I-5 Skagit River Bridge Collapse Review

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Abstract: On May 23, 2013, a truck-tractor with a flatbed semitrailer hauling an oversized container was traveling south on Interstate 5 (I-5) near Mount Vernon, Washington. While crossing the Skagit River, the oversized container struck portions of the through-truss bridge, which resulted in a 49-m (160-ft) simple-span section of the 339-m (1,112-ft) bridge collapsing into the river. This collapse occurred even though the oversized container had a permit authorizing the travel route and was being escorted by a pilot vehicle. This paper (1) identifies some key transportation issues related to this scenario, e.g., permitting, route databases, signage, buffer between posted and actual clearance, insurance coverage, and pilot car requirements/operation; and (2) presents a structural analysis of the impact that provides insight into the truss design, damage sequence, failure mode, and potential techniques to increase the robustness of this class of bridge. **DOI: 10.1061/(ASCE)CF.1943-5509.0000913.**

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Introduction

On Thursday, May 23, 2013, at about 7:05 p.m. Pacific daylight time, a 2010 Kenworth truck-tractor pulling a 1997 Aspen flatbed semitrailer with an oversized steel casing shed [Fig. 1(a)] for Arctic-related drilling was traveling south on Interstate 5 (I-5) near Mount Vernon, Washington. The oversized vehicle had a Washington State Department of Transportation (WSDOT) permit for the travel route and was being escorted by a pilot vehicle, a 1997 Dodge Ram pickup truck. I-5 crosses the Skagit River on a bridge comprising 12 simple spans, with a total length of 339 m (1,112 ft). As indicated in Fig. 2, four central through-truss spans of the I-5 Skagit River Bridge are flanked by four approach girder spans on each of the north and south sides. The oversized container struck portions of the bridge [Fig. 1(b)] in Span 8, which resulted in this 49-m (160-ft) span of the bridge collapsing into the Skagit River (NTSB 2014). Two passenger vehicles, a southbound 2010 Dodge Ram pickup truck towing a Jayco travel-trailer and a northbound 2013 Subaru XV Crosstrek, fell into the river with the bridge (Fig. 3). Eight vehicle occupants were involved in the collapse, and although three received minor injuries, there were no fatalities. The collapse impacted local communities and west coast transportation for about 6 months.

Although a structural collapse due to an overheight load strike is the most severe outcome, overheight load strikes can cause severe structural damage that requires emergency closures that still adversely affect local communities. For example, in April 2015, the Washington State Route 410 (SR-410) White River Bridge sustained significant damage due to an overheight load strike (WSDOT 2015c, d). This bridge is a single-span through truss and the impact location and structural damage correlate closely with observations for the I-5 Skagit River Bridge. The strike event for the SR-410 White River Bridge was not observed, and the structural damage was discovered during a routine inspection. In general, overheight load strikes occur frequently, with a recent survey estimating that on average, roughly four overheight load strikes occurred per day in the United States during the 4-year period spanning 2005–2008 (Agrawal et al. 2011). The NTSB investigation of the I-5 Skagit River Bridge collapse (NTSB 2014) estimates that 90% of through-truss bridges in Washington State have experienced overheight load strikes over a 10-year period. Specific to the Skagit River Bridge, a WSDOT bridge inspection report (WSDOT 2012) shows the northbound direction of Span 5 had evidence of a significant overheight load strike, causing a partial tear and deformation of the portal and some subsequent or northern sway frames.

This paper investigates some transportation-related issues pertaining to the I-5 Skagit River Bridge overheight load strike and subsequent collapse, such as permitting, route databases, signage, buffer between posted and actual clearance, insurance coverage, and pilot car requirements and operation. In addition, a structural analysis incorporating the shed impact force provides further insight into the truss design, damage sequence, failure mode, and potential techniques to increase the robustness of this class of bridge.

Transportation-Related Issues

This section of the paper discusses some of the key transportation-related issues associated with the I-5 Skagit River Bridge collapse

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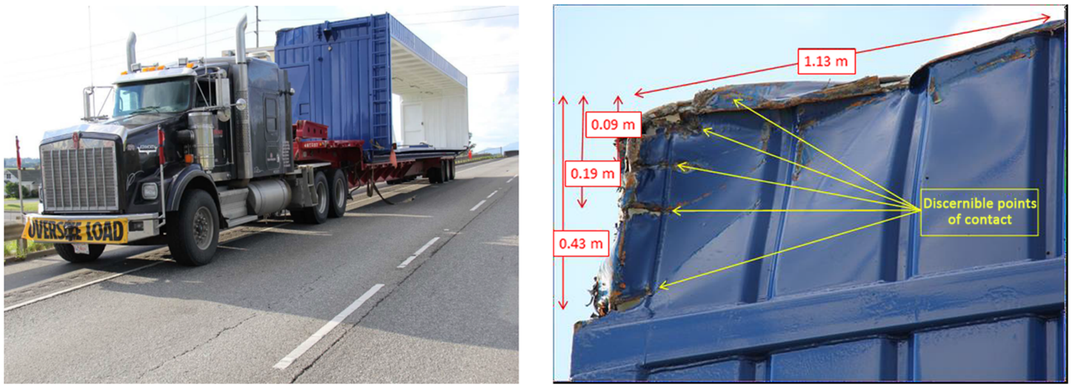

(a) (b)

Fig. 1. Overheight truck photographs (reprinted from NTSB 2014): (a) overall view with container; (b) detail of damage to oversized container

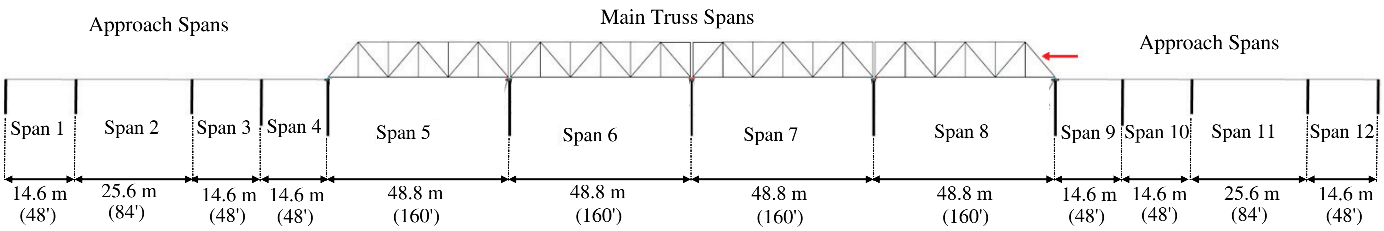

Fig. 2. I-5 Skagit River bridge elevation

Fig. 3. Bridge Span 8 collapsed in Skagit River with vehicles involved identified (reprinted from NTSB 2014)

including vertical clearance, vehicle permitting, route databases, clearance signage and buffer, pilot car requirements, insurance coverage, and best practices to avoid future collapses.

Vertical Clearance

A major factor in this incident is the vertical clearance of the bridge, in terms of both magnitude and variability. The bridge has four traffic lanes (two southbound and two northbound) with a 0.7-m (2.29 ft) shoulder on each side, for a total width of 18.4 m (60.37 ft). Fig. 4 presents a drawing of the I-5 portal frame, which is the southbound entrance point of the bridge. The published vertical clearance over the inside fog line (the left edge of the left-most south travel lane, i.e., near the bridge center) is 5.4 m (17.71 ft), while the actual vertical clearance at this location is slightly higher at 5.5 m (18 ft), For comparison purposes, the published vertical

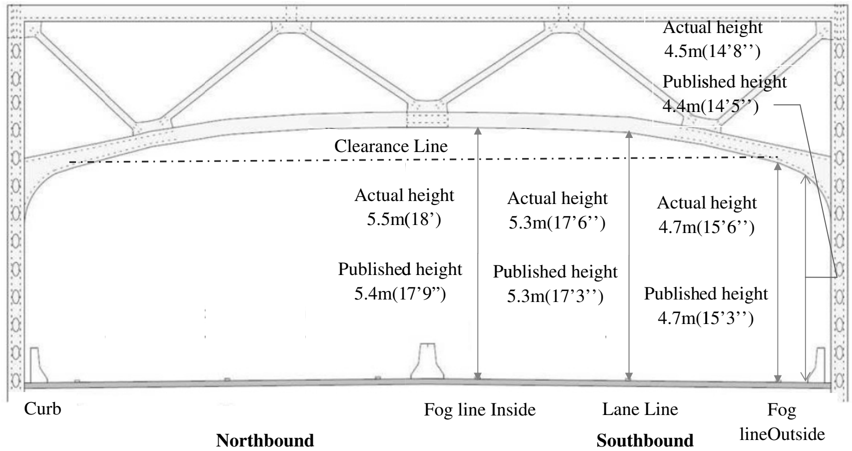

Fig. 4. Published vertical clearance values of portal frame for the I-5 Skagit River Bridge

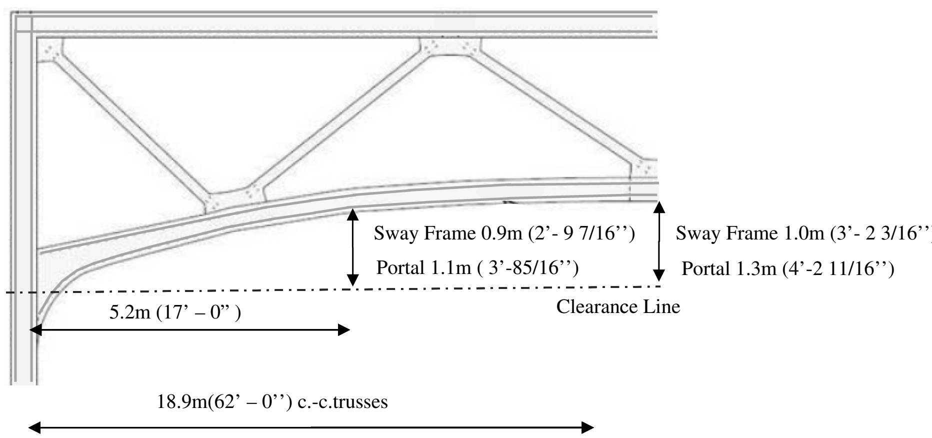

Fig. 5. Elevation of first sway frame on the I-5 Skagit River Bridge

clearance over the outside fog line [right edge of the right-most south travel lane, i.e., near the bridge edge, is only 4.4 m (14.44 ft), as shown in Fig. 4, while the actual vertical clearance at this location is slightly higher at 4.5 m (14.76 ft)].

Finally, the published vertical clearance over the lane line (line dividing the southbound approach into two lanes) is 5.30 m (17.39 ft), while the actual vertical clearance at this location is also slightly higher at 5.33 m (17 ft-6 in.). The truck shown in Fig. 1 had a permit for a maximum vertical clearance of 4.8 m (15.74 ft) and width of 3.5 m (11.5 ft), which clearly exceeded the published and actual heights at the right edge of the right-most south travel lane by over 0.3 m (1 ft).

Fig. 4 also shows the bridge had a variable vertical clearance so these different vertical clearances should be considered in the permitting process for oversize vehicles. A truck with 4.8 m (15.74 ft) vertical height exceeds the vertical clearance of the bridge over a portion of the right lane and all of the right shoulder, while it does not exceed the vertical clearance of the bridge over the left lane. If the truck was travelling in the left lane, the bridge impact probably would not have occurred. The permitting process and bridge signage should illustrate this variable vertical clearance, as discussed subsequently.

Fig. 4 shows the clearance line, which corresponds to a line above the entire width of the traveled lanes (not including sidewalk), that exceeds a minimum vertical clearance. The published clearance line is 4.7 m (15.41 ft), which is less than the vertical height of the truck, i.e., 4.8 m (15.74 ft). As a result, if the permit was based on the clearance line, it would have indicated that the truck exceeded the height of the clearance line. As discussed later, all of the actual vertical dimensions in Fig. 4 are 76 mm (3 in.) greater than the published value to create a vertical clearance buffer of 76 mm (3 in.).

In addition to variable vertical clearance across the portal frame, variable vertical clearance also existed along the I-5 Skagit River Bridge. The vertical clearance at the entrance arch (portal frame in Fig. 4) is higher than the vertical clearance at the first interior arch (first sway frame), which is shown in Fig. 5. As will be discussed later, the bridge collapse initiated when the first sway frame was impacted by the oversized container. Fig. 5 shows a sway frame that is typical for Spans 5, 6, 7, and 8 of the bridge inboard from the portal shown in Fig. 4. The sway frame has significantly lower vertical clearance over the travel lanes than does the portal frame because the sway frame has a flatter curvature than the portal frame. For example, the sway frame has a vertical clearance of only 0.9 m (2.95 ft) above the clearance line (Fig. 5), while the portal frame has a maximum vertical clearance of 1.1 m (3.31 ft) above the clearance line (Figs. 4 or 5). The permitting and signage for this bridge were based on the vertical clearance of the portal frame and not the sway frame, which is unfortunately the limiting case. In summary, the signage should reflect the variable and minimum vertical clearance and not only the portal entrance clearance.

Fig. 4 also shows the vertical clearance at the outside fog line is only 4.5 m (14.76 ft). As a result, WSDOT did not have any signage posted prior to the bridge about vertical clearance because it only posts signage for vertical clearance less than 4.65 m (15.26 ft) as discussed subsequently. WSDOT published the vertical clearance at the outside fog line as 4.4 m (14.43 ft), which provided a 76-mm (3-in.) vertical clearance buffer. WSDOT also did post reduced lane width signage even though the southbound lanes narrowed from the roadway width of 3.7 m (12 ft) to 3.5 m (11.48 ft) on the bridge.

Along a permitted route, different lanes may have different clearances. In the case of the I-5 Skagit River Bridge, the vertical clearance seems adequate in the left lane, but not in the right lane. This was particularly important in this case because the truck shown in Fig. 1 could not move to the left lane because it was being passed by another 18-wheel tractor-trailer. In addition, the lane width decreased from 3.7 m (12 ft) to 3.5 m (11.48 ft) within the bridge. This resulted in the oversized vehicle having to travel 50 mm (2 in.) over the outside fog line, i.e., on the right shoulder. This was problematic because the sway frame in Fig. 5 curves downward over the outside fog line, which further reduces the vertical clearance. It is recommended that oversized permits warn permit holders about variability in vertical clearances across lanes and shoulders and along the bridge.

Vehicle Permitting

Permitting of oversized vehicle routes is not federally regulated so states establish their own permit requirements. Many states have online permitting systems (such as WSDOT via http://www.wsdot.wa.gov/CommercialVehicle/self_issue.htm) that allow private companies to create their own oversize vehicle permits without WSDOT assistance. WSDOT places the responsibility of acknowledging vertical clearance issues on the trucking companies, which is common in many other states. However, an issue as important as vertical clearance for oversize vehicles should have agency oversight, such as a system that actually checks permits for accuracy and safety. An example of this is the Illinois Department of Transportation (IDOT 2016) system that allows trucking companies to obtain oversize vehicle permits online but then IDOT technicians manually check any permit that exceeds specific load dimensions or weights to ensure the permitted route will have adequate clearance.

In this case, WSDOT approved Mullen Trucking in Alberta, Canada to carry a load as high as 4.8 m (15.74 ft) based on it being less than the maximum usable vertical clearance of 5.3 m (17.39.). However, the vertical load height is greater than the minimum southbound vertical clearance of 4.35 m (14.27 ft) listed in the WSDOT Bridge List (WSDOT 2015c, d). This potential for inadequate vertical clearance was not recognized.

Route Databases

Some transportation agencies establish their own clearance databases to assist in safety measurements for online route assignments. An example of this is IDOT's Illinois Transportation Automated Permit (ITAP) system. The ITAP system (IDOT 2016) contains actual vertical clearance information throughout the state that was collected by Lidar technology. The ITAP system does not allow trucking companies to obtain permits for routes that involve travel under structures if the net clearance is less than 75 mm (3 in.). However, the ITAP system only contains the lowest vertical clearance for the structure from edge line to edge line, and it does not provide any information about variable clearance for different lanes. To raise confidence in safe route assignment, agencies that have or are creating these route databases should collect and provide information about vertical clearances that do not remain constant over and along the roadway, as well as vertical clearance of structures over the shoulders of the roadway. More importantly, the databases should be periodically updated because changes in vertical clearance can occur with time. For example, bridge or roadway construction can change the vertical clearance of a structure so the new clearance should be measured and recorded in the database.

For example, WSDOT posts a bridge list on their website that lists the usable vertical clearance of each bridge which is 7.6 mm (3 in.) less than the actual vertical clearance to create a vertical buffer. However, WSDOT states these usable vertical clearances are not guaranteed for accuracy, due to continuing construction activities

Clearance Signage

Clearance signage is a last resort to preventing vehicles from striking overhead structures because the signage attempts to inform oversize and escort vehicle drivers of the vertical clearance for an upcoming structure. Bridge clearance signage is another aspect of vertical clearance safety that varies from state to state. The federal requirement (FHWA 2014) is that states must place signage on structures that are 4.3 m (14 ft) or less in vertical clearance because the clear height of structures shall not be less than 4.3 m (14 ft) on urban interstate routes. Washington state also statutorily requires signage for structures that are 4.3 m (14 ft) or less in vertical clearance. The Manual on Uniform Traffic Control Devices (MUTCD) (FHWA 2012) by the Federal Highway Administration (FHWA) recommends signing any overhead structure that is not more than 0.3 m (1 ft) greater than the maximum vehicle height not requiring an oversize vehicle permit. Many states only sign structures that are 0.3 m (1 ft) greater than their maximum vehicle height as recommended by the MUTCD.

The maximum vehicle height that does not require a permit also varies by state from 4.2 to 4.6 m (13.78 to 15 ft). Some states sign all structures that are as much as 0.9 m (3 ft) greater than their maximum vehicle height, which results in more structures being marked. Prior to the I-5 incident, WSDOT required that all structures less than 4.4 m (14.44 ft) of clearance were to be signed; however, after the I-5 collapse, a new law (HB 5944, dated May 31, 2013) requires all structures with less than 4.9 m (16 ft) of vertical clearance to be signed.

Clearance Buffer

To reduce risk and account for variability in vertical clearance measurements, agencies record vertical clearances that are less than their actual measured heights. These clearance buffers are not reflected in the bridge signage, databases, or the dimensions shown in Fig. 4. These clearance buffers are used to account for variability in circumstances such as vehicles bouncing vertically while traveling on the roadway, snow or other weather conditions decreasing the vertical clearance, and measurement accuracy. WSDOT and other state transportation agencies (e.g., IDOT) have a vertical clearance buffer of 76 mm (3 in.). Some other agencies such as the Indiana Department of Transportation (INDOT) have clearance buffers of only 50 mm (2 in.). Agencies like IDOT that are using Lidar technology to accurately record the vertical clearances of structures are considering decreasing their clearance buffer from 76 to 50 mm (3 to 2 in.).

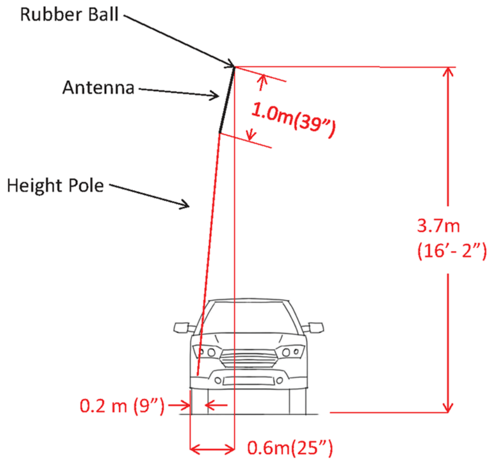

Fig. 6. Diagram of the I-5 bridge pilot/escort vehicle height

Pilot Car Requirements

Pilot vehicles provide an additional layer of redundancy for identifying vertical clearance concerns while an oversize vehicle is traveling on its permitted route. Pilot vehicles also alert surrounding vehicles to the oversize vehicle on the roadway. State agencies require oversize vehicles to have a pilot vehicle if the oversize vehicle exceeds a specific height or load, and in some cases require an additional escort vehicle behind the oversize vehicle as well. Although pilot vehicle requirements vary across states and vehicle operators typically do not need a special license, the FHWA (2015) provides useful information about pilot vehicles. This guideline is developed by the Specialized Carriers & Rigging Associations for the FHWA. In the I-5 Skagit River Bridge collapse, the escort vehicle driver is reported to have been using a cellular telephone while operating the vehicle at the time of the accident (NTSB 2014) and did not alert the oversize vehicle to the variable vertical clearance on the bridge. As a result, the NTSB (2014) recommends that the only electronic device communication permissible during escort should be between the oversize vehicle driver and the pilot vehicle driver.

WSDOT requires that escort vehicles install a height rod to at least 76–152 mm (3–6 in.) above the height of the oversize vehicle. In the I-5 Skagit River Bridge incident, the escort pole was measured to be 4.9 m (16.08 ft) for a load height of 4.8 m (15.75 ft), which is 127 mm (5 in.) greater than the load height (Fig. 6). However, the pole was installed at an angle (Fig. 7) so it was not 127 mm (5 in.) higher than the load height of 4.8 m (15.75 ft) at the right edge of the load.

The *Pilot Car Escort Training Manual* (FHWA 2004) displays the mounted pole on the right side of the vehicle (as was done in the I-5 case, as shown in Fig. 6), however many state agencies do not require a specific location for it. Not only must the pole be at the right location, but it also needs to be installed vertically to capture the full height of the oversize load. As a result, NTSB (2014) recommends that AASHTO develop a procedure so pilot/escort vehicles replicate the maximum dimension of an oversize load.

Another aspect of pilot vehicles that should be addressed is an effective following distance. If the pilot vehicle is too closely preceding the oversize vehicle, then it will not provide adequate time to inform the oversize vehicle of potential problems and for the vehicle to stop or change lanes. Based on NTSB (2014) estimates, the pilot vehicle was traveling about 400 ft (122 m) ahead of the oversize load, or about 5 s ahead based on vehicle speeds. This amount of time may not be adequate to effectively inform an oversize vehicle operator to change lanes or not to enter a structure prior to an incident occurring. Another motivation for having a longer preceding escort distance is that if the height pole makes contact with a structure, then there must be adequate roadway for the oversize vehicle to change lanes, exit the freeway, or change routes prior to approaching the structure in question.

Related to safe preceding distance is the need for an effective way for pilot vehicles to identify a potential vertical clearance issue. Because the following distance of an oversize vehicle is within a short time frame, an apparatus that clearly identifies the potential for a pilot-vehicle-mounted pole to hit a structure is extremely important. NTSB (2014) reports that a witness of the I-5 accident believed the pilot vehicle rod struck several components of the bridge, but the pilot vehicle driver reported the vehicle pole did not strike the I-5 Skagit River Bridge (although the pilot vehicle driver was engaged in a cellular telephone conversation, which may have distracted the driver from recognizing the vehicle pole hitting some of the bridge frames). As a result, a more reliable and effective method for informing the pilot vehicle driver that the height pole has struck a structure is desired. It is recommended that pilot vehicle height poles be equipped with technology that simultaneously and immediately informs the pilot and oversize vehicle drivers that a

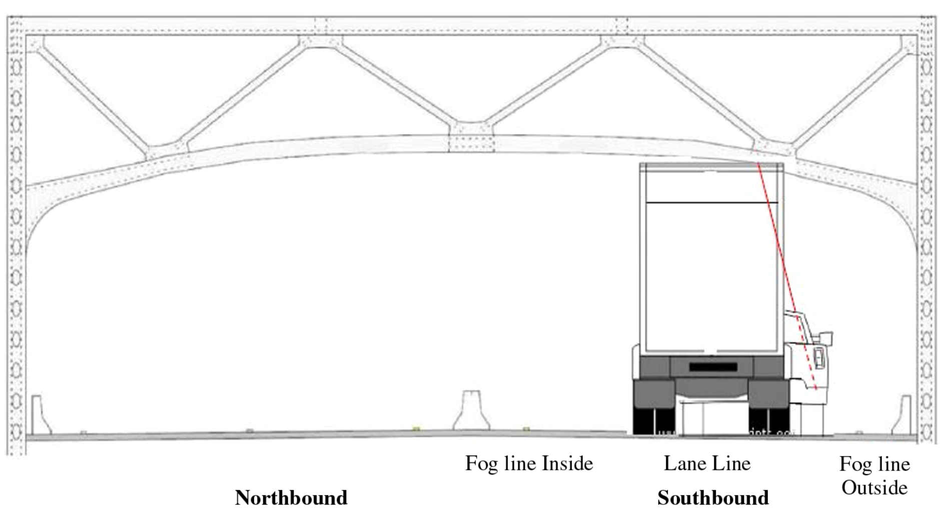

Fig. 7. Pilot vehicle vertical measurement compared to oversize load (after NTSB 2014)

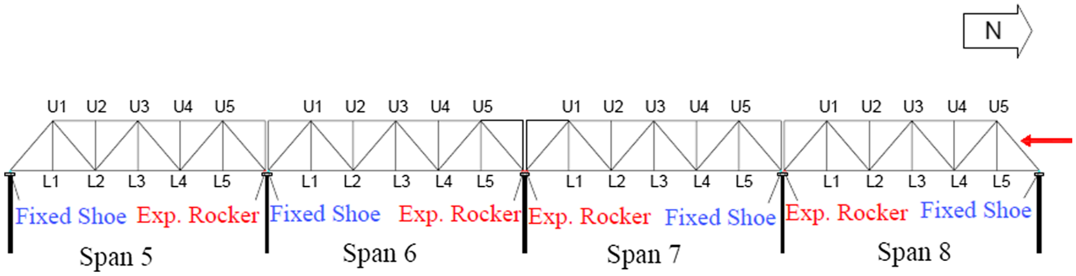

Fig. 8. Elevation view of the four truss spans

structure element has been hit by the height pole so the pilot driver does not have to relay the information to the oversize vehicle driver.

Insurance Coverage

Insurance coverage is another important factor that agencies need to consider for both pilot vehicles and trucking companies. In the case of the I-5 Skagit River Bridge incident, the pilot car company was only required to carry \$1 million worth of insurance coverage, while the bridge repair costs exceeded \$15 million (which does not include economic losses by the business community). IDOT requires trucking companies to have at least \$500,000 worth of coverage for bodily injury and property damage, as well as \$1 million for bridge repair, but these amounts are also significantly less than the I-5 bridge damages. The insurance coverage also should include some value for economic loss that is usually experienced by the local economy.

Structural Evaluation

This section discusses the truss design, a nonlinear structural analysis of the truck impact on the I-5 Skagit River Bridge, a postulated damage sequence, and potential mitigation strategies to reduce bridge vulnerability. This section is divided into the following subsections: "Truss Design and Construction," "Truss Geometry and Load Changes with Time," "Structural Numerical Model," "Vehicle Loadings," "Critical Member Failure Mechanism," "Collapse Sequence Analysis," and "Remedial Measures."

Truss Design and Construction

The I-5 Skagit River Bridge comprises 12 spans and 13 piers, with a total length of 339 m (1,112 ft) from the south to the north abutments. The bridge consists of four south and four north approach girder spans, as well as four main truss spans. The main truss portion of the bridge over the Skagit River has four independent simply-supported Warren-type through trusses. All four truss spans are made of riveted steel sections and have span lengths of 48.8 m (160 ft). Fig. 8 presents an elevation view of the four truss spans with the structural elements labeled.

Expansion rocker and fixed shoe bearing types are used at the ends of each truss span, as shown in Fig. 8. Each truss span includes six panels, and the length of each panel is 8.1 m (26.57 ft). The east and west trusses are spaced 18.9 m (62 ft) apart, and they are connected transversely by seven sway frames above the roadway and seven floor beams below the roadway for Spans 6 and 7, and five sway frames and one portal for the entrance spans, i.e., Spans 5 and 8. Sway frames located at the north and south entrances are called portals, compared to the other sway frames in the truss. The portals have the same brace pattern but are transversely connected by two diagonal members at the entrance. The lower chord of the inner sway frames have a curved (elliptical) shape across the roadway, as shown in Fig. 4. A reinforced concrete deck is supported by nine W-shaped stringers, spaced at 2.1 m (6.89 ft). The deck was constructed with a 152-mm (6-in.) thick concrete deck in 1955.

Structural carbon and low-alloy steels were used for the truss elements, conforming to ASTM A36 (ASTM 2014) and A242 (ASTM 2013) specifications, respectively. The AISC *Design guide 15* (AISC 2003) shows the specified minimum yield stress for ASTM A36 steel is 228 MPa (33 ksi) and that the ultimate strength varies between 414 and 496 MPa (60 and 72 ksi). A242 steel has a specified minimum yield stress of 345 MPa (50 ksi) and an ultimate strength of 483 MPa (70 ksi). The structural rivet steel conformed to the ASTM A502 (ASTM 2015b) specification. All floor beams and truss members used A242 steel, with the exception of four nominally zero-force members (U0-L0, U2-L2, U4-L4, and U0-U1) and two relatively lightly loaded diagonal members (L2-U3 and L4-U3) in each truss that used ASTM A36 (ASTM 2015a) steel (Table 1).

Table 1 presents the built-up box and I-shape sections that were used in the main trusses. Eight diagonal compression members and the top and bottom chords are box sections comprising two channels and continuous cover plates perforated with a succession of access holes. The access holes led to various cross sectional

Table 1. Main Truss Members for Structural Analysis

Truss member designation	Member type	Section type
L2-L4	$2C15 \times 50 + 2PL5/16 \times 14$	Built-up box
L0-L2, L4-L6, U0-U1	$2C15 \times 33.9 + 2PL5/16 \times 14$	Built-up box
U1-U5	$2C15 \times 40 + 2PL3/8 \times 14$	Built-up box
U1-L0, U5-L6	$2C15 \times 40 + 2PL7/16 \times 14$	Built-up box
U3-L2, U3-L4	$2C12 \times 25 + 2PL5/16 \times 14$	Built-up box
U1-L2, U5-L4	$4\ L6 \times 3.5 \times 3/8 + 1PL5/16 \times 14$	Built-up I shape
U1-L1, U3-L3, U5-L5	$4\ L5 \times 3.5 \times 3/8 + 1PL3/8 \times 14$	Built-up I shape
U2-L2, U4-L4	$4\ L5 \times 3.5 \times 5/16 + 1PL5/16 \times 14$	Built-up I shape

Note: C = C-shape (channel); L = L-shape (angle); PL = plate.

properties along these members, so for the purpose of modeling, the cross sections assigned to them were reduced from the gross values. The perforated box members satisfied all requirements described in AISC specification (AISC 2010) section B4.1, so the unsupported width of the plates was assumed to contribute to net cross sectional properties in the model.

Truss Geometry and Load Changes with Time

Although the main truss structure had not changed since completion of the bridge in 1955, modifications affecting the self-weight of the bridge and clearance between the roadway and overhead structure have occurred. In 1975, asphalt surfacing with a thickness of 4.6 cm (1.8 in.) was placed above the original deck (WSDOT 2014). In 1992, this asphalt overlay was removed and replaced using a microsilica-modified concrete overlay with a thickness of 3.8 cm (1.5 in.) over the original concrete deck. Since 1.3 cm (0.5 in.) of the original concrete deck was removed with the asphalt in 1992, the net increase in deck thickness due to these overlays is 2.5 cm (1 in.). In addition, all of the original side rails, the centerline barrier, and shoulder curbs along the two sides of the deck were replaced in 1992 with more substantial precast concrete traffic barriers.

Structural Numerical Models

Finite-element (FE) truss models [two dimensional (2D) and three-dimensional (3D)] of Span 8 were developed and analyzed to study truss response to the gravity and vehicle impact loads. The 2D model was used to establish a simple benchmark and the 3D model was subsequently used to conduct the primary analyses. The basic member properties and loads for the east and west main trusses are nearly identical, so the assumption of symmetry is reasonable for the 2D model representing one of the main trusses. Because the individual truss spans act independently from each other, and no redundancy was provided to Span 8 from other trusses, Span 8 was modeled independently. Secondary members (such as stringers, floor beams, and sway frames) were not explicitly included in the 2D model, but self-weight of these secondary members (as well as of the concrete deck) were included as point loads acting on the truss. Truss elements were used for all steel components in the 2D structural model, so only axial behavior was considered. The south end of the truss was supported by a roller and the north end was modeled as simply-supported (Fig. 8).

Fig. 9 shows a typical three-dimensional (3D) finite-element (FE) model, created using *MASTAN2* version 3.5 (McGuire et al. 2000), which considers material and geometric nonlinearities in the analysis. The geometry and material properties for 3D models were created using the engineering design and construction drawings. This model incorporates all significant structural steel elements, including the main trusses, sway frames, top and bottom lateral braces, stringers, and floor beams. The weight of the concrete deck was also included but any composite action between the steel structure and concrete slab was neglected due to the discontinuity of longitudinal stringers across the floor beams. The 3D beam elements, with lumped plasticity modeled through hinges at element ends, were used for all structural steel components. These elements consider axial–flexural interaction, including both strong and weak axis flexure, but potential reductions in strength due to local buckling are not considered.

Due to the eccentric nature of the impact force on the lower chord of the sway frame, local torsional effects were indirectly considered. Because the torsional rigidity is small compared to warping rigidity for this member, the twisting load is essentially resisted by flange bending. Hence, maximum stress (due to flexure plus torsion) in the cross section with the eccentric impact force is conservatively taken to be twice that of a case with a concentric impact force. To capture this difference caused by torsion, the plastic section modulus of this member was conservatively reduced by one-half. Additionally, to ensure the eccentric load would be globally reflected in the rest of the system, a 15.2-cm (6-in.) rigid element was attached to the lower chord of the sway frame and the horizontal impact force was applied at the tip.

Sway frame members and longitudinal stringers were modeled with pinned ends due to the relatively flexible connections that were employed. The riveted-steel connections between main truss members were assigned to be rigid based on the connection depths and their ability to transmit significant bending moments. Support conditions at both ends of the 3D model are the same as those applied in the 2D model and are shown in Fig. 9.

The overall loading condition of the collapsed truss span was estimated from the design drawings and the NTSB accident report (NTSB 2014). Vehicle live load was estimated at the time of collapse, and the dead load, which included superimposed dead load (SDL) and the weight of the steel structure and concrete slab, was estimated after initial construction and at the time of collapse (considering the previously described bridge modifications over time). The total dead load of Span 8 after original construction was estimated to comprise (1) self-weight of structural steel and concrete slab [5,640 kN (1,268 kips)], (2) SDL weight of the original rail posts and curbs [417 kN (93.7 kips)], and (3) an additional estimated overall SDL [246 kN (55.2 kips)], which corresponds to 287 N/m^2 (6 psf).

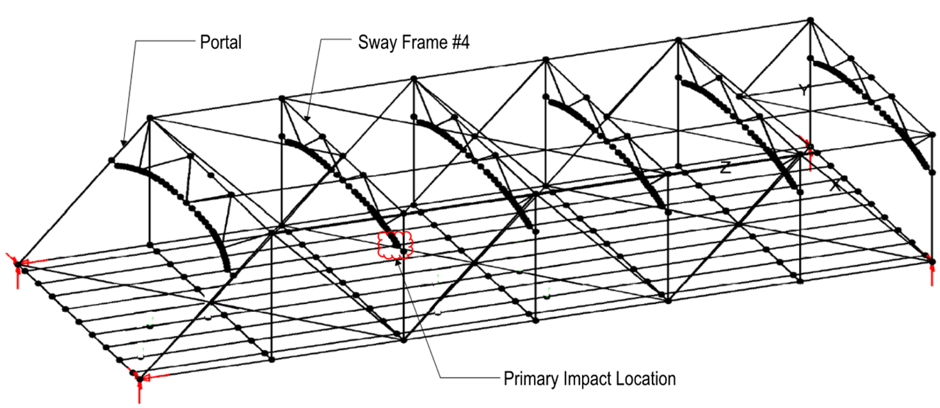

Fig. 9. Three-dimensional numerical model of Span 8

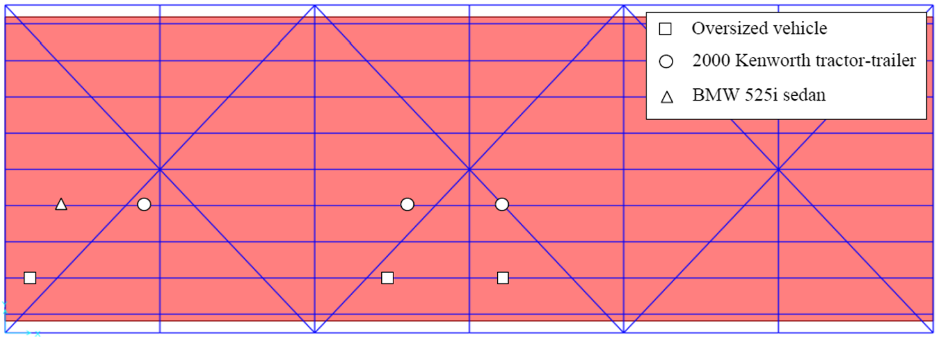

Fig. 10. Plan view of Span 8 showing location of vehicles at time of impact

The 2D and 3D numerical models were validated by comparing truss member forces obtained from the models to those reported on the engineering design drawings. The total reaction forces at the pier supports obtained using the 2D and 3D models are within 1% of the design drawing values. In addition, truss member forces obtained from the 2D model are in agreement with the design drawings, with a maximum difference of 3%. In the 3D model, the axial forces in the majority of the members are consistent with the engineering design values but small differences (less than 7%) appear in the bottom and top chords because lateral bracing systems on the top and bottom planes of the through truss take some modest tension or compression force from the truss chords.

Vehicle Loadings

A critical member failure evaluation was performed based on the collapse loading condition, i.e., existing dead load and vehicle locations. The primary change in self-weight dead load over time was due to modification of the concrete slab and overlay, with a net weight increase of 512 kN (115 kips) for each span since 1955. The overall SDL also increased by 715 kN (161 kips), primarily due to replacement of the original rail posts and curbs with reinforced concrete traffic barriers. The total weight of the exterior and interior traffic barriers [1,132 kN (255 kips)] was determined based on their estimated volume from construction drawings and using the unit weight of normal-weight concrete.

Three vehicles (the oversized combination vehicle, a Kenworth tractor-trailer in the adjacent lane, and a BMW 525i sedan) were on Span 8 at the time of the collapse (Fig. 10). The weight of the oversized vehicle was measured to be 395 kN (88.7 kips) by the NTSB (2014). The Kenworth tractor-trailer was assumed to be unloaded and estimated to weigh 178 kN (40.0 kips), and the weight of the BMW 525i sedan was estimated to be 17.8 kN (4.0 kips). When estimating vehicle locations, an assumption was made that they were traveling along the center of their corresponding lanes at the time of collapse. Video, witness, and photographic evidence were used to locate these vehicles on Span 8 at collapse, and those locations are shown in Fig. 10. The weight of the oversized combination vehicle and the Kenworth truck were each considered as concentrated loads where the truck axles were located at the time of collapse, while the modest weight of the sedan was simply considered as one concentrated load.

Critical Member Failure Mechanism

A nonlinear static analysis was conducted, using the 3D truss model in Fig. 9, to characterize the behavior of the bridge structure and the progression of damage when subjected to an increasing amplitude static horizontal force at Sway Frame 4. This approach was chosen because the actual dynamic impact force history is not known. Fig. 11 shows the characteristic nonlinear static analysis response curve for Span 8 when a horizontal force is applied at the impact location on Sway Frame 4. This curve plots the total horizontal force applied to the structure at the impact location against the corresponding horizontal displacement of the node at the impact location. Five major events that occurred during the analysis are indicated along the response curve, and normalized axial–flexural interaction histories are plotted for key locations in Fig. 12. The corresponding damage observed during a site visit is shown in Fig. 13.

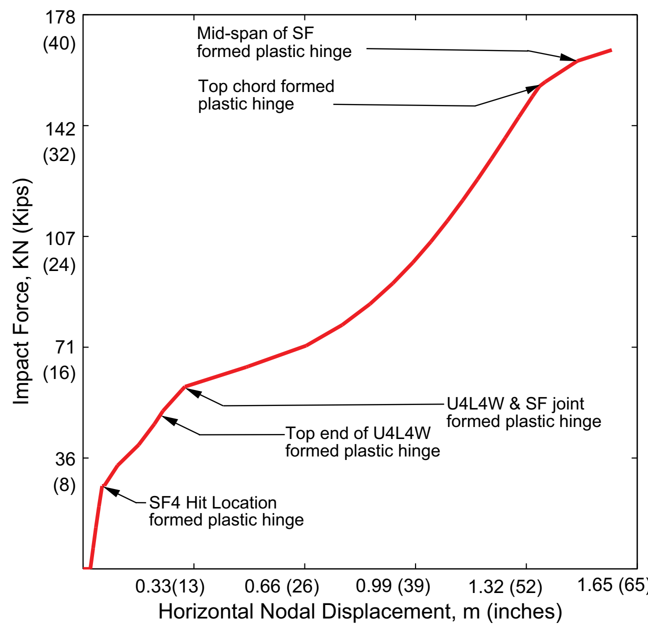

Fig. 11. Nonlinear static analysis response curve for impact location

In Fig. 11, the relative force values between the labeled events provide a means to approximate the sequence of damage. The separation in applied force between the first three major events and the last two is fairly large, indicating the first three likely occurred earlier in the actual failure sequence than the last two. The collapse sequence suggested by this analysis, which shows large sway frame deformation prior to failure of the attached upper chord, is in agreement with the NTSB (2014) conclusion about the span collapse sequence, as further discussed later. The initial behavior illustrated in Fig. 11 is nominally linear elastic prior to formation of a plastic hinge at the impact location on Sway Frame 4. Fig. 13(a) shows the impact location to illustrate the extensive local damage.

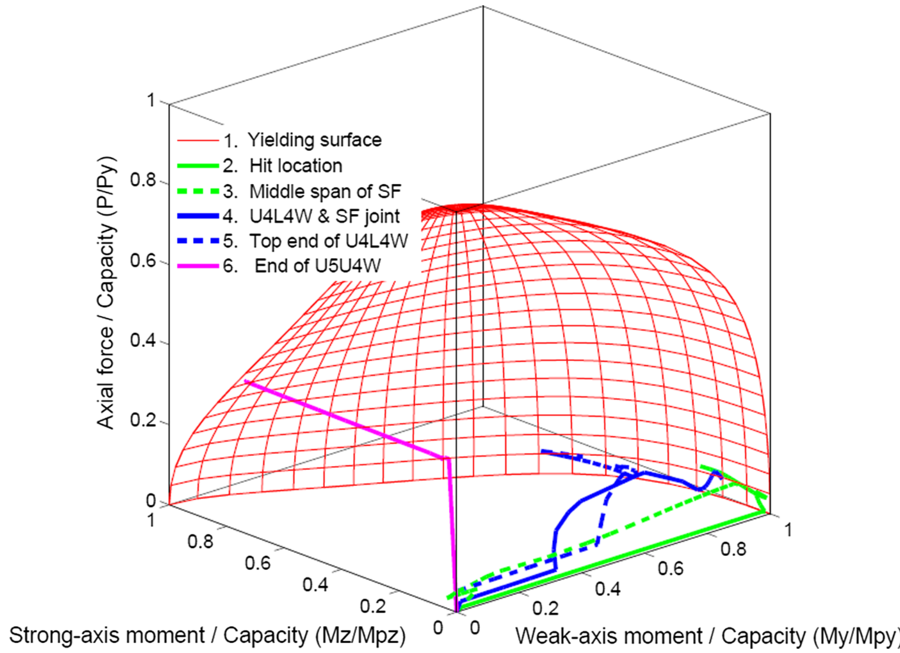

Fig. 12. Axial–flexural interaction diagrams

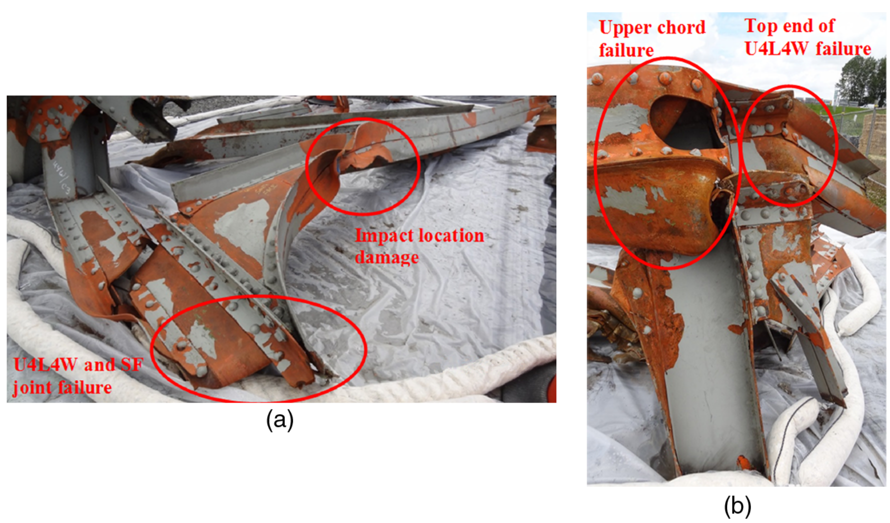

Fig. 13. Damaged components from Span 8 (images by Timothy D. Stark): (a) impact location on Sway Frame 4; (b) failure of top chord at U4 joint

In the analysis, the stiffness reduces after plastic hinge formation at the impact location and is roughly constant until two additional plastic hinges form, after which the stiffness reduces again. The final collapse occurs when a plastic hinge forms in the top chord and this critical compression member then becomes unstable. Although this static analysis provides insight into the likely sequence of damage that led to collapse, the events actually occurred close together in time as the impact was followed by collapse just seconds later.

Axial–flexural histories are shown in Fig. 12 for the five critical locations that relate to the major events shown in Fig. 11, namely, the ends of upper chord segment U5-U4W, two points on the lower chord of SF4, and the ends of the vertical member U4-L4W. The axial force in vertical U4-L4W is small from both truss action and the applied horizontal impact force. Behavior of this vertical member is initially dominated by weak-axis bending moment, indicated by M_y in Fig. 12, due to the horizontal force transferred from Sway Frame 4 (SF4). Subsequently, strong-axis bending moment, indicated by M_z in Fig. 12, starts to affect its behavior before the curve reaches the yield surface as the deformed sway frame pulls the attached vertical member inward and in the direction of the impact, inducing biaxial bending. Based on postcollapse examination, the top end of vertical U4-L4W yielded, deformed extensively, and then developed a flange fracture as shown in Fig. 13(b).

The axial–flexural interaction diagrams in Fig. 12 also indicate that the sway frame (SF) behavior is dominated by weak-axis bending (with the torsional effect of the eccentrically applied force incorporated by reducing weak axis flexural capacity). This agrees with field observations, as shown in Fig. 13(a), indicating the lower chord of SF4 deformed significantly at the impact location in the weak-axis direction, causing fracture and yielding at the bottom

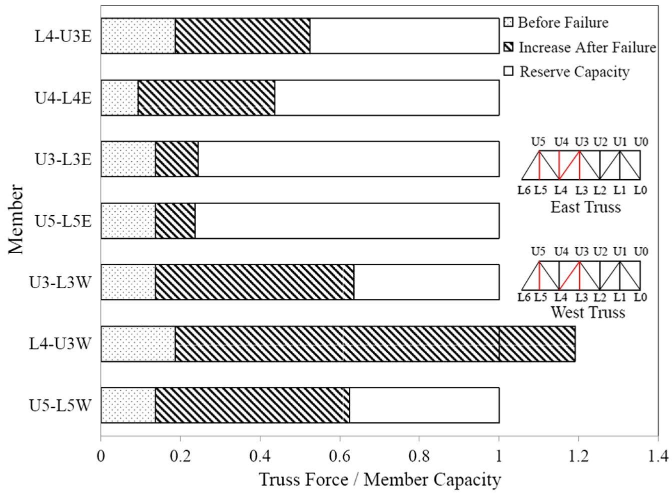

Fig. 14. Axial force demand-to-capacity ratios after failure of U3-U5W and L4-U4W

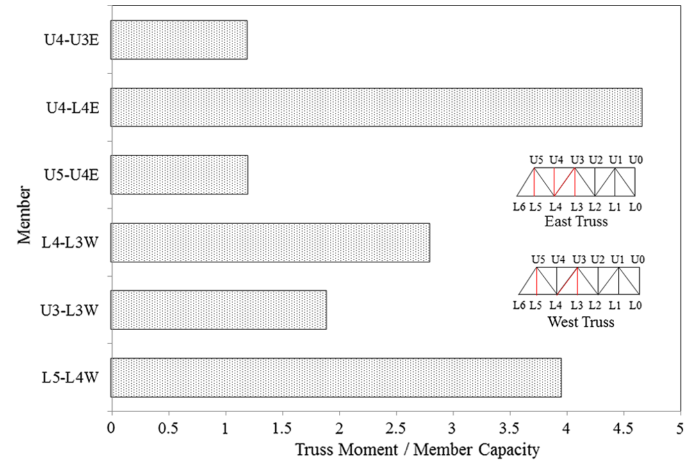

Fig. 15. Moment demand to capacity ratios after failure of U3-U5W and L4-U4W

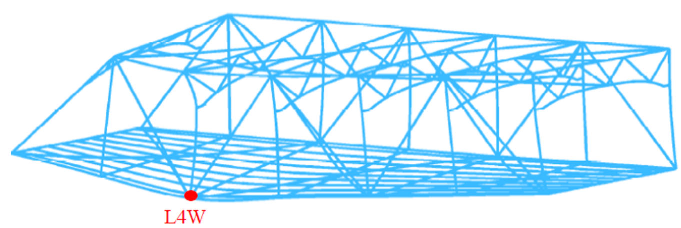

Fig. 16. Elastic deflection of Span 8 in the area of L4W after failure of U3-U5W and L4-U4W

flange. On the basis of these observations, the sway frame system was not able to carry the longitudinal (i.e., along the axis of the bridge) load applied by the truck, which caused severe damage to SF4 and transferred demand to adjacent elements (particularly the truss upper chord).

Initial behavior of top chord U5-U4W is controlled by the axial force due to truss action. After two plastic hinges developed in the attached vertical member (U4-L4W), strong-axis bending then dominates (with little decrease in axial force) before the member reaches the yield surface as shown in Fig. 12. This is consistent with the damage shown in Fig. 13(a), where the upper chord is folded about its strong axis at the U4 joint and significant local yielding and buckling occurred in both the webs and flanges of this upper chord member.

Collapse Sequence Analysis

Surveillance videos, visible damage patterns in retrieved bridge members, and the NTSB's (2014) investigative conclusions indicate a collapse sequence of Span 8 of the Skagit River Bridge of the following three main steps: (1) collision at the lower chord of Sway Frame 4 resulting in transverse deformation of the adjacent vertical truss member (L4-U4W); (2) the adjacent upper chord member (U3-U5W) being pulled downward at Joint U4 by the deformed vertical member, resulting in buckling failure of U3-U5W; and (3) failure of this critical upper chord member on the west truss initiated overloading of Span 8, which resulted in collapse. To simulate the collapse sequence of Span 8 following the failure of upper chord member U3-U5W, an analysis was conducted based on truss load redistribution and verified using available surveillance videos and photos.

An elastic analysis was conducted for the truss load redistribution study, with the strength of the upper chord (L4-L3W and L5-L4W) and attached vertical member (U4-L4E) in the west truss set to zero to approximate failure of these members. Critical truss members subject to large increases in force after failure of the upper chord (U3-U5W) and vertical (L4-U4W) members are shown in Fig. 14 to illustrate the transition in truss forces. Fig. 14 illustrates that critical members are located adjacent to the failed members in the west truss or near the U4 joint in the east truss. As U3-U5W and L4-U4W lose their load-carrying capacity, force is redistributed and increases significantly in the west truss (impact side). As a result, the axial force demands in the compression diagonal L4-U3W exceeded its nominal capacity.

Furthermore, change of moment in truss members following the failure of the upper chord is considered. Moment demands about two principal axes of all truss elements were compared with their associated nominal bending capacity. Fig. 15 plots the ratio of moment capacities and actual moments about the most critical principal axis for select members having demand in excess of capacity. The figure indicates the moment substantially increases in the bottom chords (L4-L3W and L5-L4W) and vertical (U4-L4E) members. This is a reasonable load path redistribution because once U3-U5W and L4-U4W lost their ability to carry load, demand would increase at the adjacent members and an alternative load path would be required at the other supporting end (at L4E) to carry the gravity load applied on node L4W. Fig. 16 illustrates elastic deformation of the 3D model when zero strength was assigned to the upper chord (U4-U3W and U5-U4W) and attached vertical member (U4-L4W) in the west truss. Significant downward deflections, along with severe distortion and twisting of the span, appear in the region of node L4W.

Based on this simple load-redistribution study (Figs. 14 and 15) and related elastic deflection of the span (Fig. 16), the following collapse sequence is inferred:

1. Failure of top chord U3-U5W and vertical L4-U4W due to the impact load on SF4 causes the west truss to lose its function, so gravity load near Node L4 has to be carried by the bottom chord (L4-L3W and L5-L4W) and adjacent diagonal member (L4-U3W). This results in a large moment demand on these members, causing hinging of the bottom chord (L4-L3W and L5-L4W).

2. The floor beam connecting Node L4 to the west and east trusses now functions similar to a cantilever beam, which generates large negative bending moment in the corresponding vertical member (U4-L4E) of the east truss.
3. The deformed vertical member (U4-L4E) pulls the attached upper chord (U3-U5E) downward and causes failure of the upper chord in the east truss, analogous to what had already occurred in the west truss.
4. After failure of Node 4 on both trusses, the whole structure for Span8 is effectively reduced to a scenario of a simply-supported beam with a hinge at Node 4. In this case, the span cannot carry any gravity load and it collapses, pulling the two ends of the span toward the river as well.

Video evidence agrees with this span collapse sequence suggesting that a kink developed at Node U4W after the collision and the span began to fall, with the west side tipping downward at Node L4W. Severe distortion of the west truss near Node U4W began after downward movement at Node L4W. The video ended at this point and did not capture the entire collapse process of Span8 so the east side truss collapse sequence was not observed because this truss was hard to see in the video due to the camera angle. Even though there are a number of assumptions embedded in this assessment, distortion and twisting found in the elastic-deflection analysis and results of the load-redistribution study are consistent with the surveillance video and photographic evidence, which reinforce this collapse sequence for Span 8.

Remedial Measures

Following the truss span collapse on May 23, 2013, a temporary span was installed on June 19, 2013, for Span 8 to accommodate the I-5 traffic. The elliptically curved lower chords of the sway frames in the remaining truss spans were then replaced by straight members in August 2013. This retrofit created a uniform vertical clearance of 5.5 m (18 ft) across the remaining truss spans, which allows an oversized truck to easily travel in either lane (or even onto the shoulder) throughout the entire bridge so any consideration of traveling in an incorrect location for oversized trucks is eliminated. On the night of September 14, 2013, the temporary Span 8 was removed and a permanent replacement span installed.

Fig. 17 shows the results of the nonlinear static analysis for the original truss design and two retrofit schemes. The response curve for impact location with the original truss design is similar to the response curve in Fig. 11. However, Fig. 17 shows the enhanced retrofit presented herein provides a better, i.e., less displacement, than the WSDOT retrofit. As expected, the WSDOT retrofit scheme does provide a better response curve than the original design.

Hypothetical alternative retrofits to strengthen the sway frames against impact damage were analyzed with a 3D model similar to the one described earlier for the primary study, but modified for different sway frame geometries. One option for protecting the upper chord is to stiffen and brace all sway frame vertical members in the longitudinal direction so that a horizontal impact force applied to a sway frame can be resolved into the truss without placing significant demand on the top chord. In the 3D retrofit model, longitudinal struts were placed between each vertical sway frame member in the plane of the bottom chords of the sway frames and lateral braces were placed at the lower level of the sway frames following the same pattern as the top lateral bracing system to stabilize the vertical members transversely.

Nonlinear static response analysis curves for the original model, the WSDOT retrofit model (sway frame with straight lower chord), and the proposed enhanced retrofit model show the original model and WSDOT retrofit model follow similar patterns of behavior,

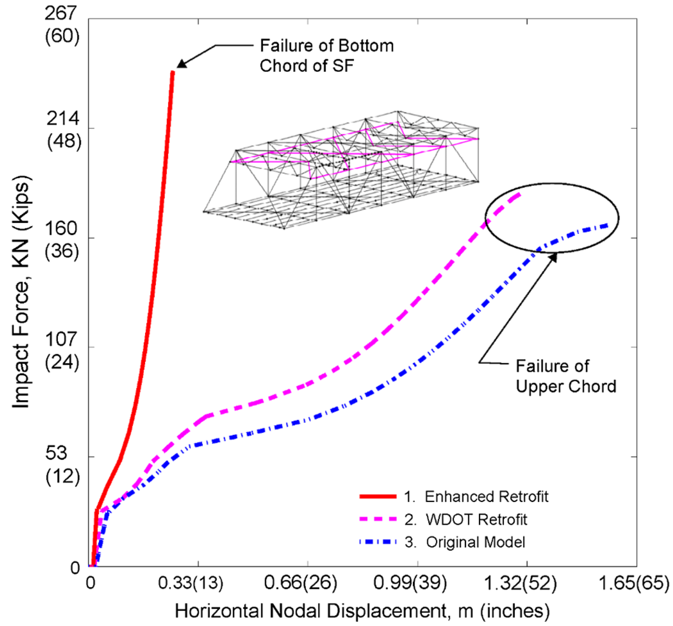

Fig. 17. Nonlinear static analysis response curve for impact location with original truss and two retrofit schemes

and the structure eventually becomes unstable due to failure of the upper chord (Fig. 17). Hence, these two structures have similar behavior with regard to a collision at the lower chord of a sway frame (although the likelihood of such a collision has been significantly reduced by the WSDOT retrofit). On the other hand, the proposed enhanced retrofit has larger horizontal stiffness than the other two models after the first plastic hinge develops at the collision point and therefore deformation of the vertical member and attached upper chord are smaller than in the other two scenarios.

Although the I-5 Skagit River Bridge has some modest redundancy due to large truss connection regions that can transfer moment, it was classified as a nonredundant fracture-critical bridge. A fracture-critical bridge is defined by AASHTO (2015) one that contains nonredundant tensile members whose failure would likely result in the collapse of the bridge. Increasing system redundancy can improve bridge performance in the event of a horizontal impact and other unpredictable accidents. Due to limitations of the through-truss structural configuration, adding primary load-carrying elements in the span to provide load-path redundancy is complicated and difficult to achieve. An alternative approach is to provide additional structural redundancy through secondary elements in the span.

Summary and Recommendations

Based on the observations, data, and analyses used to investigate the I-5 Skagit River Bridge collapse, the main findings and recommendations of this investigation are as follows:

- Bridge span collapse was initiated when an oversize container hit Sway Frame 4 in Span 8 of the west truss, causing horizontal deformation of the adjacent vertical member (L4–U4). This deformation pulled the attached upper chord member (U3–U5) downward, causing instability in the upper chord.
- A comprehensive database of actual bridge clearances using minimum clearance should be developed for permitting purposes. The database should include variable heights across

the bridge, prior bridge strike data from inspection reports, and changes in vertical clearance with time.

- Vertical clearance signage should convey variable clearances across and along the roadway. Variable clearance signage should appear well before the structure to allow lane changes.
- A vertical clearance buffer of greater than 76 mm (3 in.) should be used because changes in pavement thickness, weather conditions, and vehicle bounce will occur over time.
- The pilot vehicle driver should not use any electronic communication device that is not related to communication with the oversize vehicle driver.
- Pilot car height poles should be mounted vertically and be equipped with technology that immediately informs the escort and oversize vehicle drivers that a structural element has been contacted by the height pole, so the escort driver does not have to relay the information.

Acknowledgments

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