test Comparison to AASHTO and SALOD

The behavior measured during load tests of five bridges was compared to the predicted behavior based on AASHTO and SALOD. Behavior of the flat slabs is quantified here in terms of effective width and maximum moment or micro-strain. Based on past experiences and the literature review, the AASHTO effective widths are presumed to be very conservative compared to actual behavior. Effective widths calculated from SALOD are presumed to be conservative, but more reasonable than AASHTO. The purpose of the comparison was to measure the difference between predicted behavior and measured behavior and determine if SALOD provides a more reasonable prediction of behavior than AASHTO. The tested bridges have aspect ratios ranging from roughly 1.2 to 4.8 (width:length) and varying edge conditions as shown in Figure 18. Their characteristics accurately represent the range of flat slab bridges in FDOT inventory and their measured behavior can be considered typical flat slab behavior.

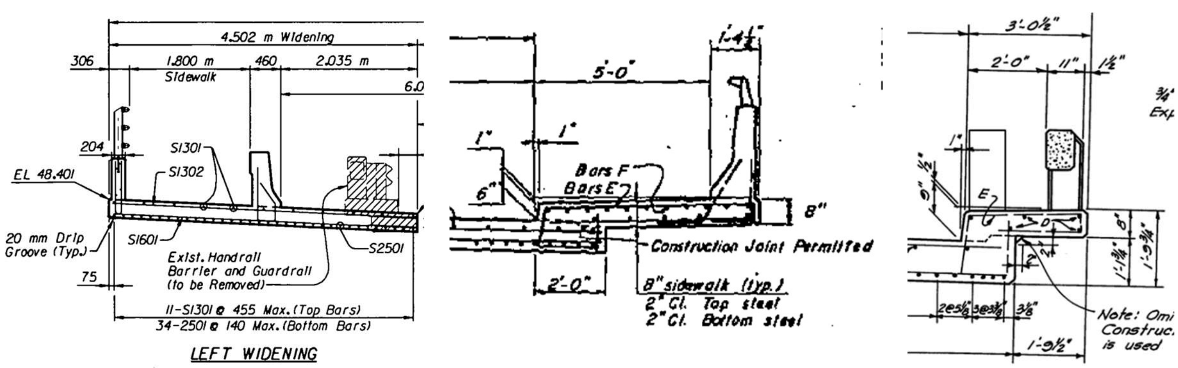


Figure 18: Example of Different Edge Conditions on the Tested Bridges

The effective width for each of the tested bridges was calculated using SALOD as discussed in the previous section, Potential Load Rating Improvements. Only the SU4 truck was used as it is the controlling load case for all subject bridges. Effective widths were also calculated according to AASHTO LRFD BDS (AASHTO, 2017). The resulting effective widths per SALOD and AASHTO were used to calculate the predicted microstrain with a MathCAD worksheet, using only the controlling load test truck for each bridge. Bridge test data selected for comparison was chosen based on the controlling load case as well. Certain load cases produced a larger individual peak while some load cases produced a larger average of strains. When this conflict occurred, the load case producing the larger average strains was chosen for this analysis.

Two different methods were used to analyze the load test data. The first method uses applied moment and measured strain to calculate an effective width and is based on Equation 9, the effective width equation used for SALOD (Hays & Hachey, 1984). The simple beam moment is the moment that would be calculated in a beam with the same span length and loading as the bridge. In effect, it is the total moment applied to the bridge. For the work done by Hays & Hachey, the FEM moment is the moment in the node of interest in the FEM. A 1-foot width is implied. The 2 in the denominator is because effective width calculated by SALOD is for a wheel (half axle) load, not the full truck load. For calculating the effective width of an entire lane, the 2 is removed. Applying Equation 9 to the load test results, the FEM moment is replaced by an equation for the moment measured in the bridge. Assuming that the plane cross-section remains plane and employing Equation 10 for the extreme fiber stress and Equation 11 for the stress-strain relationship, the moment measured during the test is represented by Equation 12, where S is the section modulus for a 1-foot width of the bridge, ε is the maximum strain measured during the test and E is the concrete modulus of elasticity. The drawback to this method of analysis is that the concrete modulus of elasticity is not well known. If the modulus is underestimated, as is typical, the resulting effective width will be larger than it should be. The results from this analysis method are presented in Figure 20 and Figure 21 as "Load Test." The strain value shown in the graphs is the maximum measured micro-strain for the loading condition considered and the width shown is the effective width calculated using Equation 13 and the maximum measured strain.

$$Effective\ width = \frac{simple\ beam\ moment}{2\ x\ FEM\ moment} \times \frac{1}{12}$$

Equation 9: Effective Width (Hays & Hachey, 1984)

$$\sigma = {}^{M}\!/_{S}$$

Equation 10: Extreme Fiber Stress

$$\sigma = \varepsilon \times E$$

Equation 11: Stress-Strain Relationship

$$M = S \times \varepsilon \times E$$

Equation 12: Measured Moment

$$Effective\ Width\ = \frac{simple\ beam\ moment}{S \times \varepsilon \times E}$$

Equation 13: Effective Width Based on Applied and Measured Moment

A second method used to analyze the load test results incorporates the relative section properties across the bridge width. Herein, it is called the Section Modulus method. This calculation produces effective widths based on the section properties of bridge segments that each gage location represents. The overall bridge cross section is divided based on the strain gage spacing as shown in Figure 19. The section modulus is then calculated for each representative section. Based on those properties and strain data, a weighted average is taken and converted into an effective width of bridge resisting the load. In certain bridge tests, not all strain gages produced usable data: in these cases, spacing on the plans was still maintained as the width of the gage area. The sections with no functioning usable strain data were skipped. The equation used to calculate effective width based on this method is shown in Equation 14. The advantage of this processing method over the previously discussed method is that the concrete modulus of elasticity is not used and therefore results are not skewed by the underestimation of concrete modulus. A secondary advantage is that multiple strain values from the load test are used to calculate the effective width, whereas the previous method only uses the maximum strain measured. The results from this section modulus method are presented in Figure 20 and Figure 21 as "Sec. Mod." The effective width shown in the graphs was calculated using Equation 14 with strains measured during the tests and section moduli calculated based on construction drawings. The resulting effective widths were used to calculate micro-strain with a MathCAD worksheet, employing the same method used to calculate micro-strain for this method as was used to calculate micro-strain for effective widths calculated per AASHTO and SALOD.

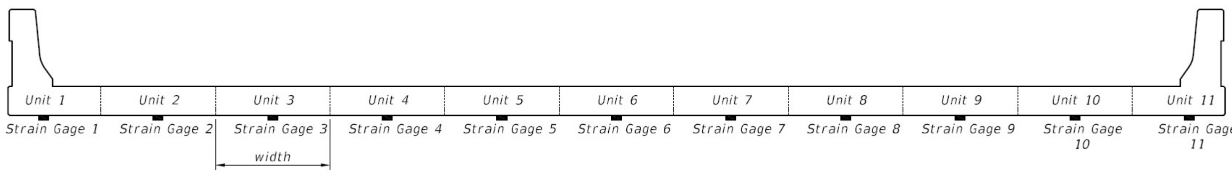


Figure 19: Divisions for Section Modulus Method

$$Effective\ Width\ for\ unit\ n = \frac{width\ x\ \sum \varepsilon_i x\ S_i}{\varepsilon_n x\ S_n}$$

Equation 14: Effective Width Based on Section Modulus

AASHTO and SALOD are not specifically tailored to account for the added stiffness that different edge conditions can provide a flat slab bridge. However, barrier and curb stiffness is inherently accounted for in the strain data from the bridge testing. Using the Section Modulus method also takes the added stiffness of the barrier into account. Each gage section for each bridge is individually calculated therefore the specific barrier geometry is used in each bridge example. This difference makes the four selected methods to represent effective widths complement each other well enough to make meaningful comparisons.

Graphs presented in Figure 20 and Figure 21 are in increasing order of aspect ratios. From top to bottom in Figure 20, the aspect ratios are: 1.2, 1.7, 1.9. Aspect ratios greater than the operating range of SALOD are found on the following page, they are 3.7 and 4.8. The effective width for AASHTO will generally be smaller than that provided by SALOD and produces a higher predicted strain value. Both methods are far more conservative than the actual behavior of the bridge. The graphs show how the load testing data, and the Section Modulus method, generally can produce a much larger effective width than that provided by AASHTO or SALOD. Although the Section Modulus method was not as consistent as would have been desired, it did in produce a width that was more generous than those from SALOD or AASHTO. This method would be impractical to implement on a typical load rating due to it requiring test data or some good estimation of strain across the slab cross section to be used correctly. For the bridges within the formulated range of SALOD things are more as expected with AASHTO being the smallest width with highest predicted strain, SALOD being slightly wider with a lower predicted strain, and the actual test data showing a much larger width and much lower measured strains. For bridge 340048, the effective width calculated based on SALOD is wider than the effective width calculated from the load test data using the section modulus method. But, the corresponding strain is still very conservative compared to the strain measured during the load test. SALOD averaged an increase of 28% in effective width over AASHTO for these tested bridges.

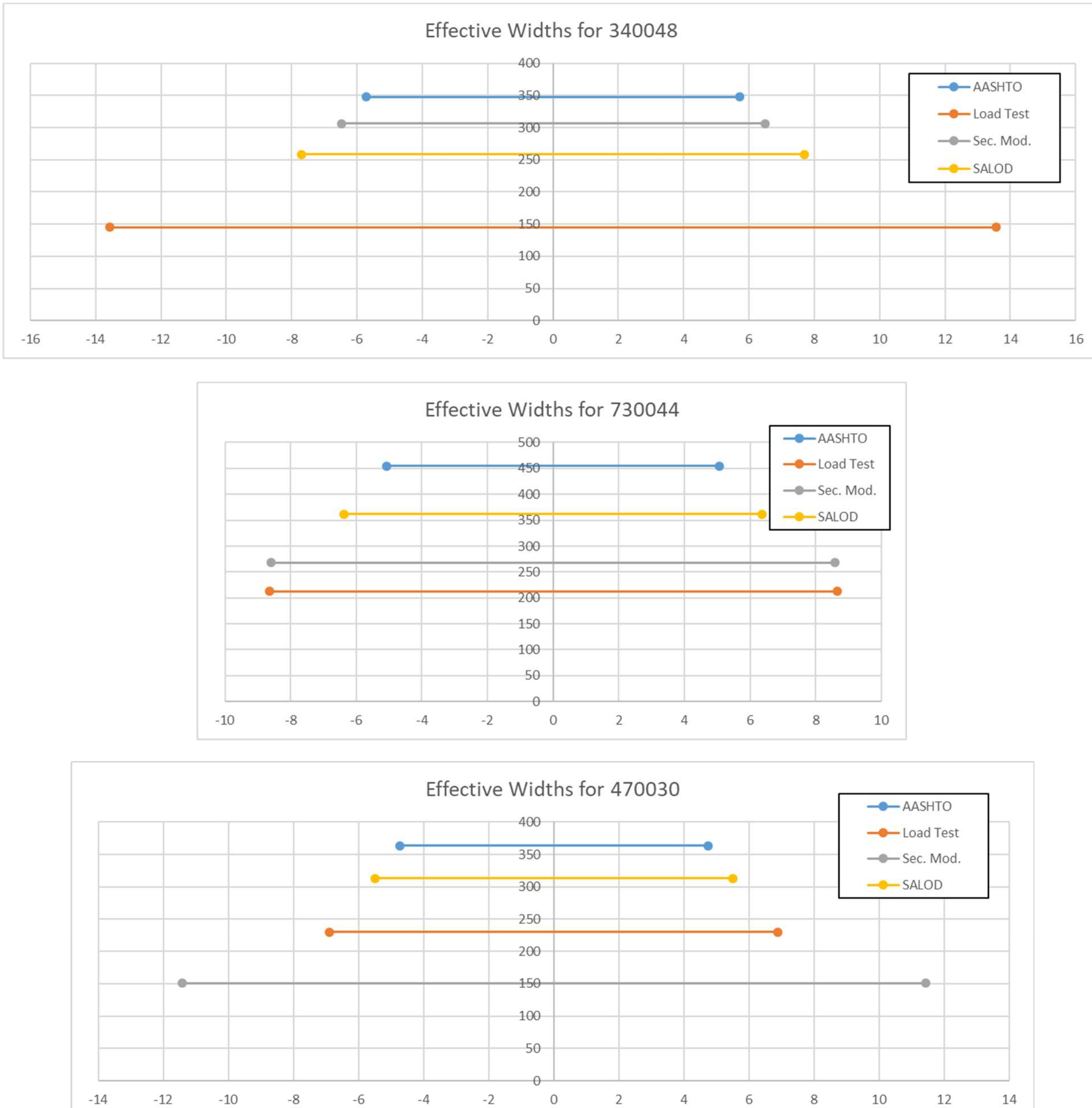


Figure 20: Graphs of Different Effective Width for Tested Bridges within the Range of SALOD

Overall, SALOD works very well for bridges that fall within its formulated aspect ratios of 0.5-3.0. Once the aspect ratio begins to drift far from SALOD's upper limit effective widths become unconservative and should not be used. This limitation should come as no surprise as the program was designed to interpolate between 2 values within the range of finite element models used to create the influence surfaces at the basis of SALOD. Short very wide bridges violate this underlying modeling done to create SALOD. AASHTO, although overly conservative at times, still provides a more desirable width for these cases and conservative results for the bridge. It also appears that as the 3.0 limit of SALOD is surpassed the results continue to grow more unconservative the greater the aspect ratio becomes. This is seen in the comparison of Bridges 280023 and 260038. SALOD's width produces strains roughly equal to the Load Test width for the first bridge with an aspect ratio equal to 3.7 while severely underestimating the strains for the much wider bridge with an aspect ratio equal to 4.8.

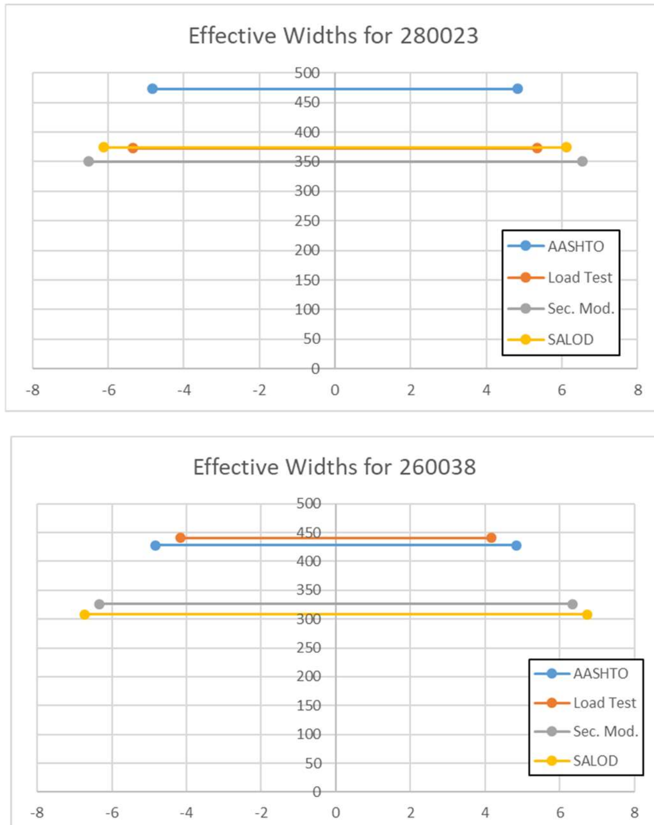


Figure 21: Graph of Different Effective Widths for Tested Bridges Outside the Range of SALOD

Revised Program Based on SALOD Methodology

Using the SALOD methodology for future flat slab load ratings will be the best way to improve load ratings and avoid postings for many flat slab bridges. The SALOD method provides an effective width that is conservative compared to load testing results, but less conservative than the effective width calculated using AASHTO LRFD Bridge Design Specifications. Although the effective width calculated using SALOD is less conservative than the effective width calculated using equations in AASHTO, it is still an acceptable method per AASHTO. The Manual for Bridge Evaluation, section 6A.3.3 allows the use of refined methods of analysis for "bridges that exhibit insufficient load capacity when analyzed by approximate methods" (AASHTO, 2018). SALOD was developed and is based on isotropic plate models, which meet the requirements of AASHTO refined methods of analysis (AASHTO, 2017).

Use of SALOD has fallen out of favor in recent years due to the "black box" nature of the program. The program interface only allows the user to see input and output values. Intermediate calculations done by the program to calculate an effective width are not visible to the user and therefore cannot be checked. And, unless the program user has reviewed the report on original development of the program, they would not understand the logic used to generate effective widths (Hays & Hachey, 1984). To address those problems, the SALOD methodology has been incorporated into a MathCAD worksheet so that underlying calculations can be easily reviewed by the user. Review of the SALOD Methodology section of this report will aid the program user in understanding how the program generates an effective width. The methodology used in the MathCAD program matches the SALOD program with a few exceptions, which are appropriate considering current design practice and code changes which have occurred since the original development of SALOD.

There are several differences between the process followed by the MathCAD program and SALOD. The first difference simplifies the program based on typical analysis processes. For a second lane placed on the bridge, SALOD will re-analyze for the maximum longitudinal moment in order to place the truck longitudinally. This is unnecessary as, typically, only one type of truck needs to be analyzed at a time. In the case of multiple of the same truck placed on the bridge, they can all be placed at the same longitudinal position because the longitudinal position that produces maximum moment will be the same for all trucks. This change simplifies the computational effort of the program and is sufficient for load rating according to AASHTO requirements. For the rare case in which two different trucks need to be loaded on the bridge, the user could input each truck as a user defined truck and then use the smaller or average of the resulting effective widths. Or the user could input the two trucks together as a single user defined truck and modify the MathCAD programming to recognize the user defined truck as a truck with 4 axles.

The SALOD program uses a transverse wheel spacing and standard truck gage that is not in line with current AASHTO. See Figure 5.2 in (Hays & Hachey, 1984) and Figure 26 for the SALOD transverse truck placement. The truck gage spacing used by the SALOD program is 6'-4", whereas the current gage spacing required BY AASHTO is 6'-0" (AASHTO, 2017). For the MathCAD program a gage spacing of 6'-0" will be used. Case 3 and 4, used in SALOD and shown in Figure 26, will not be included in the MathCAD program. By inspection, case 1 will produce maximum effects at the exterior edge of the bridge and case 2 will produce maximum effects at the interior of the bridge. Cases 3 and 4 are extraneous and unnecessary. Two additional cases are added to the MathCAD program to address bridges with clear width between 20 and 24 feet. AASHTO requires that bridges with clear widths in that range accommodate 2 lanes, with each lane having a width equal to half the clear width (AASHTO, 2017). Figure 27 shows the two cases to address those bridge widths along with SALOD cases 1 and 2, modified to have the current AASHTO truck gage spacing of 6'-0".

The final difference between SALOD and the MathCAD program concerns movement of the truck transversely across the bridge. In order to determine the controlling effective width, the truck is moved transversely across the bridge and the effective width is analyzed for each transverse movement. SALOD moves the truck transversely across the bridge in increments of 1 foot. For simplicity and because bridge widths vary greatly, a set distance will not be used for incremental transverse movement of the trucks in the MathCAD program. Instead, the truck will be moved transversely by 25 divisions of the allowable truck movement shown in Figure 27. This will result in increased accuracy for narrower bridges than wider bridges, but the difference between this method and the current SALOD method was measured to be 0% to 1% for typical bridges.

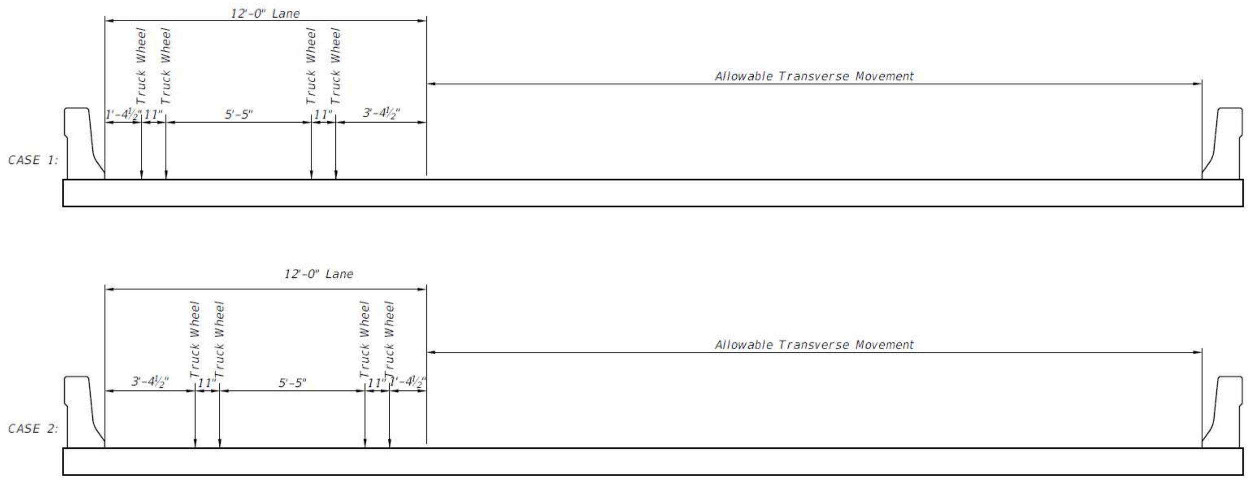


Figure 22: SALOD Transverse Truck Placement, 1 Lane

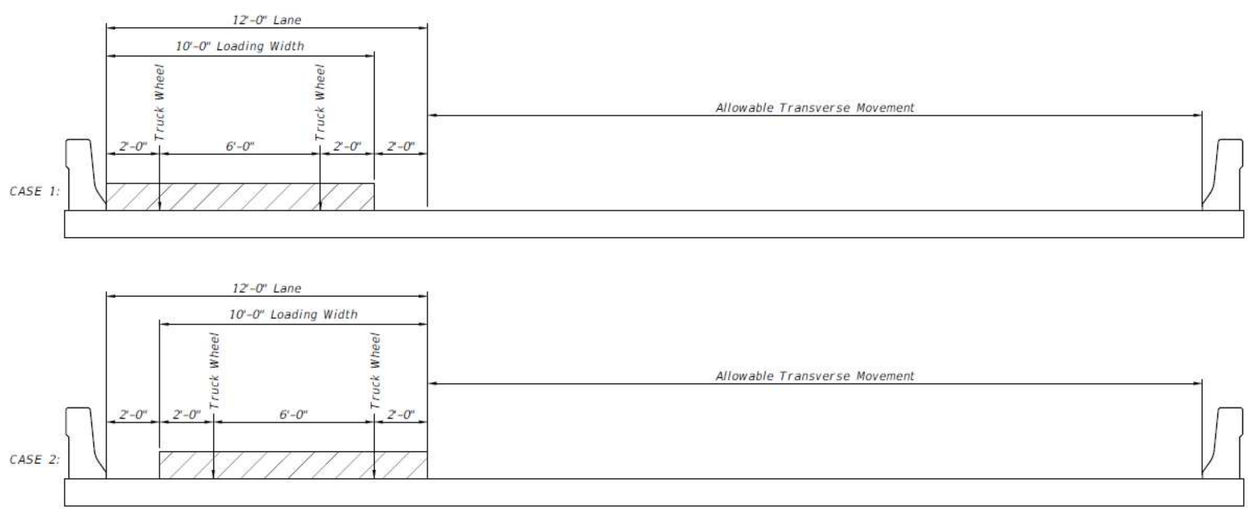


Figure 23: MathCAD Program Transverse Truck Placement, 1 Lane

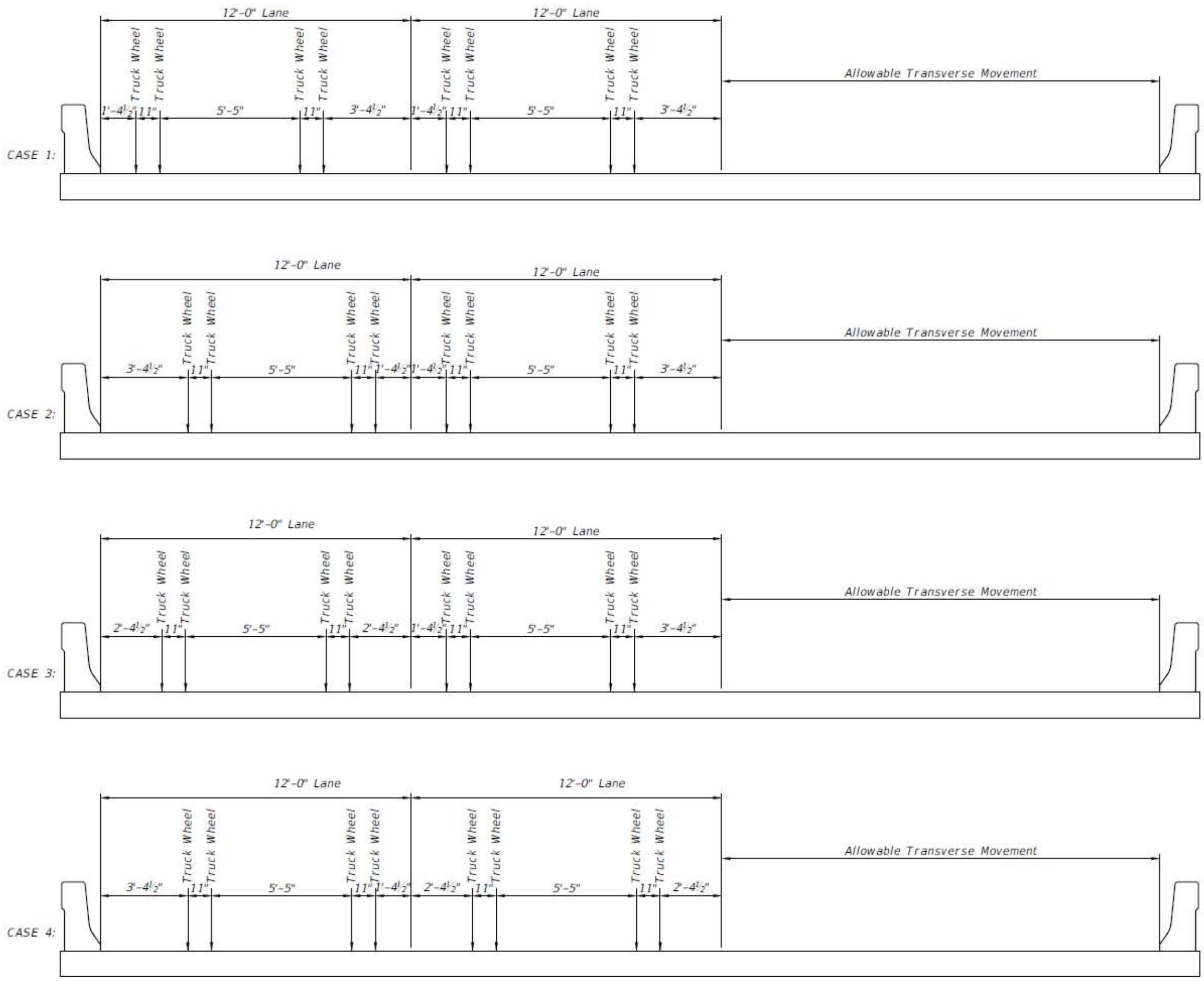


Figure 24: SALOD Transverse Truck Placement, 2 Lanes

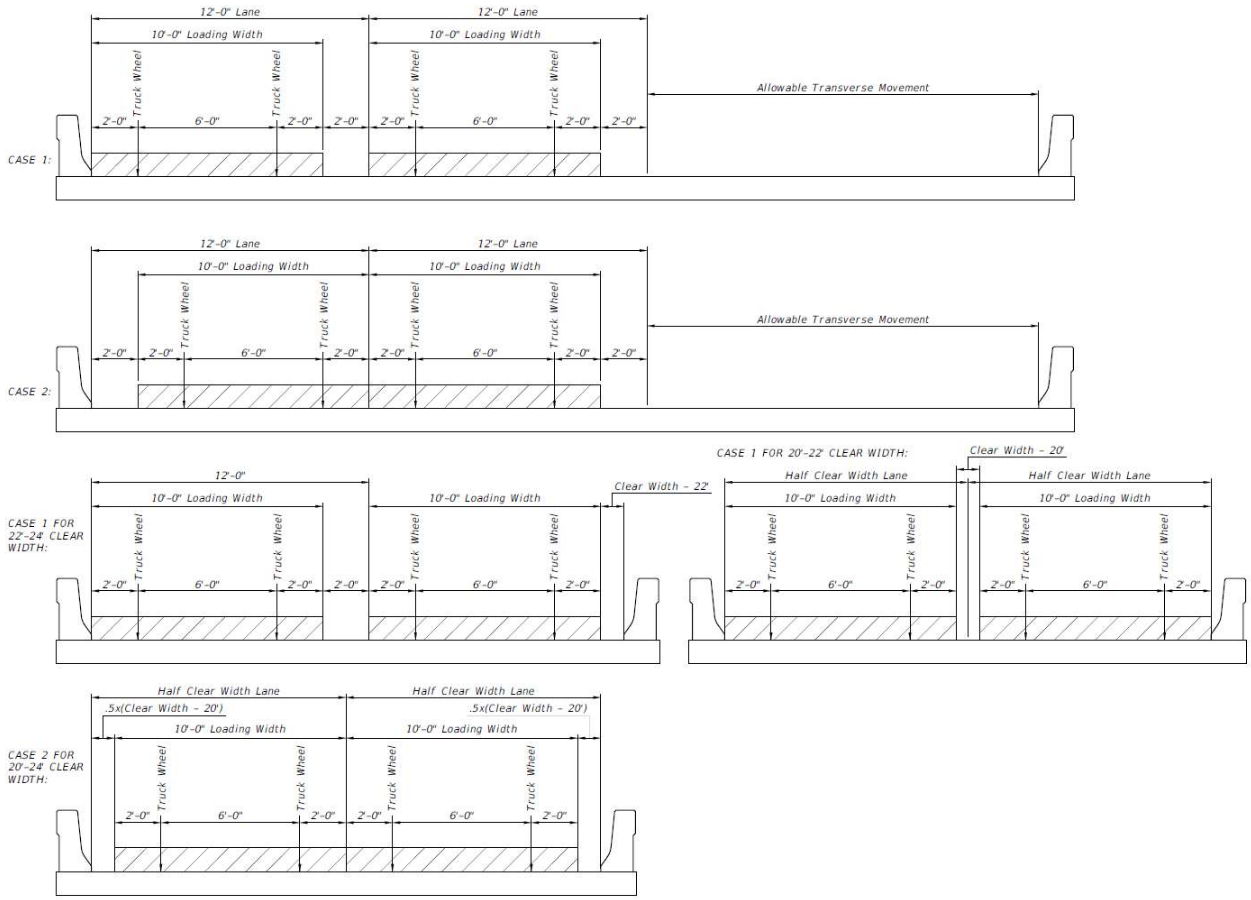


Figure 25: MathCAD Program Truck Placement, 2 Lanes

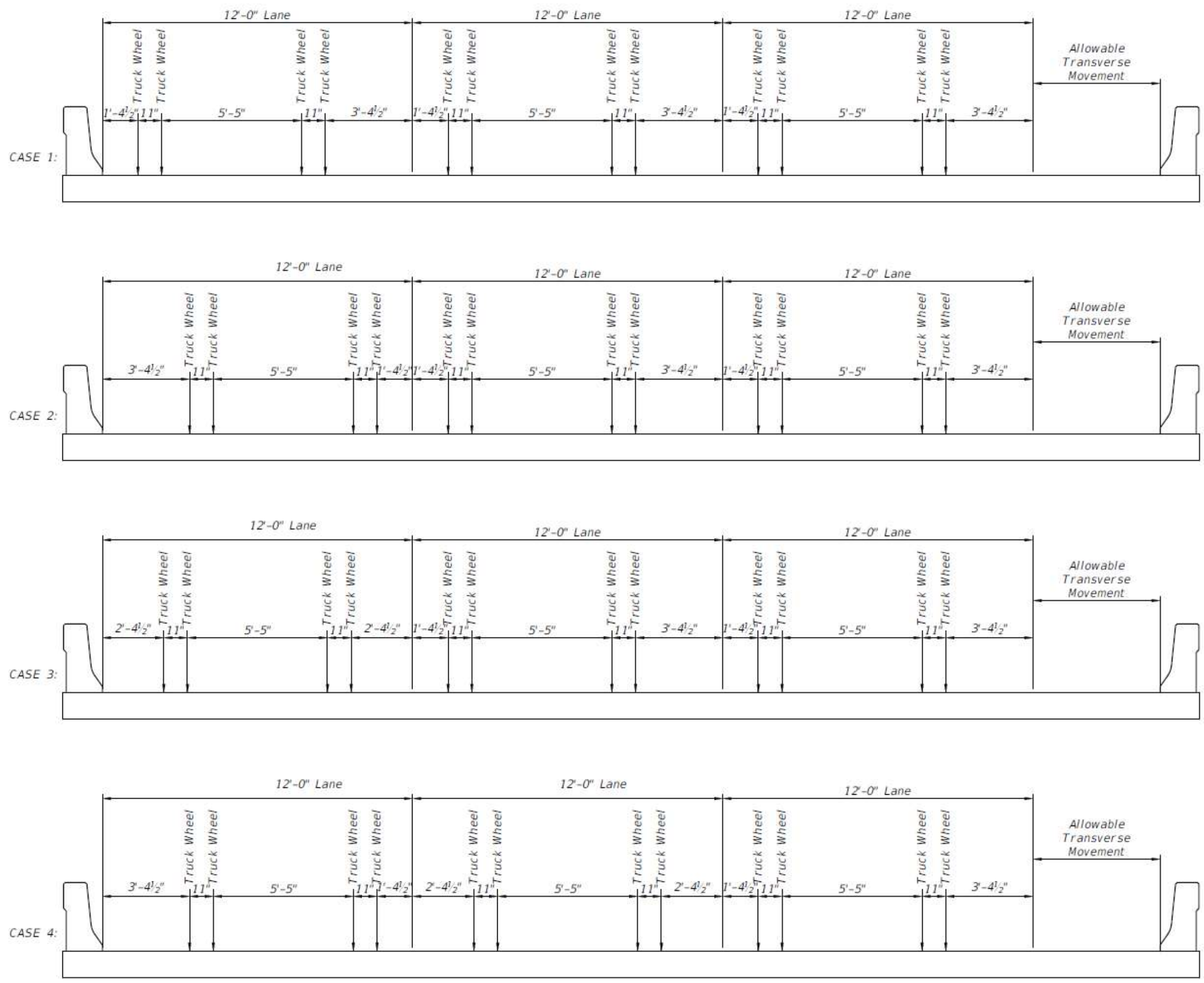


Figure 26: SALOD Transverse Truck Placement, 3 Lanes

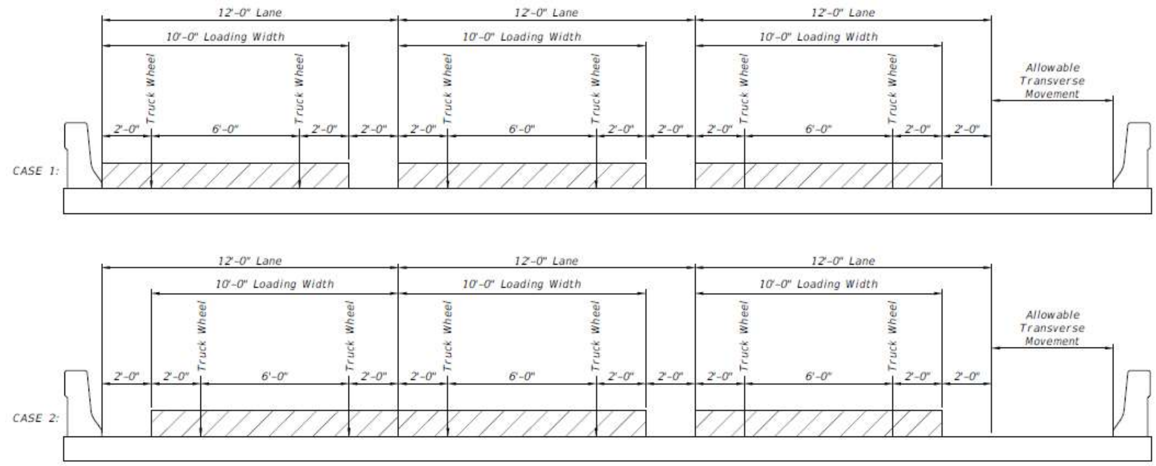


Figure 27: MathCAD Program Transverse Truck Placement, 3 Lanes

Lane loads are not addressed in the SALOD program and will not be addressed in the MathCAD program. The interpolated influence surface methodology used in SALOD is not readily adaptable for use with distributed loads. In the absence of a program to calculated effective width, the lane load can be applied to a unit width of bridge or a reasonable effective width based on engineering judgement. The effective width used for the lane load may be slightly conservative compared to actual bridge behavior, but the effect on typical bridge load ratings and postings will be minimal. This effort addressed simple span slab bridges, all of which have spans less than 200 feet. For those bridges, the lane load is only considered for the HL-93 rating. The lane load for the permit truck does not apply per FDOT requirements (FDOT, 2017). If better accuracy is required for the HL-93 load rating for these simple span flat slab bridges, further investigation could be performed.

The analysis method for SALOD-based MathCAD program is described in the steps below. In summary, the program determines moment effects at locations 1-6 for one wheel at a time for the subject bridge's aspect ratio, then combines the effects of individual wheels to get the moment effect of single or multiple trucks at locations 1-6. The effective width is calculated per Equation 9, where the FEM moment is equal to the combined effect of all wheels at locations 1-6 and the simple beam moment is the sum of locations 1 through 11 adjusted for bridge width.

Program Steps:

1. Input bridge characteristics, including: length, width, parapet width, number of lanes, truck pattern and wheel loads.
2. Determine longitudinal truck placement for maximum moment. That value is displayed for the load rating engineer's use. For calculating the effective width, the axle closest to mid-span is moved to mid-span to produce the maximum moment at midspan because effective width is only calculated at midspan.
3. Determine how much transverse truck movement is possible within the width of the bridge parapets, considering transverse lane spacing and truck gage per AASHTO specifications.
4. Determine transverse truck positions to analyze based on step 3. Truck positions are incremental from one side to the other. Increments of 1/25 of possible truck movement is used.

5. Determine the bridge specific influence surface arrays for location 1-6 based on the bridge aspect ratio, by interpolation of the SALOD influence surface stored data arrays.
6. Determine the influence moment coefficients at locations 1-6 and 5'-1' for each load. 5'-1' are mirrored arrays of 5-1. The influence constant is the moment coefficient at a location, 1 to 1', for a certain load position. The influence constant is interpolated from the bridge specific influence surface array for the desired location and load position.
7. Sum the influence constants for each load combination. A combination is comprised of multiple wheel loads from one or more trucks. The individual influence constant for each wheel is multiplied by the wheel load magnitude (in kips) and the product is summed for all wheels. The result will be a summed influence constant for each location, 1 to 1', and each transverse position of the truck(s).
8. Sum the total moment across the bridge width for each load combination and transverse truck position. This is essentially the area under a transverse moment curve. So, using the results from step 7, location 1 & 2 are averaged then multiplied by the distance between location 1 & 2, location 2 & 3 are averaged then multiplied by the distance between location 2 & 3, same process for locations 3 & 4, repeated up to location 1'. All of the products are summed.
9. The effective width is equal to the sum of the moment effect (step 8) divided by the sum of the influence constants for each load combination and transverse position (step 7). Calculate the effective width for locations 1-6 for each load combination and transverse position.
10. Determine the minimum effective width from results calculated in step 9.

After the base of the MathCAD programming was completed a comparison was made to results from SALOD for quality control purposes. To properly compare the outputs, changes to both programs were necessary. In SALOD the "User Defined Truck" option was selected and SU4 and HS20 trucks that match the dimensions and wheel configuration in the FDOT Load Rating Manual was defined (FDOT, 2017). This new truck was placed on the bridge in 1 lane, 2 lane, and 3 lane configurations. The MathCAD programming was altered by using a standoff distance of 16.5" from the parapet instead of 24" as would be used in current practice. To validate the 16.5" distance was correct a 0" parapet and a 12" parapet were used in both programs. Runs were made for bridge aspect ratios of 2.67, 1.87, 1.0, and 0.70. The results from MathCAD and SALOD were within 5% of each other for all controlling effective widths.

Conclusions

This report documents work completed for the FDOT Structures Maintenance Office to examine problematic flat slab load ratings. As of March 2016, when the Florida inventory was queried, there were approximately 980 flat slab bridges in the state. Nearly 170 of those were posted for load restriction. As design loads continue to increase and design provisions become more stringent, that number can reasonably be expected to grow. This report addresses the need for reasonable load rating tools for analyzing flat slab bridges which are conservative, but not overly so.

The capacity of flat, reinforced concrete slab bridges is frequently underestimated by available design codes. The effective width calculated per the AASHTO LRFD Bridge Design Specifications and AASHTO Standard Specifications are overly conservative compared to actual bridge behavior. For this study, results from physical load tests of 5 flat slab bridges were examined and compared to available load rating analytical methods. The selected bridges represent the FDOT inventory with a range of aspect ratios, lengths, thickness, age, barrier type and widening history. For all but one of the bridges, the AASHTO LRFD BDS effective width predicted significantly higher strains than were measured during the load test. The single bridge for which AASHTO predictions were accurate is extremely wide, with an aspect ratio of 4.8.

Results from the load tests were also compared with predicted strain based on the effective width calculated using the SALOD program. Except for the bridge with a high aspect ratio of 4.8, which falls outside the range of aspect ratios SALOD was developed for, the predicted strain based on SALOD is conservative compared to the test results, but not as conservative as AASHTO predictions. Most posted flat slab bridges in the Florida inventory have aspect ratios (width:length) that fall within the range that SALOD was developed for, 0.5 to 3.0.

The SALOD program is the most readily available tool to analyze flat reinforced slabs with aspect ratios between 0.5 and 3.0. The method is well vetted and provides an effective width that is based on interpolation between stored influence surface array results of isotropic plate finite element models. As it is based on refined analysis, it is supported by the AASHTO codes, but also less conservative than the AASHTO equation for effective width. The drawback of SALOD is the "black box" nature of the program. Intermediate calculations are not visible to the user and cannot be easily checked. As part of this work, the SALOD program methodology was used to write a program in MathCAD. The MathCAD-based program will allow the user to review programming language and check intermediate steps. The MathCAD program is based on the SALOD method but uses the current AASHTO requirements for truck placement and gage spacing. Since MathCAD uses the same stored influence surfaces as SALOD, the MathCAD program was checked against SALOD for quality control.

For future load ratings of flat reinforced concrete slabs, the MathCAD program based on SALOD should be used as a low cost tool for improving load ratings and avoiding postings. Its use is not a drastic departure from current load rating practices and is supported by AASHTO codes. As proven by the load tests performed during this study, some bridges have more capacity than can be predicted even with an effective width derived from SALOD. The higher capacity could be due to barrier stiffness or actual material strengths. For some bridges that cannot be load rated appropriately using analytical methods, physical load tests may be used by the load rating engineer.

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