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By E-mail dane.doleman@vancouver.ca

2007 July 26

City of Vancouver
National Works Yard
701 National Avenue
Vancouver, BC V6A 4L3

Attention: Dane Doleman, P.Eng., Structures Engineer - Streets Operations, Engineering Services

Dear Sirs,

Re: Granville Bridge - Load Limit Review

This letter report presents the results of a preliminary load limit review for the Granville Bridge conducted by Buckland & Taylor Ltd. (B&T) for the City of Vancouver (City).

Background

Although transit buses are allowed to transit the Granville Bridge, truck traffic and heavier types of tour buses are prohibited. The bridge is posted for a 27 tonne load limit and the City provides intermittent monitoring and enforcement of the type and weight of crossing vehicles. We understand that the posted load limit was selected following the completion of a 1974 study by CWMM entitled "Effects of B.C. Vehicle Load Regulations on City Bridges".

Recently the City has been approached with requests to allow heavier dual axle tour buses on the structure. While the City is interested in accommodating the tour bus operations, some uncertainty exists regarding the applicability of the 27 tonne load limit for current bridge design/evaluation standards. Therefore, the City engaged B&T to conduct a preliminary load limit review of the Granville Bridge to provide an indication of the potential to alter the currently posted load limit.

Project Scope

The scope of work for this project consists of evaluating the adequacy of selected bridge superstructure components, in accordance with the provisions of Section 14 of CAN/CSA-S6-06 (CHBDC), for a number of transit bus and tour bus configurations supplied by the City. Engineering judgment is used to select bridge components that are likely to govern the live load capacities of various segments of the bridge. The goal of this review is to provide preliminary assessments of the potential for accommodating the tour buses on the bridge, the suitability of the existing 27 tonne posted load limit and the potential value of conducting a full load capacity evaluation for all bridge components.

The findings of this review are to be presented to the City in the form of a letter report.

Project Inputs

The City provided the following documents:

- design drawings for the various components of the Granville Bridge
- portions of the original specifications pertaining to structural steel and concrete
- various configurations of transit and tour buses.

As part of this assignment, on 2007 March 27 B&T personnel (Darrel Gagnon, David Queen and Jackie Wong) performed a cursory inspection of most bridge superstructure components from ground level and no significant signs of deterioration were observed. A detailed inspection of all bridge components is recommended if a full evaluation of all bridge components is conducted.

Material Properties

The original specifications indicate the following material properties:

- Structural Steel – Equivalent of 230 MPa grade steel (F_y = 230 MPa)
- Reinforcement Steel – (F_y = 230 MPa)
- Concrete – Minimum compressive strength of 17.2 MPa. However, a higher compressive strength of 30 MPa was used in the evaluation calculations. This issue is discussed further in Discussion of Findings.

Loadings

Dead loadings were assessed as per the provisions of S6-06 and the information contained on the as-built drawings of the bridge.

Live loadings considered were as follows and a diagram of these loadings is attached:

1. The CL-3 loading from CHBDC which is considered to conservatively include all typical bus loadings and straight trucks (no tractor trailer units).
2. The heaviest two axle Coast Mountain bus configuration (GVWR = 19178 kg). This vehicle never governed over the Articulating Bus.
3. The Coast Mountain Articulating bus configuration (GVWR = 30892 kg). This vehicle produced load demands very similar to those produced by the heaviest Tour Bus.
4. The heaviest Tour Bus (Prevost H3-45 GVWR = 23665 kg).

Summary of Results

Results of the preliminary load limit review are displayed on the attached Figures 1 to 10. The results are provided in terms of a live load capacity factor (LLCF) in accordance with the provisions Section 14 of CHBDC. The LLCF indicates what portion of the evaluation vehicle can be carried by the bridge component to the target level of safety required for that particular bridge component. A LLCF of 1.0 or higher indicates that the component is satisfactory for that loading. A LLCF of less than 1.0 indicates that the component is deficient for that loading and what portion of the loading that can be safely carried. A LLCF of less than 0.0 indicates that the component does not have sufficient capacity to carry dead load to the target level of safety.

Deck System on Steel Truss Spans

The concrete deck, steel stringers and steel floorbeams for a typical deck segment on the steel truss spans were evaluated for the CL-3 loading and found to be adequate. This indicates that all the supplied bus configurations could also be allowed on the steel truss spans. Note that a previous review by B&T indicated that the steel span trusses could likely carry the CL-625 design loading, if a few deficient components were upgraded.

Seven Girder Concrete Approach Spans

The seven girder concrete approach spans, 11 spans in total, are located at either end of the steel truss spans prior to the side approach ramps separating from Granville Street. As shown in Figure 1, these spans have sufficient moment capacity for the CL-3 loading which indicates that they are also adequate in flexure for all the proposed bus loadings. However, these spans were found to contain shear capacity deficiencies for both the CL-3 and Tour Bus loadings, as shown in Figures 2 and 3. Note that the vertical dashed lines on the shear capacity plots indicate the limits of zones close to the piers where the shear forces are considered to transfer directly to the piers and any indicated shear capacity issues are not applicable. For the CL-3 loading the shear deficiencies are more pronounced at the non-continuous ends of the spans and near midspan. For the Tour Bus the deficiencies are less severe or extensive but still significant. Minimum live load capacities factors for the CL-3 and Tour Bus loadings are approximately 0.25 and 0.50, respectively. Live load capacity factors of less than 1.0 indicate that the structure does not provide the desired level of safety for the loading being evaluated. Less severe shear deficiencies could also be expected for the two axle transit bus configurations.

Five Girder Concrete Approach Spans

The five girder concrete approach spans, 22 spans in total, are located along Granville Street from the points where the side approach spans separate from Granville to the ends of the bridge. These spans have sufficient moment capacity for the CL-3 loading and all bus configurations. However, similar to the seven girder spans, these spans were found to contain shear capacity deficiencies for both the CL-3 and bus loadings, as shown in Figures 4 and 5. Minimum live load capacity factors for the CL-3 and Tour Bus loadings are approximately 0.28 and 0.50, respectively.

Three Girder Concrete Approach Spans

The three girder concrete approach spans, 23 spans in total, consist of the Howe and Seymour Street ramps at the north end of the bridge. These spans were found to be acceptable for all transit and tour bus configurations but are deficient for the CL-3 loading in both shear and moment, shown in Figures 6 and 7. The minimum live load capacity factors for the CL-3 moments and shears are approximately 0.75 and 0.95, respectively.

Two Girder Concrete Approach Spans

The two girder concrete approach spans, 45 spans in total, consist of the Hemlock, Fir and 4th St. ramps on the south end of the bridge. These spans were found to have sufficient moment capacity for all the bus configurations and the CL-3 loading, as shown in Figure 8. However, these spans were found to be deficient in shear for both the CL-3 and Tour Bus loadings over substantial lengths of the spans, as shown in Figures 9 and 10. Minimum live load capacity factors for the CL-3 and Tour Bus loadings are approximately 0.20 and 0.25, respectively.

Discussion of Findings

While the deck system on the steel truss spans was found to be adequate for the CL-3 loading and all bus configurations being considered, substantial and widespread deficiencies for these loadings were identified on most of the concrete girder approach spans. This implies that portions of the bridge lack sufficient capacity to carry the CL-3 and Tour Bus loading with the level of safety prescribed by the current bridge design standard, CHBDC (S6-06). In addition, portions of the bridge could be considered deficient for the loadings imposed by the existing two axle transit buses.

Although portions of the bridge may be deficient for the existing transit bus loadings, the bridge has successfully carried these loadings for an extended period of time while exhibiting no significant signs of distress. The OHBDC 3rd Edtion (Ontario Highway Bridge Design Code) contained provisions indicating that concrete bridges that have carried a loading for an extended period of time with no signs of distress, shall be considered adequate for that loading. The OHBDC is the design standard on which the current standard CHBDC was based. Although this provision was not included in CHBDC, it does provide a rationale for maintaining the existing transit bus traffic on the bridge. However, no increases in the vehicle loadings on the bridge should be considered.

In all cases, the moment capacities of the bridge components considered in this assessment were found to be adequate for the flexural demands imposed by the transit bus and tour bus configurations and only deficient for the CL-3 loading at one location on the spans with five girders. This is not unexpected since the bus configurations are relatively similar to the original bridge design loadings and the means of calculating moment resistances have not changed substantially.

While the steel deck system on the truss spans were adequate for the shear loadings imposed by the CL-3 loading, substantial and widespread shear deficiencies were identified in the girders of the concrete approach spans for both the CL-3 and the bus loadings. This was not unexpected as shear capacity issues are often identified in concrete girders from this time period. The shear capacity of a concrete section is typically comprised of a component provided by the concrete, V_C, and by the transverse reinforcement (stirrups), V_S. Since the design of the Granville Bridge, the amount of shear capacity that is permitted to be obtained from the concrete, V_C, has dropped dramatically. The design specifications for the Granville Bridge limited the allowable shear stress on a concrete section to $0.03f'_c$ with no requirements for a minimum amount of transverse reinforcement. While the determination of concrete shear capacity by the current design standard, CHBDC, is complex, approximate comparable limits for shear carried by concrete are $0.025f'_c$ with minimum transverse reinforcement and $0.009f'_c$ with less than minimum transverse reinforcement. Therefore, the effective reduction in V_c resistance from the original design provisions is about 17% when minimum transverse steel requirements are met but about 70% when they are not met. All of the shear capacity deficiencies identified for the concrete girder spans occurs in locations where the minimum transverse reinforcement requirements are not achieved. Although the provisions of CHBDC typically produce higher values for V_s component of shear resistance compared to the original design, most of the deficient portions of the girders contain such low amounts of transverse reinforcement that this increase does little to offset the reduction in V_c.

A concrete girder that lacks a sufficient amount of transverse reinforcement can fail suddenly with the formation of the first shear induced crack. This type of failure is referred to as a brittle failure and typically occurs with little or no warning signs and can result in a complete loss of load carrying capacity. If the structural system of the bridge is non-redundant, this could lead to a partial or complete collapse of the span. Conversely, a girder with sufficient transverse reinforcement tends to exhibit multiple shear cracks prior to achieving its ultimate capacity and retains some post ultimate load carrying capacity. This type of component behaviour is referred to as ductile and typically provides warning of approaching shear failures and improves the probability that redundancies in the structural system will help prevent a collapse of the span. Therefore, the provision of minimum transverse reinforcement is typically considered to be desirable. Although modern design standards do not strictly mandate the provision of minimum transverse reinforcement, very substantial penalties are applied to the V_c component of the shear resistance if it is not provided.

The V_c component of shear resistance used in this assessment was based on an effective compressive concrete strength of $f'_c = 30$ MPa. This is higher than the minimum compressive concrete strength of 17.2 MPa stated in the original design specifications. A value of 30 MPa was selected based on the water/cement ratios given in the specification and the age of the concrete. However, the appropriateness of the 30 MPa value should be confirmed based on testing of concrete cores. Note that such testing may not be necessary if an upgrade design for the concrete girders does not require concrete strengths in excess of 17.2 MPa.

Although the bridge has performed well to date under the existing loadings, the identified shear deficiencies are significant and it is recommended that appropriate upgrades to the structure be developed and implemented in a timely manner. In addition, it is recommended that no increases be permitted in the weights of the vehicles using the bridge.

Discussion of Potential Shear Capacity Upgrades

Upgrades to the shear capacities of the concrete girders can be accomplished by a number of means. However, the issues that need to be considered by all potential upgrade systems include:

- The level of increased shear capacity to be provided (transit bus loadings, tour bus loadings, CL-3 loading, CL-625 highway legal design loading, etc.)
- reliability of the increased shear capacity (robustness of the upgrade system)
- level of component ductility provided (Is minimum transverse reinforcement to be achieved in all locations?)
- aesthetics (some upgrade systems will have greater impacts than others)
- construction access and impacts
- costs

Potential upgrade techniques include, but are not limited to, the installation of external stirrups, application of a fibre-reinforced polymer (FRP) layer to the girder exteriors and the application of a reinforced concrete layer onto the girders. Each of these techniques has potential advantages and disadvantages that should be assessed prior to the selection of the preferred upgrade system. A brief discussion of these potential upgrade techniques follows.

External Stirrups

External stirrups could be installed at any location along the girder to enhance the shear capacity. The external stirrup could consist of a vertical 'U' shaped galvanized steel bracket that wraps under the bottom of the girder and is anchored at the top by a bolt placed in a core hole through the girder. This arrangement can be highly effective in increasing shear capacity, is robust, increase the level of component ductility, the brackets are relatively simple to fabricate and install, can be installed from below deck level from man lifts and are expected to be relatively economical. However, the external stirrups may not be as aesthetically pleasing as other options.

FRP Reinforcement

The shear capacity of the girders could be increased by adhering a FRP material to the sides and bottoms of the girders. This technique is likely to provide a more aesthetically pleasing appearance but may not be as robust as the external stirrups. Access requirements for the FRP will likely be significantly higher as cleaning of the existing surfaces are required and the FRP is typically applied over larger areas.

Addition of Stirrups in Concrete Jacket

Additional stirrups and longitudinal reinforcement could be added to the girders by encasing the girders in a reinforced concrete jacket. Although this technique could provide a very robust component and may result in a more aesthetically pleasing appearance, substantially higher costs should be expected due to the increased requirements for access, materials and labour.

Upgrade Design Cost Estimate

A preliminary cost estimate range for completing the assessment all concrete girder approach spans on the Granville Bridge and developing preliminary and detailed design documents for the anticipated shear capacity upgrades is $200,000 to $300,000, (GST not included).

Closing

Please contact us to discuss this issue further.

Yours truly,

BUCKLAND & TAYLOR LTD.

Darrel Gagnon, P.Eng.

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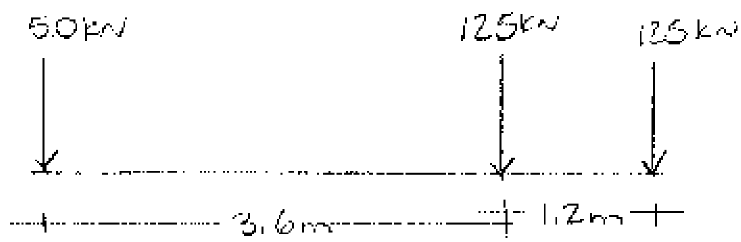
PROJECT: Granville Bridge — CONT: 1803
SUBJECT: Load Rating Pre-Assessment — PAGE:
CHECKED BY: — DATE: — CALCULATIONS BY: EER — DATE: June /07

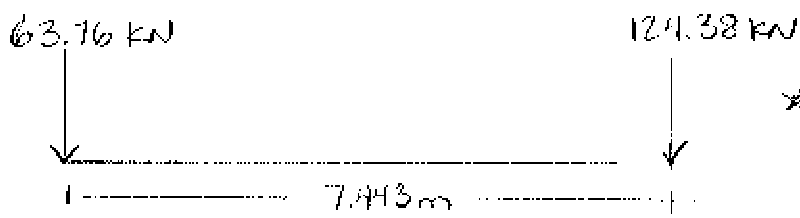
Vehicle Loadings

1. CL3
2. Coast Mountain typ. bus
3. Coast Mountain articulating bus
4. tour bus.

1 CL3 $GVWR = 0.48 \times CL625 = 0.48(625) = 300\,kN$

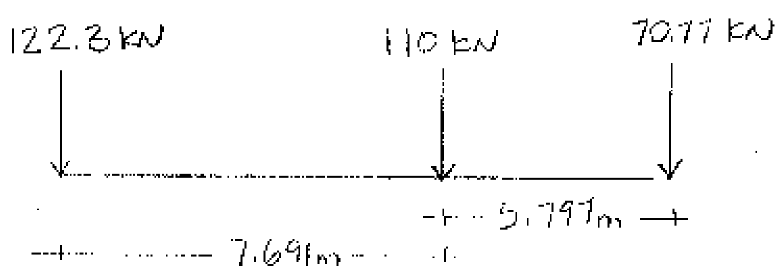


2. Coast Mountain typical bus. $GVWR = 19178\,kg$
$= 188.14\,kN$



* note: this vehicle was not used in the analysis due to the relatively low GVWR and absence of tandem.

3. Coast Mountain Articulating Bus $GVWR = 30892\,kg$
$= 303.05\,kN$



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PROJECT Grenville Bridge CONT 1803

SUBJECT Load Rating Pre-Assessment PAGE

CHECKED BY DATE CALCULATIONS BY EER DATE June /07

4. Tour Bus (Prevost H3-45) GVWR = 23665 kg
= 232.2 kN

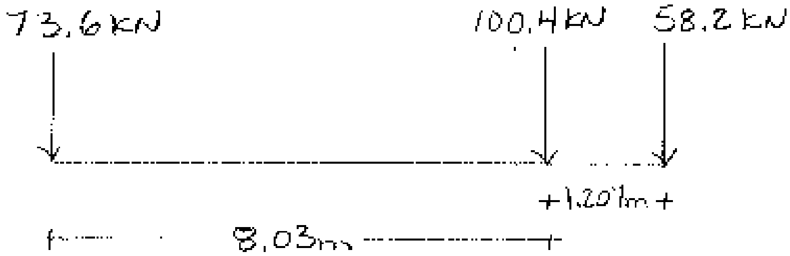


Figure 1 - Seven Girder Concrete Approaches 2 Span (S24-S26)
CL-3 Moment

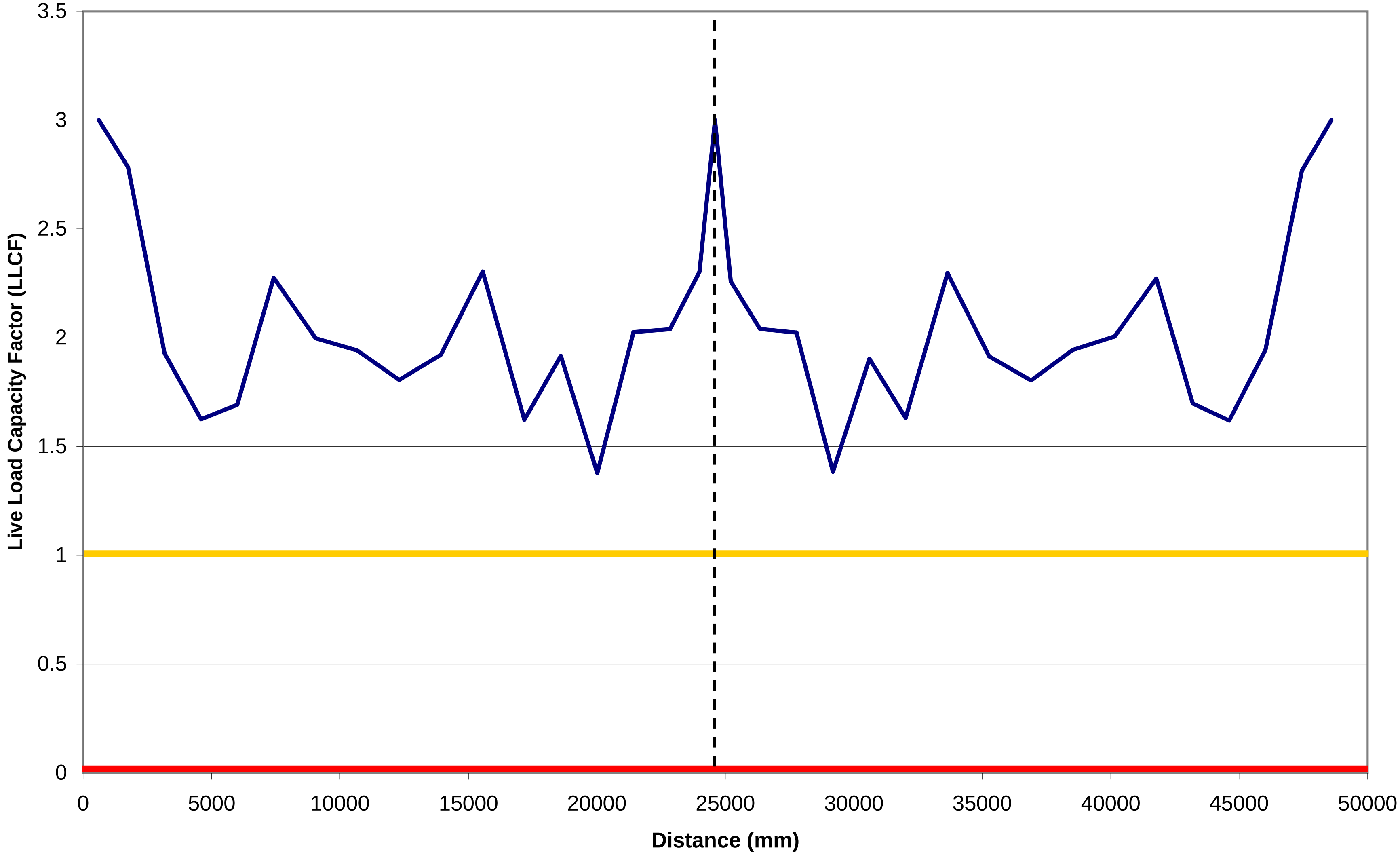



Figure 2 - Seven Girder Concrete Approaches 2 Span (S24-S26)
CL-3 Shear
Live Load Capacity Factor (LLCF)
3.5
3
2.5
2
1.5
1
0.5
0
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
50000
Distance (mm)

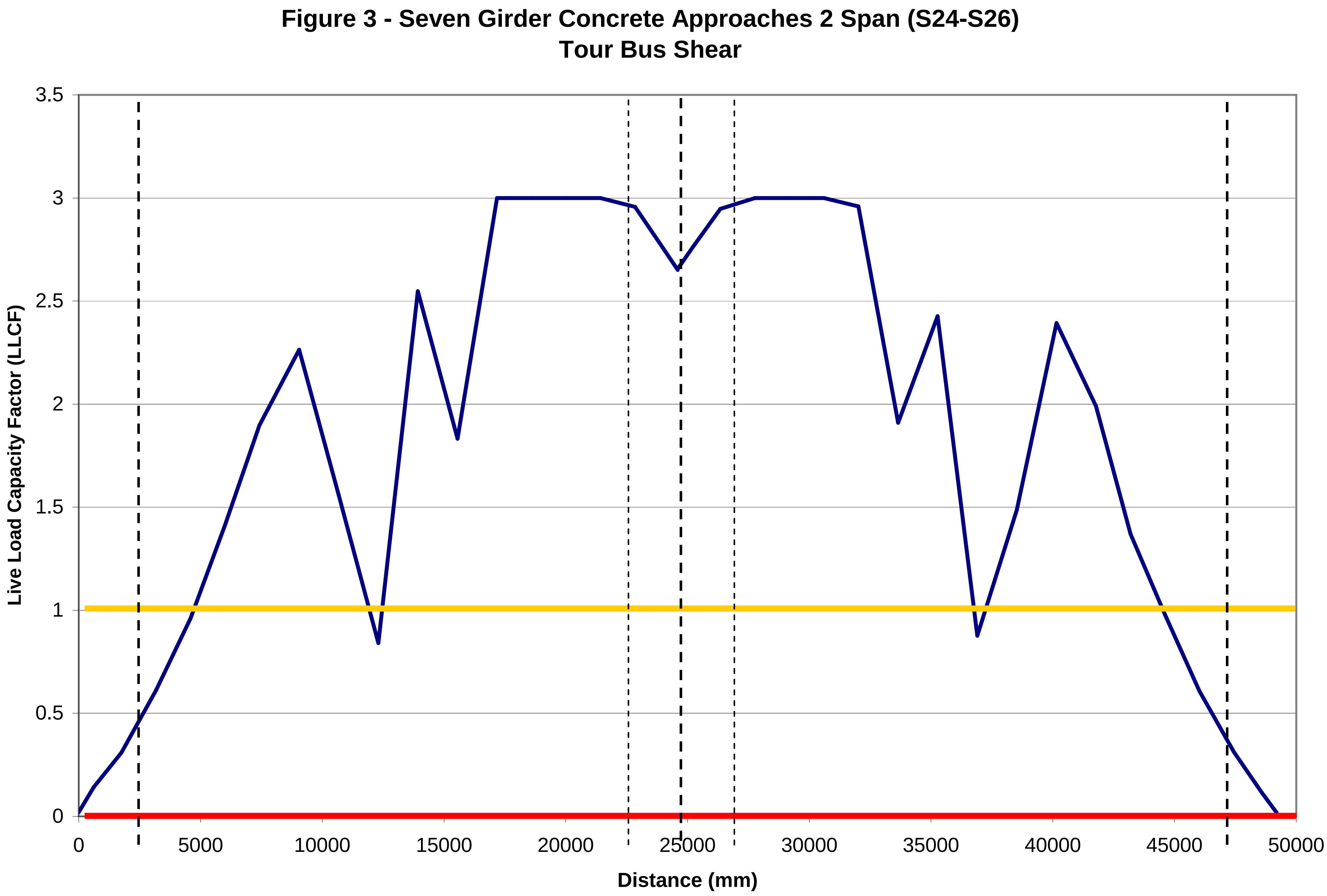
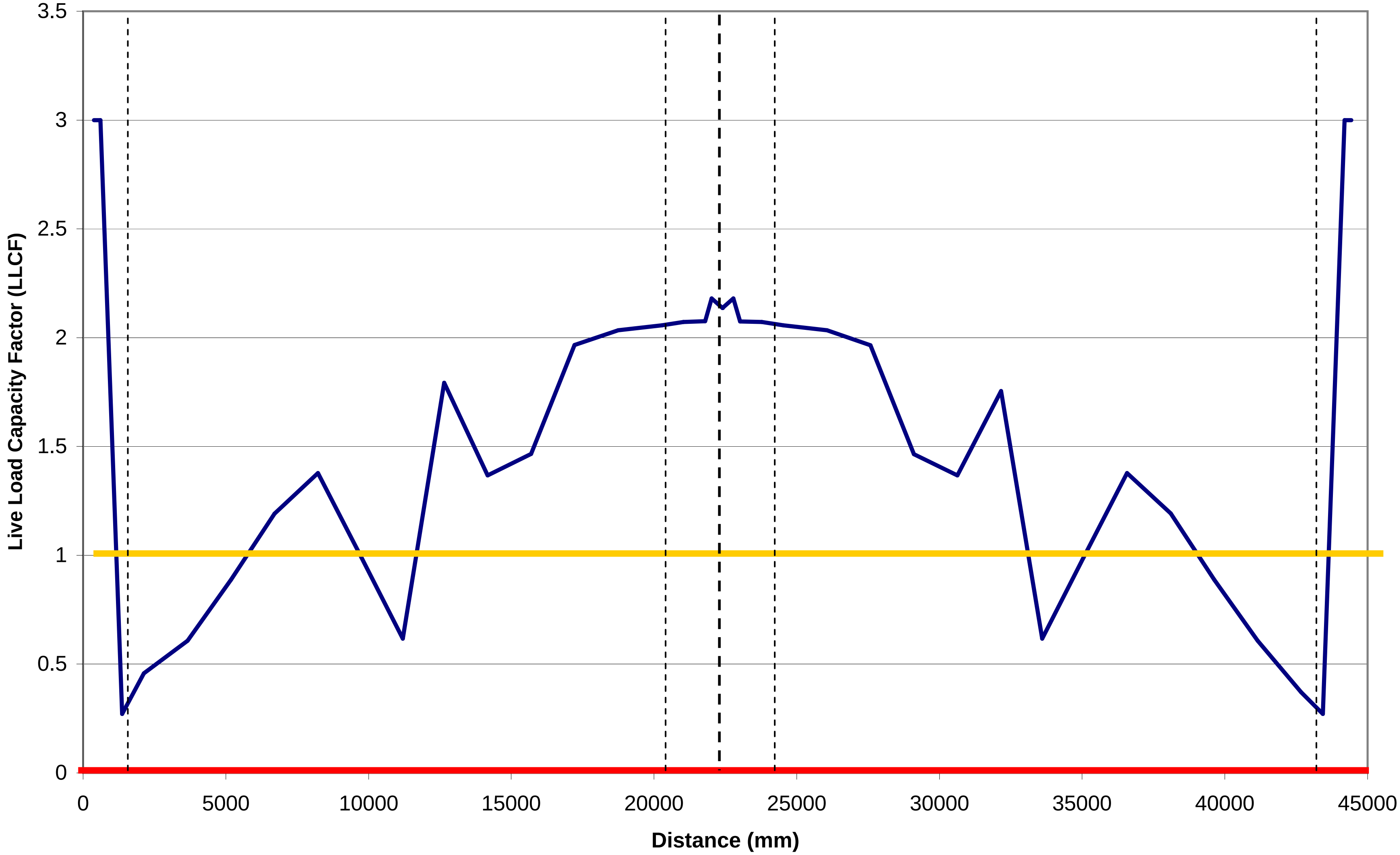


Figure 3 - Seven Girder Concrete Approaches 2 Span (S24-S26)
Tour Bus Shear
Live Load Capacity Factor (LLCF)
3.5
3
2.5
2
1.5
1
0.5
0
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
50000
Distance (mm)

Figure 4 - Five Girder Concrete Approaches 2 Span (N14-N16)
CL-3 Shear



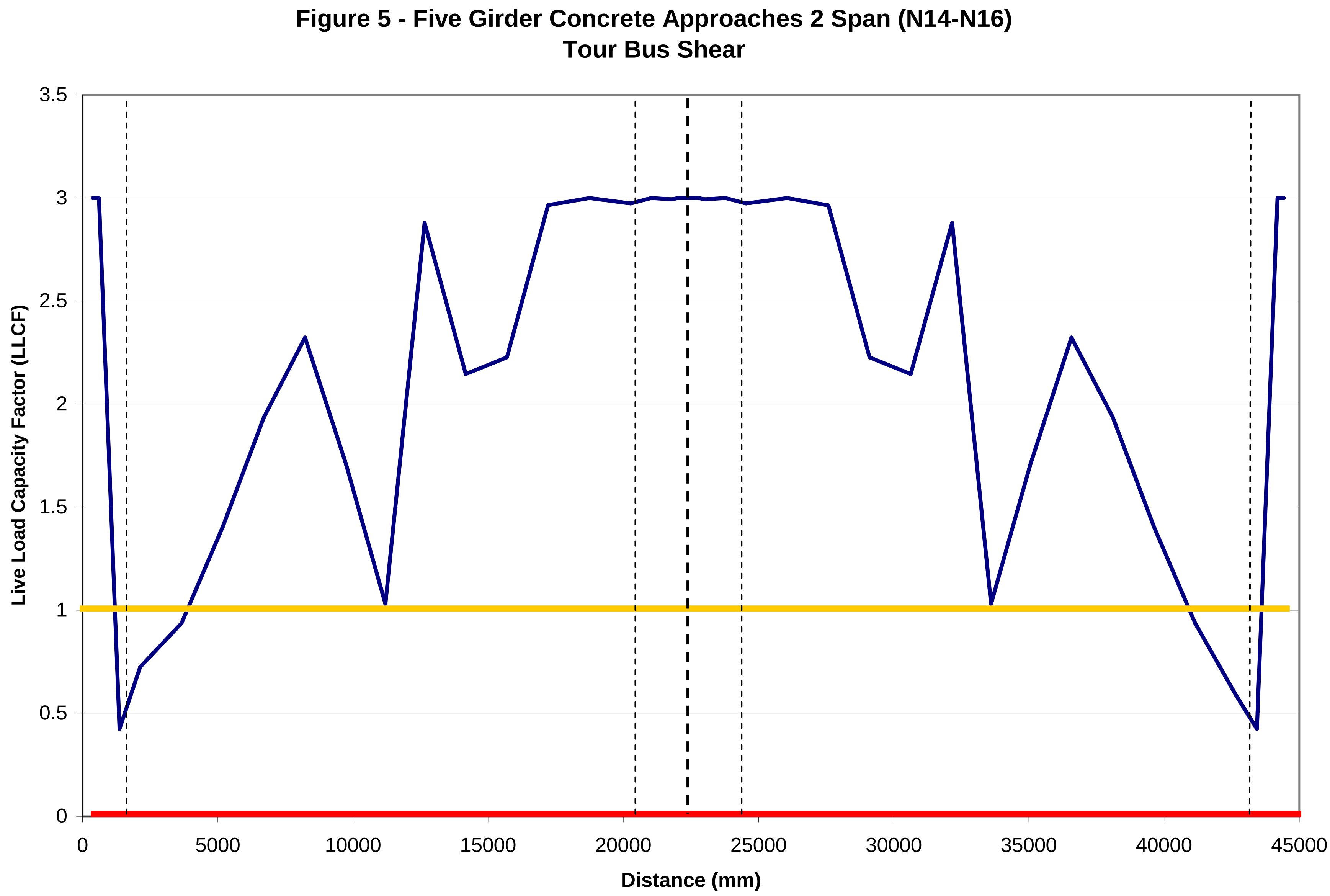
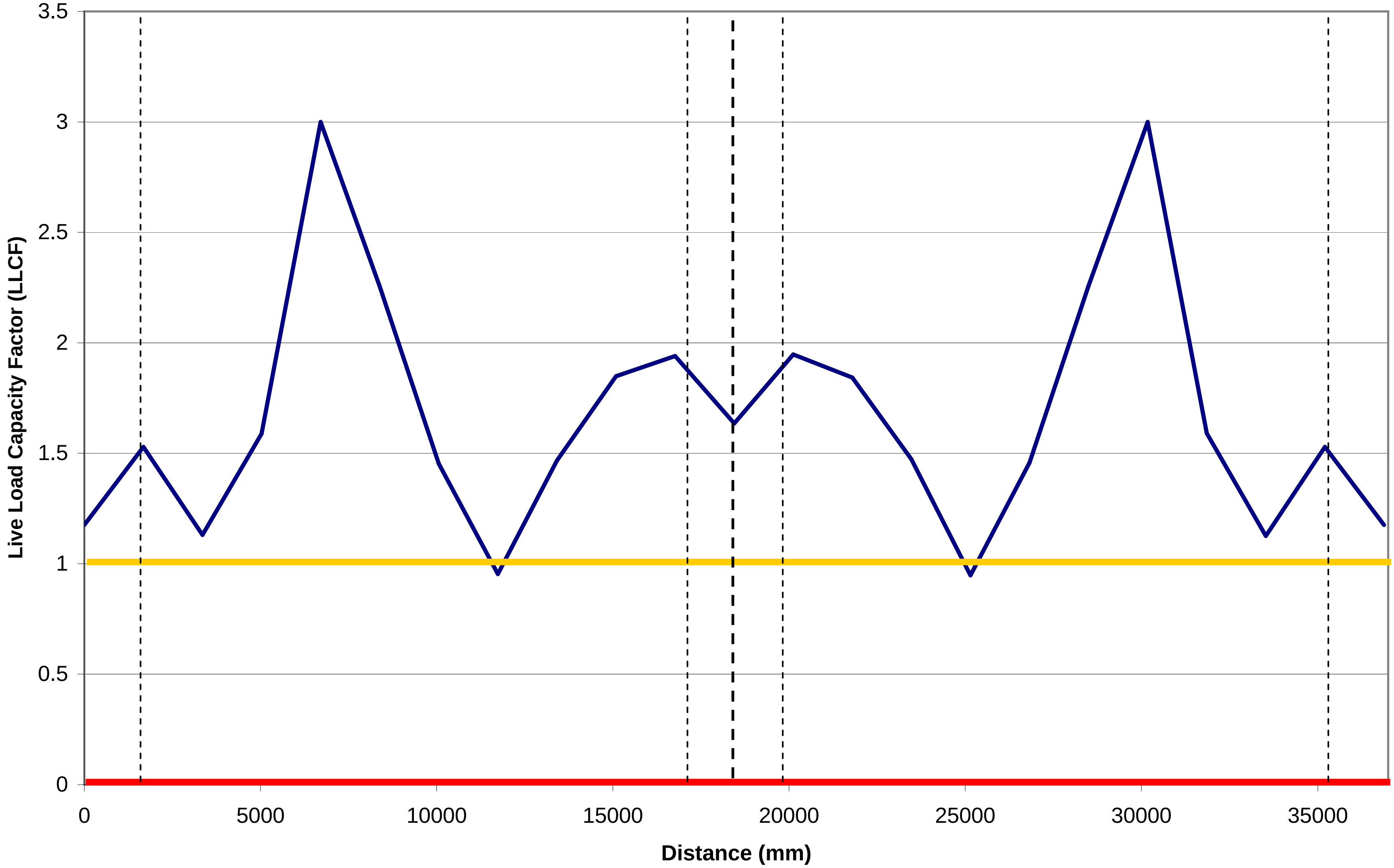


Figure 5 - Five Girder Concrete Approaches 2 Span (N14-N16)
Tour Bus Shear
Live Load Capacity Factor (LLCF)
3.5
3
2.5
2
1.5
1
0.5
0
0
5000
10000
15000
20000
25000
30000
35000
40000
45000
Distance (mm)


Figure 6 - Three Girder Concrete Approaches 2 Span (N52-N54)
CL-3 Moment
Live Load Capacity Factor (LLCF)
Distance (mm)
4
3.5
3
2.5
2
1.5
1
0.5
0
0
5000
10000
15000
20000
25000
30000
35000

Figure 7 - Three Girder Concrete Approaches 2 Span (N52-N54)
CL-3 Shear



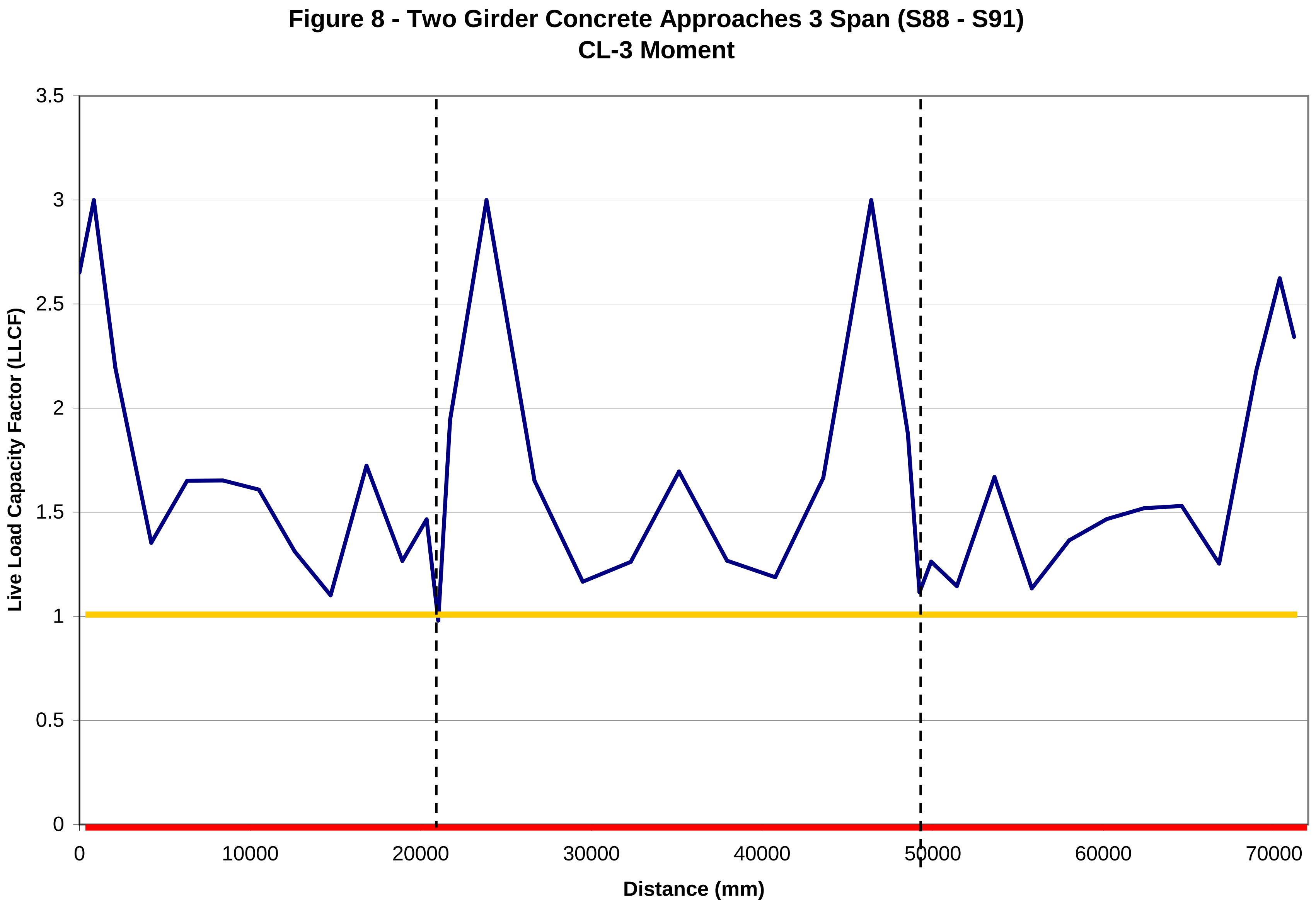


Figure 8 - Two Girder Concrete Approaches 3 Span (S88 - S91)
CL-3 Moment
Live Load Capacity Factor (LLCF)
3.5
3
2.5
2
1.5
1
0.5
0
0
10000
20000
30000
40000
50000
60000
70000
Distance (mm)

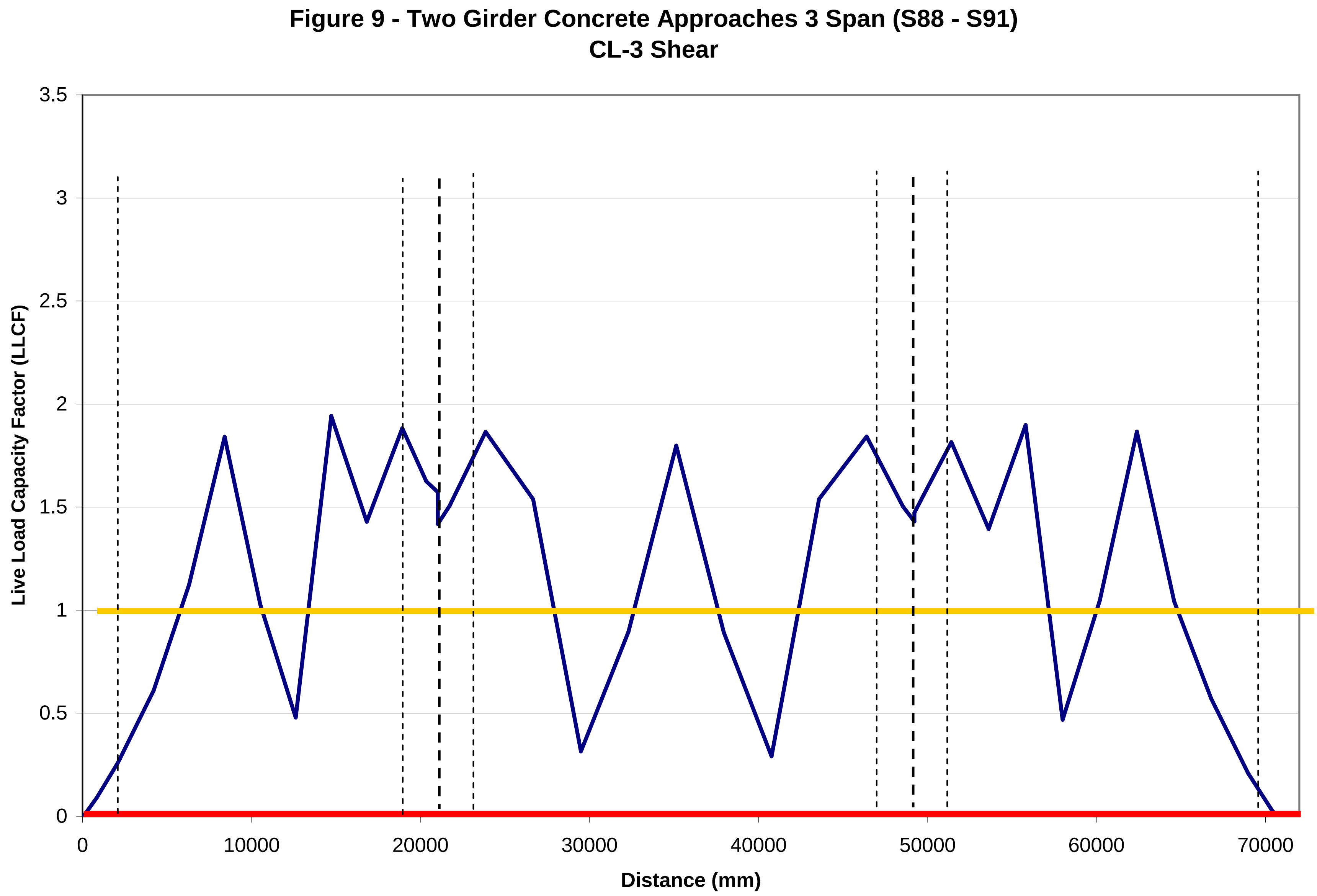


Figure 9 - Two Girder Concrete Approaches 3 Span (S88 - S91)
CL-3 Shear
Live Load Capacity Factor (LLCF)
3.5
3
2.5
2
1.5
1
0.5
0
0
10000
20000
30000
40000
50000
60000
70000
Distance (mm)

Figure 10 - Two Girder Concrete Approaches 3 Span (S88 - S91)
Tour Bus Shear