Sensitivity Analysis of Rail-Structure Interaction Force Effects for Direct-Fixation Bridges

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ABSTRACT

This paper will discuss the sensitivity of predicted rail-structure interaction effects acting on direct-fixation light rail bridges. Specifically, the effects of variable design-parameters will be evaluated for both the forces due to thermal variation and the predicted gap lengths at rail break events through the use of nonlinear finite-element modeling. The modeling parameters that are considered for the sensitivity analysis include: (1) flexural stiffness of substructure, (2) rail fastener stiffness, and (3) the locations of continuous welded rail terminations. Results will be presented for the design of the proposed bridge carrying the FasTracks I-225 light rail corridor over I-225 in Denver, CO, as well as for an independent analysis of the FasTracks West Corridor LRT over Colfax and 6th in Lakewood, CO. The rail-structure interaction results obtained from nonlinear finite-element modeling will also be compared to results found using equations commonly included in the design criteria for other light rail facilities in the U.S.
INTRODUCTION

Rail-structure interaction effects in bridges with direct-fixation, continuously welded rail (CWR) are dependent on a number of parameters such as pier longitudinal stiffness, rail-fastener longitudinal stiffness, and the distance that CWR extends beyond the bridge. Many of these parameters may be unknown during design. For example, the longitudinal pier stiffness will vary depending on the assumed point of fixity of the piers below the ground line. The longitudinal stiffness of the rail fasteners may be uncertain, since the stiffness of fasteners produced by different manufacturers varies. The exact locations of joints between segments of CWR beyond the bridge may also be unknown.

These uncertainties mean that the engineer must ensure that the design can accommodate a range of conditions. This is particularly challenging because although most LRT design criteria require the engineer to consider rail-structure interaction, very few actually define the procedures that should be used to perform the analysis. Most design criteria are silent with regards to how uncertainties in the rail-structure interaction analysis parameters should be treated. In addition, few references are available that describe the analysis undertaken for specific projects. Therefore, the engineer has little guidance on which design parameters are most critical, and how to develop an effective and efficient model to evaluate force effects.

This paper aims to provide a design reference by presenting two case studies of rail-structure interaction analysis for LRT bridges, and by discussing the sensitivity of these analyses to variable parameters. Specifically, three design parameters are discussed: (1) flexural stiffness of the substructure, (2) rail fastener stiffness, and (3) the locations of continuous welded rail terminations. This paper will show that rail-structure interaction analysis is very sensitive to the
assumed substructure stiffness, and relatively less sensitive to variances in the assumed longitudinal stiffness of the rail fasteners. Rail-structure interaction results vary depending on whether CWR expansion joints are located at the abutments, or if the CWR extends away from the bridge beyond the abutments. For CWR extensions of 500 ft or longer beyond the abutments, the structural behavior is similar to when the rails are fully fixed above the abutments.

![Light rail vehicle traversing elevated direct-fixation track.](image)

**Figure 1.** Light rail vehicle traversing elevated direct-fixation track.

**PROJECT DESCRIPTIONS**

**Primary Analysis: FasTracks I-225 Corridor, LRT Median Flyover Bridge over I-225**

In Denver, Colorado, the Regional Transportation District (RTD) is currently extending the existing light rail I-225 corridor to the north. The Median Flyover Bridge is a new, 1,135-foot, nine-span structure that will carry two LRT tracks from the median of I-225 to the east side of the northbound traffic lanes. Figure 2 shows the location of the bridge and illustrates the geometry and general conditions.
The LRT corridor is generally at-grade with the adjacent highway, thus resulting in steep vertical grades at each end of the bridge and a high point peaking over the traffic lanes. Mechanically stabilized earth retaining walls flank both bridge approaches and wrap around the abutments to minimize the bridge length. The horizontal alignment uses reverse curvature to achieve the crossover, while long post-tensioned concrete straddle-bent piers help keep spans lengths within conventional limits.

Each span consists of (four or five) 72 inch deep, BT72 prestressed concrete bulb-tee girders with a cast-in-place concrete deck and rail plinths. Hammerhead piers with tapered caps are used, except at the straddle bent piers (Pier 5 and Pier 6). 96 inch diameter drilled caisson foundations are provided at the piers, while abutments are supported by 24 inch diameter caissons.

In accordance with RTD criteria for bridges over 400 feet in length [1], this bridge was designed to be constructed using direct-fixation tracks with CWR. The bridge was divided into three superstructure units separated by expansion joints at Pier 4 and Pier 7, along with expansion joints at both abutments. The direct-fixation CWR will be attached to the bridge
deck with the use of clip fasteners mounted on concrete plinths. A rendering of the proposed structure is shown in Figure 3.

![Figure 3. Rendering of the proposed Median Flyover Bridge.](image)

**Secondary Analysis: FasTracks West Corridor, LRT over Colfax and 6th**

The Colfax Bridge is a six-span direct-fixation single track structure located on the west corridor of the Denver RTD light rail system. Span lengths are 100’-123’- 91’-137’-141’-100’. Simpler in geometry than the Median Flyover Bridge, the structure is straight in plan, but includes a vertical crest curve. The superstructure is comprised of three prestressed concrete BT-72 bulb tee girders per span, which are made continuous over the length of the bridge through placing cast-in-place diaphragms at the interior piers. The structure is free to move longitudinally at the end piers, but is fixed longitudinally at each interior pier. The interior piers are single-column hammerhead piers. Each interior pier column is supported by a single 84 inch diameter drilled caisson. The abutments are supported by multiple 30 inch diameter drilled caissons. An elevation view of the bridge is shown in Figure 4.
Figure 4. Elevation view of the Colfax Bridge.

The construction for the bridge is complete, but the bridge is not yet in-service. The independent analysis was performed based on the details provided in the original construction documents. The authors conducted this independent analysis to determine if conclusions drawn from the design of the Median Flyover Bridge for rail-structure interaction would be applicable to a LRT bridge with less complex geometry.

GENERAL DESCRIPTION OF RAIL-STRUCTURE INTERACTION

In general, there are four primary considerations that should be evaluated during the design process when considering the effects of rail-structure interaction: structure forces induced by thermal variations between the rail and bridge, structure forces induced by a single rail break.
at locations of extreme stresses, control of the rail gap size in the event of a rail break, and control of forces in the CWR. This study will focus on the sensitivity of calculated structure forces induced by thermal variation and rail break gap sizes to changes in assumed substructure stiffness, rail fastener stiffness, and CWR fixity at abutments.

DESIGN CRITERIA FOR RAIL-STRUCTURE INTERACTION

Standard Practice

Currently, there is no single nationwide light rail design code. Most agencies tend to develop design specifications based on the AASHTO LRFD Bridge Design Specifications [2] code and the AREMA Manual for Railway Engineering [3]. However, the most recent AASHTO LRFD 5th edition code does not include light rail design provisions, and consequently does not address rail-structure interaction. The current AREMA 2010 Manual for Railway Engineering is intended for heavy freight railroad design, and does not directly address rail-structure interaction for light rail structures.

Just as there is no single nationwide light rail design code, there is also no single accepted approach to rail-structure interaction design. As stated in the Track Design Handbook for Light Rail Transit, “opinions differ throughout the transit design ‘community’ regarding the level of complexity required to design aerial structures subjected to thermal interaction forces from CWR” [4]. Several light rail system design criteria, including those for the Utah Transit Authority [5], and the Honolulu High-Capacity Transit Corridor Project [6], provide equations that specify the longitudinal and transverse forces on structures caused by rail-structure interaction. Other agencies, such as the NJ Transit Hudson-Bergen Light Rail Transit System [7], specify that a detailed finite element analysis should be performed in which the
superstructure, substructure, CWR, and rail fasteners are included in a single model. With that, designers are left on their own to develop a rational approach to modeling, analyzing, and designing for this condition.

**Project-Specific Design Criteria**

The Denver RTD and consultant design team elected to create rail-structure interaction design criteria specifically for the Median Flyover Bridge project. The same criteria was also applied to the independent analysis of the Colfax Bridge. The complexity of the Median Flyover structure prompted the design team to utilize a detailed 3D finite element model of the track and bridge to determine rail-structure interaction effects. Within the model, parameters such as substructure stiffness, fastener stiffness, and the limits of CWR extensions beyond the bridge were varied to examine the sensitivity of the results to these parameters. The selection of a 3D finite element analysis also built upon past Denver RTD design practice of determining rail-structure interaction forces using 2D finite element models [8]. Figure 5 shows a screenshot of the 3D Median Flyover Bridge structure model.

![Figure 5. Screenshot of the Median Flyover Bridge analysis model.](image-url)
The load combinations and structural design provisions of the AASHTO *LRFD Bridge Design Specifications 4th Edition* [2] were utilized to follow current light rail structure design practices [9]. The Denver RTD and the consultant design team collaborated to determine appropriate rail-structure interaction design procedures based on past Denver RTD design practice [8], the provisions of the *Track Design Handbook for Light Rail Transit*, and rail force criteria provided in *Design of Modern Steel Railway Bridges* [10]. These project specific analysis parameters are listed below. (Please note that these parameters may not be appropriate for all light rail bridges.)

**Modeling and analysis parameters**

- The analysis model thermal rise load case utilized a rail thermal rise of 70º F and a structure thermal rise of 20º F.
- The analysis model thermal fall load case utilized a rail thermal fall of 125º F and a structure thermal fall of 55º F.
- The rail fastener spacing utilized in the model is 30 inches, following standard industry practice.
- The rail fasteners on the bridge were considered rigid (infinitely stiff) with respect to transverse and vertical translation.
- The rails extended 2,500 feet beyond each end of the bridge, and had a tie spacing of 30 inches, except that 21 inch spacing is used on the approach slabs.
- The ties beyond the bridge had longitudinal, transverse, and vertical stiffness of 15.0 kips/inch, 0.5 kips/inch, and 60 kips/inch per rail per tie, respectively.
- Following the results of soil-structure interaction studies, drilled caissons were modeled at each pier as extending to a point of fixity of 25 feet below the ground line.

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Structure forces induced by thermal variation

- The forces generated from the analysis were considered AASHTO LRFD force effects due to uniform temperature (TU). Following the provisions of AASHTO LRFD 3.4.1, a TU load factor of 0.5 at the Strength limit state was used in conjunction with piers modeled using the gross moment of inertia.
- The rail faster longitudinal stiffness value used in the analysis model for thermal variation structure force was 18.0 kips/inch per rail per fastener. Denver RTD specified that the stiffness of the direct fixation fasteners utilized by the contractor must not exceed this value.
- The fastener longitudinal slip force was 3.5 kips per rail per fastener for analysis of thermal variation structure forces. This force was the maximum probable slip force that could be developed by the rail fasteners, and followed past Denver RTD design practice. The maximum longitudinal slip force used is comparable to the 3.2 kip maximum longitudinal toe load recommended by AREMA for elastic clips used in heavy rail direct fixation concrete slab track [3]. The use of maximum probable fastener stiffness and slip force values was intended to be conservative for thermal variation structure forces.

Structure forces induced by rail breaks

- Rail break forces effects were evaluated at each rail at each expansion joint location (16 locations total).
- Rail break forces were considered using the Denver RTD-specified Extreme Event III load combination.
Rail break gap control

- The maximum permitted rail break gap was 3 inches.
- The rail fastener longitudinal stiffness value used in the analysis model for rail break gap control was 12.0 kips/inch per rail per fastener. Denver RTD specified that the longitudinal stiffness of the direct fixation fasteners utilized by the contractor must not be less than this value.
- The fastener longitudinal slip force was 2.0 kips per rail per fastener for analysis of rail break gap lengths. This force was the minimum probable slip force that Denver RTD assumed could be developed by the rail fasteners. The minimum longitudinal slip force used is comparable to the 2.2 kip minimum longitudinal toe load recommended by AREMA for elastic clips used in heavy rail direct fixation concrete slab track [3]. Using minimum probable fastener longitudinal stiffness and slip force values was intended to be conservative for rail gap analysis.

Control of CWR forces

- The maximum permitted axial tensile force for unbroken rail was $0.65F_u$, based on past design practice. Following recommendations given in Design of Modern Steel Railway Bridges [10], the maximum permitted compressive force in the rail was 150 kips.

MODELING APPROACH FOR RAIL-STRUCTURE INTERACTION ANALYSIS

Modeling of the Median Flyover Bridge

A 3D analysis model was developed using the MIDAS/Civil analysis software package. The substructure piers, pier caps, abutment piles and pile caps, diaphragms, individual
superstructure concrete I-girders, and individual steel rails were modeled using beam elements. Four-noded shell elements were utilized to model the bridge deck. Piers were modeled using gross section properties.

The modeling of the connection between the rail and structure is shown in Figure 6. The rail fasteners were modeled using longitudinal bilinear springs in parallel with beam elements with zero axial stiffness, and rigid transverse stiffness. For calculation of maximum force effects due to thermal variations or from a rail break event, a stiffness of 18 kips/inch per rail per fastener up to a total slip force of 3.5 kips was used. For purposes of evaluating the maximum rail gap that can occur during a rail break event, the models used bilinear springs with a stiffness of 12 kips/inch per rail per fastener, up to a total force of 2.0 kips. Beyond the slip force values, the springs had near zero longitudinal stiffness. The bilinear force-displacement relationship for the 18 kips/inch per rail fasteners is shown in Figure 7.

![Figure 6. Modeling of the connection between the rail and structure](image-url)
Employing bilinear springs made the analysis a nonlinear iterative process. To reduce analysis time, bilinear springs were only utilized for rail-structure interaction load cases. All other forces, such as those due to dead loads and live loads, were computed using a separate model with linear rail fastener springs.

Modeling of the Colfax Bridge

The authors followed a modeling approach similar to that used for the Median Flyover Bridge. MIDAS/Civil software was utilized to create 3D models that were solved using a nonlinear iterative process. Rail fasteners were modeled using bilinear springs with a slip force of 3.5 kips per rail per fastener. The spacing of the fasteners was 30 inches. Models were created for a fastener stiffness of 12 kips/inch and 18 kips/inch, and for pier points of fixity at the ground line, and at points 20 ft below the ground line along each drilled caisson. Following AASHTO LRFD 3.4.1, the piers and caissons were modeled with gross section properties.

Varying the pier fixity and fastener stiffness parameters resulted in the creation of four separate models: 12 kips/inch fasteners with piers fixed at the ground line, 12 kips/inch fasteners with piers fixed 20 ft below the ground line, 18 kips/inch fasteners with piers fixed at
the ground line, and 18 k/inch fasteners with piers fixed 20 ft below the ground line. As will be discussed, the results of the Median Flyover Bridge analysis showed that length of rail extension beyond the abutment, (as long as the extension was at least 500 feet), did not have a significant influence on the analysis results. Therefore, for simplicity, the Colfax Bridge was modeled by attaching the rail elements above each abutment to fully-fixed supports. Each analysis model utilized a thermal fall load case comprised of a 125°F temperature drop for the rail and a 55°F temperature drop for the structure. A screen shot of an analysis model (with the top 20 ft of each drilled caisson included) is shown in Figure 8.

Figure 8. Screenshot of Colfax and 6th structural analysis model.

SENSITIVITY OF CALCULATED RAIL-STRUCTURE INTERACTION EFFECTS TO ANALYSIS PARAMETERS

The sensitivity of analysis results for structure forces due to thermal variation and rail break gap lengths are discussed below for each of the three parameters considered.
Analysis Parameter 1: Relative Stiffness of the Substructure

Structure Forces Due to Thermal Variation in the Median Flyover Bridge

Resultant shear forces at the bases of the Median Flyover Bridge piers were compared for two analysis models: in the first, the piers were fixed at the ground line, and in the second, the upper portion of each drilled caisson was modeled down to the calculated equivalent depth of fixity 25 feet below the ground line. Resultant shear forces were calculated for the thermal fall load case. The rail fastener stiffness and CWR extension parameters were held constant in both models. Rail fastener stiffness was modeled as 18 kips/inch, and the CWR was extended to a point 2,500 ft beyond each end of the bridge. The resultant shear force is defined as \( F = (F_x^2 + F_y^2)^{1/2} \) where \( F_x \) and \( F_y \) are the shear forces parallel to the local x and y axes of the pier cross section.

As shown in Figure 9, varying the stiffness of the substructure caused significant differences in the predicted forces due to thermal variations. Base of pier resultant shear forces for the hammerhead piers (Piers 2, 3, 4, and 7, 8, 9) were on average 35% less in the case where flexibility of the foundations was considered when compared to the fixed base condition. Straddle-bent base of pier resultant shear forces (Piers 5W, 5E, 6W, and 6E) calculated by the model with foundation flexibility were 68% lower on average than those calculated by the model with piers fixed at the ground line.
Figure 9. Structure forces due to thermal variation in the Median Flyover Bridge for piers fixed at the ground line, and piers fixed 25 ft below the ground line.

**Structure Forces Due to Thermal Variation in the Colfax Bridge**

Similar to the Median Flyover Bridge, the calculated shear forces at the base of the Colfax Bridge were compared for two analysis models. In the first, the piers were fixed at the ground line, and in the second, the piers were fixed 20 ft below the ground line. Forces were calculated for the thermal fall load case. The rail fastener stiffness parameter was held constant at 18 kips/inch, and the CWR was fixed at the abutments. Figure 10 shows that following the trend observed for the Median Flyover Bridge, the results vary widely depending on the assumed pier fixity location.
Figure 10. Structure forces due to thermal variation in the Colfax Bridge for piers fixed at the ground line, and piers fixed 20 ft below the ground line.

Rail Break Gap Lengths in the Median Flyover Bridge

Contrary to the structure forces due to thermal variation, calculated rail break gap lengths for the Median Flyover Bridge were insensitive to changes in substructure stiffness. Rail break gap lengths were calculated using two analysis models, in which the piers were either fixed at the ground line or fixed 25 ft below the ground line. Gap lengths for a break of the east rail of the northbound track at Abutment 10 were calculated for the thermal fall load case. In both models the rail fastener stiffness parameter was held constant at the lower bound value of 12 kips/inch, and the CWR was extended 2,500 ft beyond both abutments. As expected, the gap length calculated by the model with foundation flexibility was larger than the model with fixed piers, although the difference in results was small. The gap length calculated by the model with
piers fixed at the ground line was 3.25 inches, while the gap length calculated by the model with foundation flexibility was 3.31 inches. These virtually identical results show that for the Median Flyover Bridge, calculated rail break gap length values were not very sensitive to changes in substructure stiffness. Figure 11 is a screen shot showing the 3.31 inch rail break gap predicted by the analysis model with foundation flexibility at this location. The break is shown with a scale factor of 1.0, and rail supports beyond the structure are not shown.

![Figure 11](image)

Figure 11. Predicted rail break gap at Abutment 10 for the thermal fall load case.

**Rail Break Gap Lengths in the Colfax Bridge**

Similar to the Median Flyover Bridge, calculated rail break gap lengths for the Colfax Bridge were relatively insensitive to changes in substructure stiffness. Rail break gap lengths were calculated for the west rail above the north abutment. Calculations were performed using two analysis models, in which the piers were either fixed at the ground line, or extended 20 ft below the ground line to fixed supports. The rail fastener stiffness parameter was held constant at 12 kips/inch, and the CWR was fixed beyond both abutments. The gap length calculated by the model with piers fixed at the base was 1.78 inches, while the gap length calculated by the
model that included foundation flexibility was 1.91 inches, resulting in a small difference of 0.13 inches.

**Analysis Parameter 2: Longitudinal Stiffness of the Rail Fasteners**

*Structure Forces Due to Thermal Variation in the Median Flyover Bridge*

Resultant shear forces at the bases of the Median Flyover Bridge piers were compared for two analysis models: in the first, the rail fastener stiffness was 18 kips/inch, and in the second, the rail fastener stiffness was 12 kips/inch. These fastener stiffness values are the lower and upper bounds defined in the Median Flyover Bridge design criteria. The substructure stiffness parameter was held constant by modeling the top 25 ft of each drilled caisson in both models, and the CWR parameter was held constant by using 2,500 ft track extensions. Resultant shear forces were calculated for the thermal fall load case. The base of pier forces computed using the 18 kip/inch fastener stiffness model were 8% higher on average than the forces computed using the 12 kip/inch fastener stiffness model, and otherwise identical parameters. Structure forces due to thermal variations were therefore significantly more sensitive to changes in pier bending stiffness than to changes in pier fastener stiffness. Figure 12 plots base of pier shear resultant forces due to thermal variations for both analysis models.
Figure 12. Structure forces due to thermal variation in the Median Flyover Bridge for fasteners with a longitudinal stiffness of 12 kips/inch, and a stiffness of 18 kips/inch.

Structure Forces Due to Thermal Variation in the Colfax Bridge

Similar to the Median Flyover Bridge, the calculated shear forces at the ground line of each pier were compared for two models in which fasteners have a longitudinal stiffness of either 12 kips/inch or 18 kips/inch. The substructure stiffness parameter was held constant by including the top 20 ft of each drilled caisson in the models, and the CWR parameter was held constant by fixing the rail at the abutments. The shear forces are shown in Figure 13 and inspection of the figure shows that the results are virtually the same for both values of fastener longitudinal stiffness. Following the trend observed for the Median Flyover Bridge, the base of shear forces calculated by the Colfax Bridge models were much more sensitive to changes in substructure stiffness than to changes in fastener stiffness.
Figure 13. Structure forces due to thermal variation in the Colfax Bridge for fasteners with a longitudinal stiffness of 12 kips/inch, and a stiffness of 18 kips/inch.

Rail Break Gap Lengths in the Median Flyover Bridge

Calculated rail break gap lengths for the Median Flyover Bridge were even less sensitive to changes in fastener stiffness than to changes in substructure stiffness. Rail break gap lengths of the east rail of the northbound track at Abutment 10 were calculated using two models in which the fasteners had a longitudinal stiffness of either 12 kips/inch or 18 kips/inch. The substructure stiffness parameter was held constant by fixing the piers at the ground line, and the CWR was extended 2,500 ft beyond each abutment. The gap length calculated using the model with 12 kip/inch fastener stiffness was 3.25 inches, while the gap length calculated using the 18 kip/inch fastener stiffness model was 3.23 inches. The rail break gap for the lower fastener stiffness is larger as expected, but the difference of only 0.02 inches between the two
analyses was not significant, and was less than the difference of 0.06 inches calculated when the substructure stiffness parameter was varied while fastener stiffness was held constant.

*Rail Break Gap Lengths in the Colfax Bridge*

Rail break gap lengths at the west rail above the north abutment were calculated using two models in which the fasteners had a longitudinal stiffness of either 12 kips/inch or 18 kips/inch. The substructure stiffness parameter was held constant by including the upper 20 ft of each drilled caisson in the models, and the CWR was fixed beyond the abutments. The gap length calculated using the model with 12 kip/inch fastener stiffness was 1.91 inches, while the gap length calculated using the 18 kip/inch fastener stiffness model was 1.87 inches. Similar to Median Flyover Bridge, the rail break gap for the lower fastener stiffness was larger as expected, but the 0.04 inch difference between the two analyses was insignificant. While changes to both parameters caused small differences in calculated rail break gap lengths, the predicted gap lengths were more sensitive to changes in substructure stiffness than to changes in fastener stiffness. These trends can be observed in Figure 14, which plots all of the calculated rail break gap results for the Colfax Bridge.
Figure 14. Colfax Bridge predicted rail break gaps for the west rail above the north abutment.

Analysis Parameter 3: Location of CWR Termination Relative to the Structure

Structure Forces Due to Thermal Variation in the Median Flyover Bridge

The objective of this comparison was to determine the sensitivity of the bridge response relative to the assumed location of the CWR termination. Resultant shear forces at the bases of the Median Flyover Bridge piers were compared for four analysis cases: termination of CWR at 2,500 ft beyond each abutment, 500 ft beyond each abutment, fixed at each abutment (infinite length), and finally a case where the termination occurs at the abutments (free condition). The substructure stiffness parameter was held constant by including the top 25 ft of each drilled caisson in each model, and the rail fastener stiffness parameter was kept constant at 18 kips/inch. This comparison was only performed for the Median Flyover Bridge.
The base-of-pier resultant shear forces due to the thermal fall load case were evaluated for each of the four CWR termination options. The analysis results show that the thermal forces on the bridge were very similar for all cases where the CWR extended beyond the limits of the bridge. Only the case where the rails were free at the abutments produced differing results. In this case, the base-of-pier resultant shear forces at end Piers 2 and 9 were significantly higher than those calculated by the other models, while the shear forces at all other locations were relatively smaller. The results of the four CWR termination conditions are shown below in Figure 15.

**Figure 15.** Median Flyover Bridge base-of-pier resultant shear forces due to thermal variation for models with 2,500-foot track extensions, 500-foot track extensions, a fully fixed rail condition, and a free rail condition above the abutments.
The results of this analysis indicate that the modeled length of CWR extension beyond the limits of the bridge is not critical, as long as the termination is not located above the ends of the bridge. For simplicity during design, it may be advantageous to only consider a case where the rails are modeled as fixed at the abutments.

The sensitivity of rail break gap lengths at abutments was not compared for different CWR extension lengths.

**Comparison of results to other light rail system design criteria**

The analysis results for structure forces due to thermal variations were compared to the results obtained by applying an equation found in the criteria of other systems to the Median Flyover and Colfax Bridges. The Utah Transit Authority, the Honolulu High-Capacity Transit Corridor Project, and Denver RTD commuter rail design criteria [5,6] provide the following formula for the longitudinal rail-structure interaction force \( T \) per structure per rail:

\[
T = 0.65PL
\]  

(1)

where \( P \) = fastener slip force per linear foot and \( L \) = average span length of two adjacent spans (ft). The Honolulu design criteria impose a maximum limit of 200 kips on this force.

The Denver RTD commuter rail design criteria define the transverse rail-structure interaction force in curved direct fixation track structures to be

\[
T = \frac{179}{R}
\]  

(2)

where \( T \) = transverse force per structure per rail per foot, and \( R \) = radius of curvature in feet.
Comparison for the Median Flyover Bridge

Longitudinal rail forces were computed using Eq. (1) for the I-225 structure, using an average span length of 120 ft, a fastener slip force of 2.6 kips and a fastener spacing of 30 inches. The resulting force from Equation 1 was 0.26 kip/ft/rail. Transverse forces were computed for the two 1000 ft radius curves on the structure, resulting in a transverse force of 0.179 kip/ft/rail. These forces were applied to the rail elements in the I-225 structure analysis model as longitudinal and transverse line loads acting along the rail element local longitudinal and transverse axes. Foundation flexibility was included in the model, rail fasteners were modeled with linear springs with a stiffness of 18 kips/inch, and the CWR was extended 2,500 ft beyond each abutment.

Figure 16. Base of pier thermal variation force comparison between design criteria equations and nonlinear finite element analysis results.
Applying the loading derived using Eq. (1) and Eq. (2) to the Median Flyover Bridge structural model with the parameters described above yielded analysis results for the longitudinal ($F_x$) and transverse ($F_y$) shear forces at the ground line of each pier. The resultant shear force $F$ at the base of each pier due to this loading was computed using the relation $F = \sqrt{(F_x)^2 + (F_y)^2}$. These force resultants are plotted in Figure 16, and are compared to the forces computed for the thermal fall load case using a nonlinear analysis in which the foundation flexibility, fastener stiffness, and CWR extension parameters were identical. Inspection of Figure 16 shows that the base-of-pier forces computed using Eq. (1) and Eq. (2) are closely comparable to the 3D nonlinear finite element analysis results for the hammerhead piers (Piers 2, 3, 4, and Piers 7, 8, and 9). However, there are significant differences between the 3D nonlinear finite element and Eq. (1) and Eq. (2) shear forces for the straddle-bent piers (Piers 5W, 5E, 6W, and 6E, near the center of the structure).

Comparison for the Colfax Bridge

Eq. (1) from other light rail system design criteria was applied to the Colfax Bridge analysis. Since the Colfax Bridge is straight, Eq. (2) was not applicable. The parameters used in Eq. (1) were an average span length of 115.33 ft, a fastener slip force of 3.5 kips, and a fastener spacing of 30 inches. The resulting force from Eq. (1) was 0.23 kip/ft/rail. This force was applied to the rails as a longitudinal line load, in a model in which the depth of fixity of the piers was 20 ft below the ground line, fasteners were modeled as linear springs with a stiffness of 18 kips/inch, and the CWR was fixed above the abutments. In Figure 17, the resulting shear forces at the base of the piers are compared with those from a nonlinear finite element analysis of the thermal fall load case with the same pier stiffness, fastener stiffness, and CWR extension parameters. Unlike Median Flyover Bridge, there are large differences between the
shear forces predicted from both analyses at each pier. The pier shear forces at the ground line obtained from the nonlinear finite element analysis were significantly larger than those obtained using Eq. (1).

**Figure 17.** Predicted pier shear forces at the ground line obtained from a typical light rail design criteria equation and the nonlinear finite element analysis.

**CONCLUSIONS**

The analysis of the Median Flyover and Colfax Bridges indicates that calculated rail-structure interaction effects are sensitive to the designer’s assumptions for substructure stiffness, rail fastener stiffness, and CWR extension length. However, the degree of sensitivity depends on which parameters are considered.

For example, the rail-structure interaction analysis of both the I-225 Flyover Bridge and the Colfax Bridge showed that pier forces due to thermal variations are strongly dependent on the
assumed substructure stiffness. The magnitude of effects could be expected to vary by upwards of 30%, depending on modeling conditions. For this reason, it is recommended that the finite element model provide as much accuracy of the substructure and foundation elements be possible. Calculated rail break gap lengths were less sensitive to changes in assumed substructure stiffness than pier forces.

Conversely, variations in rail longitudinal fastener stiffness (within the typical range of fastener products) had little effect on predicted thermal variation pier forces or predicted rail break gap lengths. However, it is still recommended that minimum and maximum stiffness parameters be considered in the model and defined in the construction documents. These assumed values are important when evaluating the actual fastener products selected by the contractor to ensure that they are within the range considered in the design.

Rail-structure interaction analysis results varied depending on if the CWR was terminated at the bridge abutments or continued beyond the structure. CWR extensions of 500 ft or greater beyond the abutments produced results comparable to those obtained when the CWR was assumed to be fully fixed above the abutments. To ensure that the design parameters reflect field conditions, it is recommended that the CWR termination not be located on the bridge, or at the bridge ends. For simplicity, consideration should be given to modeling the CWR with a fixed condition at the bridge ends to limit the number of nodes and analysis run time.

Finally, the rail-structure interaction results for these two case study bridges were compared to empirical equations provided in the design criteria for several LRT Facilities. The pier shear forces results of the 3D nonlinear finite element analysis were similar to those obtained using Eq. (1) for the hammerhead piers of the Median Flyover Bridge, but were not comparable for
the Median Flyover Bridge straddle bent piers, or the Colfax Bridge piers. The use of 3D nonlinear finite element modeling may therefore be justified for the design of other direct-fixation light rail structures.

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