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Synthesis: Assess Hazard and Vulnerability Prioritization for Texas Bridges and Tunnels

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University of Houston

Lu Gao

Yi-Lung Mo

Center for Transportation Research,

University of Texas at Austin

Zhe Han

UNIVERSITY of HOUSTON

University of Houston

Center for Transportation Research, The University of Texas at Austin

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16. Abstract This report documents the final results of TxDOT Research Project 0-7239, which developed a statewide procedure for screening and prioritizing bridge and tunnel hazards and vulnerabilities. The work synthesized research, agency guidance, state DOT practices, and incident experience; identified applicable data sources; developed ten adopted risk modules; applied the methods to the Texas bridge inventory; and translated the results into response and implementation guidance. The adopted modules address explosive/security criticality, fatigue, flood overtopping and scour, hurricane surge, heavy-vehicle pier collision, overload, overheight collision, tornado, vessel collision, and wildfire. Each module defines the data inputs, calculation steps, assumptions, and bridge ranking procedure. Statewide results include risk-specific rankings, maps, and Top 10 bridge lists, together with a multi-hazard recurrence ranking. TxDOT can apply these results through one six-step process: select candidate bridges; check bridge identity and source information; review current condition and identify the ranking factor; select the response through the applicable hazard-specific decision tree; update the relevant data and records; and rerun and review the statewide rankings. Recommended implementation language identifies changes to TxDOT bridge design, inspection, hydraulic, project development, and maintenance procedures. The resulting framework directs bridge-level review, inspection, analysis, mitigation planning, and data improvement toward the conditions identified by the rankings.		
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Synthesis: Assess Hazard and Vulnerability Prioritization for Texas Bridges and Tunnels

by

Lu Gao, Ph.D., and Yi-Lung Mo, Ph.D., P.E.
University of Houston

Zhe Han, P.E.
Center for Transportation Research
The University of Texas at Austin

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Disclaimer

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration (FHWA) or the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation.

Engineer in responsible charge: Yi-Lung Mo, Ph.D., P.E., Texas P.E. No. 110677.

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SECTION 1. Introduction

1.1 Project Background

Texas bridges and tunnels support statewide mobility, freight movement, emergency access, evacuation, and economic activity. These assets face multiple hazards, including natural hazards, operational impacts, structural deterioration, and human-caused threats. A single bridge may have low exposure to one hazard but high exposure to another. A bridge may also have moderate physical vulnerability but high consequence because of traffic volume, detour length, freight importance, emergency access, or corridor criticality. For this reason, hazard and vulnerability prioritization must consider exposure, vulnerability, and consequence together.

TxDOT already manages many components of bridge risk through established procedures. Bridge inspection identifies condition and critical findings. Hydraulic procedures address waterway, scour, and coastal-storm design concerns. Load rating and load posting procedures manage capacity and permit vehicle restrictions. Damage-inspection and emergency-response procedures support decisions after impacts, floods, fires, and other events. Asset-management systems and bridge files preserve records. The research opportunity is to connect these pieces into a repeatable statewide screening framework and then translate that framework into practical implementation language.

1.2 Research Objectives

The project objectives were to:

1. Synthesize literature, agency practice, standards, technical guidance, and incident cases related to bridge and tunnel hazard risks.
2. Develop mitigation recommendations that TxDOT can use in design, inspection, maintenance, operations, protection, and management workflows.
3. Develop a bridge/tunnel risk assessment and vulnerability prioritization workflow that uses available inventory, hazard, event, and consequence data.
4. Apply the workflow to Texas bridge inventory data to support high-risk bridge identification and GIS mapping.
5. Translate the technical results into recommended TxDOT manual, recordkeeping, and implementation language.

1.3 Report Integration Approach

The report is organized as eight numbered sections followed by two appendices. The numbered sections present the project purpose, risk-assessment methods, statewide results, use of the rankings, hazard-specific responses, the GIS tool, proposed manual language, and conclusions. Appendix A contains the detailed literature and practice synthesis, and Appendix B presents the Value of Research assessment.

1.4 Report Organization

Section 1 introduces the project. Section 2 presents the final risk assessment methodology, including the data used by each risk module. Section 3 provides the risk-specific statewide results and concludes with the overall multi-hazard results. Section 4 explains how TxDOT should use the rankings. Section 5 provides hazard-specific review and response recommendations. Section 6 describes the Bridge Risk Screening GIS tool. Section 7 presents recommended TxDOT manual and implementation language. Section 8 states the conclusions. Appendix A contains the detailed literature and practice synthesis, and Appendix B presents the Value of Research assessment.

SECTION 2. Risk Assessment Methodology

The adopted statewide screening procedure is organized as a sequence of steps so that each bridge can be followed from source data to final rank. Within every risk subsection, the steps define eligibility, assign the required data, calculate the risk measure, and establish the statewide order.

2.1 Explosive/Security Criticality

1. **Select the bridges for analysis.** Begin with the Texas bridge inventory. Retain records with a valid structure identifier and location and with enough information to calculate access opportunity, structural vulnerability, and bridge importance.
2. **Prepare the indicator values.** Convert each available indicator to a 0–1 value, where a larger value represents greater access opportunity, vulnerability, or importance. Table 2.1 identifies the source and weight for every indicator used in the statewide calculation.

Table 2.1: Security-criticality indicators, data sources, and adopted weights

Component	Indicator	Data source and calculation	Weight
Occurrence/opportunity	Parking or staging access	NBI service-under and lanes-under information; 1 when a roadway with traffic lanes passes beneath the bridge, otherwise 0	0.1720
Occurrence/opportunity	Sidewalk access	NBI left and right curb or sidewalk widths; 1 when either width is positive, otherwise 0	0.0605
Structural vulnerability	Columns per support	Texas element quantities for column elements divided by the estimated number of supports from the span count, then converted to the adopted 0–1 class	0.1784
Structural vulnerability	Sufficiency	Matched 2018 NBI sufficiency rating converted so that lower sufficiency produces a larger 0–1 value	0.0655
Structural vulnerability	Pier protection	NBI Item 111 converted to the adopted 0–1 protection-deficiency value	0.0906
Bridge importance	Maximum span	NBI Item 48 converted to its statewide percentile	0.1250
Bridge importance	AADT	NBI Item 29 converted to its statewide percentile	0.1459
Bridge importance	Evacuation function	Project GIS intersection with designated evacuation routes; 1 for a designated route, otherwise 0	0.1225

Note: The weights are the original analytic-hierarchy-process weights for the corresponding NCHRP 20-07/Task 378 Simple Bridge Security Checklist items [2]. The statewide calculation retains the checklist items supported by Texas statewide data and normalizes their weights within each component.

3. **Calculate the three components.** Use the same available-data equation for occurrence/opportunity,

structural vulnerability, and bridge importance:

$$C = \frac{\sum_{j \in \mathcal{A}_C} w_j x_j}{\sum_{j \in \mathcal{A}_C} w_j}, \qquad C \in \{O, V, I\} \tag{2.1}$$

where C is the component being calculated; O is occurrence/opportunity; V is structural vulnerability; I is bridge importance; j identifies an indicator listed in Table 2.1; $\mathcal{A}_C$ is the set of indicators with data for that bridge and component; x_j is the 0–1 indicator value; and w_j is the adopted weight. A missing indicator and its weight are omitted from both sums because the denominator must match the information used in the numerator.

4. **Apply the historic-landmark rule.** After calculating I, set $I = 1$ when the NBI historic-significance record identifies the bridge as a historic landmark. This rule gives the maximum importance value to a bridge with documented historic significance.

5. **Calculate the final security-criticality score.** Multiply the three components using

$$R_{\text{sec}} = 100OVI \tag{2.2}$$

where R_{sec} is the security-criticality score on a 0–100 scale and O, V, and I are the components calculated with Equation 2.1. The multiplicative form gives the largest scores to bridges that are accessible, structurally vulnerable, and important at the same time.

6. **Rank the bridges.** Sort eligible bridges from the largest to the smallest value of R_{sec}. Use the structure identifier to order equal scores.

2.2 Fatigue

1. **Select the bridges for analysis.** Begin with the 2025 Texas bridge inventory and retain steel bridge kinds 3 and 4. Require a valid year built, positive truck traffic, positive maximum span, and an assigned Texas truck-weight spectrum. This produces 1,742 candidate bridge records before the element-data requirement is applied.

2. **Calculate bridge age.** Subtract the NBI year built from the 2025 analysis year. The result is Age, the number of years used in the cumulative exposure calculation.

3. **Calculate daily truck demand.** Obtain AADT and truck percentage from the assigned TxDOT roadway record or NBI record. Multiply AADT by the truck percentage to calculate the average number of trucks crossing the bridge each day. Denote this bridge-level daily truck traffic by T_b. The final fatigue calculation uses this bridge-level truck volume directly.

4. **Develop the Texas vehicle-load distributions.** Use STARS II weigh-in-motion stations together with the other Texas weigh-in-motion and vehicle-classification stations included in the final statewide assignment. For FHWA vehicle Classes 5–13, determine class shares and gross-vehicle-weight groups. Within each class and weight group, retain 24 probability-weighted representative axle configurations. Each representative configuration contains axle weights, axle spacings, and a probability weight.

5. **Assign a vehicle-load distribution to each bridge.** First match a bridge to monitoring data on the same roadway and travel direction. When an exact roadway match is not available, use functional class, truck-volume pattern, and network distance to select the station that best represents the truck stream carried

by the bridge. The assigned station provides the truck-class proportions and axle configurations used for that bridge.

6. **Calculate the load effect for each representative truck.** Use the governing bridge span from NBI Item 48, rounded to the span used by the truck-load library. Move each representative axle configuration across a simple-span midspan-moment influence line and retain the largest moment, M_j. Repeat the calculation with the 8-, 32-, and 32-kip HS20 reference axles. Check the permitted HS20 rear-axle spacings and retain the largest reference moment, M_{HS20}.

7. **Calculate cumulative cyclic demand.** Combine bridge age, critical-lane truck demand, vehicle-class proportions, and relative moment effects using

$$E_{\text{raw}} = \text{Age} \times 365 \times T_b \sum_j p_j \left(\frac{M_j}{M_{\text{HS20}}} \right)^3 \tag{2.3}$$

where E_{raw} is cumulative fatigue exposure; Age is bridge age in years from Step 2; 365 converts daily truck passages to annual passages; T_b is bridge-level daily truck traffic from Step 3; j identifies a representative axle configuration in the assigned Texas vehicle-load distribution; p_j is the combined probability of that vehicle class and representative configuration; M_j is the maximum simple-span moment produced by configuration j; and M_{HS20} is the HS20 reference moment on the same span. The exponent of 3 is the adopted fatigue moment exponent.

8. **Calculate the superstructure-condition component.** Obtain the superstructure condition rating from NBI Item 59. Convert the rating to a 0–1 component using

$$C = \begin{cases} 0, & \text{Item 59} = 7, 8, 9 \\ 0.25, & \text{Item 59} = 6 \\ 0.50, & \text{Item 59} = 5 \\ 0.75, & \text{Item 59} = 4 \\ 1, & \text{Item 59} \leq 3 \end{cases} \tag{2.4}$$

where C is the superstructure-condition component and Item 59 is the NBI superstructure condition rating. Ratings of 7–9 produce $C = 0$, while ratings of 3 or lower produce $C = 1$. Therefore, a larger value of C represents poorer observed condition.

9. **Calculate distress for each fatigue-related steel element.** Obtain element quantities and condition states from the 2025 Texas element-level inspection dataset. Use element numbers 28, 29, 30, 102, 107, 113, 120, 141, 147, 148, 152, 161, and 162. For each eligible steel element, calculate

$$D_e = \min \left(1, \frac{Q_{e,3} + 2Q_{e,4}}{Q_{e,\text{total}}} \right) \tag{2.5}$$

where D_e is the distress ratio for element e; $Q_{e,3}$ is the inspected quantity in condition state 3; $Q_{e,4}$ is the inspected quantity in condition state 4; and $Q_{e,\text{total}}$ is the total inspected quantity of that element. Condition-state-4 quantities receive twice the influence of condition-state-3 quantities. Limit the result to 1.0.

10. **Calculate the bridge-level element component.** Compare the distress ratios for all fatigue-related steel elements on the bridge and retain the largest value:

$$D = \max_{e} D_e \tag{2.6}$$

where D is the bridge-level element-distress component and D_e is the distress ratio from Equation 2.5. Using the maximum value makes the most distressed fatigue-related element control this component.

11. **Require fatigue-related element data.** Remove a candidate bridge when none of the 13 fatigue-related element numbers has a valid inspected quantity. Of the 1,742 candidate bridge records, 671 are removed by this requirement and 1,071 remain.

12. **Combine parallel records at one physical bridge site.** Group bridge records when their centroids are no more than 75 m apart, they carry the same route, and at least 55 percent of the normalized crossing-feature words overlap. Apply the two documented manual grouping exceptions. For a grouped site, retain the record with the largest preliminary fatigue score as the primary record, use the member with the largest cumulative exposure for traffic inputs, use the poorest superstructure condition, and use the largest element-distress component. This produces 778 physical bridge sites; 290 sites contain grouped parallel records representing 583 inventory records.

13. **Convert cumulative exposure to a statewide percentile.** Order the 778 physical bridge sites by E_{raw}. Convert each value to its statewide percentile, E, on a 0–1 scale using average ranks for equal values. A site with greater cumulative truck-load exposure receives a larger value of E.

14. **Calculate the final fatigue priority score.** Combine exposure, observed superstructure condition, and element distress using

$$S_{\text{fat}} = 100(0.65E + 0.20C + 0.15D) \tag{2.7}$$

where S_{fat} is the fatigue priority score on a 0–100 scale; E is the cumulative-exposure percentile from Step 13; C is the superstructure-condition component from Equation 2.4; and D is the bridge-level element-distress component from Equation 2.6. The adopted weights assign 65 percent of the score to cumulative exposure, 20 percent to superstructure condition, and 15 percent to element distress.

15. **Rank the bridge sites.** Sort the 778 eligible steel bridge sites from the largest to the smallest value of S_{fat}. Use the structure identifier to order equal scores.

2.3 Flood Overtopping and Scour

1. **Select waterway bridges.** Begin with the Texas bridge inventory. Retain NBI Item 42B codes 5, 6, 7, 8, and 9. Also retain an SNBI record when its feature type identifies a bridge and its feature description begins with waterway or flood-relief coding. Remove grade separations and other records that do not represent a waterway crossing.

2. **Match each bridge to the FAST flood profile.** Use the bridge coordinates to locate the corresponding FAST flood profile. Check the coordinate distance and the shape of the profile to confirm that the selected profile crosses the bridge. Retain the local ground elevation, bridge low-chord elevation, and mapped 10-, 100-, and 500-year flood depths.

3. **Calculate low-chord clearance.** Subtract the FAST ground elevation from the FAST bridge low-chord elevation. The result is the height of the bridge low chord above the local ground reference. Use the same ground reference for the mapped flood depth and the bridge clearance.

4. **Calculate the 500-year overtopping difference.** Compare the 500-year mapped flood depth with the low-chord clearance using

$$\Delta h_{500} = h_{500} - c_{\text{low chord}} \tag{2.8}$$

where Δh_{500} is the vertical overtopping difference in feet; h_{500} is the FAST 500-year flood depth above local ground; and $c_{\text{low chord}}$ is the bridge low-chord clearance above the same ground reference from Step 3. A positive value means the mapped flood surface is above the low chord. A negative value means the low chord remains above the mapped flood surface.

5. **Match each bridge to a National Water Model stream reach.** Use bridge location, drainage area, and stream order to select the stream reach that represents flow at the bridge. Retain the 1979–2023 retrospective velocity time series for the matched reach. A bridge enters the combined ranking only when it has both a valid FAST clearance and a valid National Water Model velocity. The common set contains 8,909 bridges.

6. **Calculate the maximum modeled flow velocity.** Search the full retrospective time series and retain the largest velocity:

$$v_{\text{max}} = \max_{t \in [1979,2023]} v(t) \tag{2.9}$$

where v_{max} is the maximum modeled flow velocity at the bridge in meters per second; t is a time step in the 1979–2023 National Water Model retrospective period; and $v(t)$ is the reach-average velocity at time t for the stream reach assigned in Step 5. A larger value represents stronger modeled flow exposure.

7. **Calculate the statewide overtopping rank.** Sort bridges with a valid Δh_{500} value from largest to smallest. Assign rank 1 to the bridge with the largest positive overtopping difference. This produces $R_{\text{overtopping}}$.

8. **Calculate the statewide scour-exposure rank.** Sort the 8,909 common-set bridges by v_{max} from largest to smallest. When two bridges have exactly the same modeled velocity, order NBI Item 113 values as 0, 1, 2, 3, 4, U, 6, 5, 7, 8, T, 9, and N. Use the structure identifier when both velocity and Item 113 are equal. This produces R_{scour}.

9. **Calculate the combined flood rank measure.** For each bridge with both component ranks, calculate

$$\bar{R}_{\text{flood}} = \frac{R_{\text{overtopping}} + R_{\text{scour}}}{2} \tag{2.10}$$

where $\bar{R}_{\text{flood}}$ is the combined flood rank measure; $R_{\text{overtopping}}$ is the statewide overtopping rank from Step 7; and R_{scour} is the statewide velocity-based scour rank from Step 8. The arithmetic mean gives equal influence to overtopping and velocity-based scour exposure.

10. **Rank the bridges.** Sort bridges from the smallest to the largest value of $\bar{R}_{\text{flood}}$. When average ranks are equal, use the overtopping rank, then the scour rank, and then the structure identifier. A smaller average rank receives a higher statewide flood priority. Report the FAST flood values, National Water Model

velocity, and NBI Item 113 rating with the rank. Report Hazus loss and failure values as supporting flood information.

2.4 Hurricane Surge Exposure

1. **Select bridges in the coastal analysis area.** Begin with the Texas bridge inventory and retain bridges located within the area covered by the hurricane probability and storm-surge datasets.
2. **Assign hurricane-category probabilities.** For each bridge location, obtain the annual exceedance probability for Category 1 through Category 5 hurricanes from the project hurricane probability dataset. An exceedance probability for category k represents the annual probability of a hurricane at category k or higher.
3. **Assign storm-surge depths.** Overlay each bridge location on the NOAA/NHC SLOSH v4 surge surfaces. Record the mapped inundation class for Categories 1, 2, 3, 4, and 5. Convert SLOSH classes 1, 2, 3, and 4 to lower-bound depths of 0, 3, 6, and 9 ft, respectively. Use that lower-bound depth as H_k for each hurricane category.
4. **Determine effective low-chord clearance.** Calculate the height of the bridge low chord above its available vertical reference. First use the FAST center-profile low-chord-above-ground clearance. If FAST is unavailable at a SLOSH-positive bridge, use NBI Item 39 navigation clearance; use NBI Item 116 for a vertical-lift bridge. If neither value is available, use the corrected 3DEP LiDAR estimate or the final imagery-reviewed estimate. For an open bridge, the LiDAR estimate subtracts 4 ft from deck-above-ground height to represent superstructure depth. Record the selected value and source.
5. **Convert each category-exceedance probability to an annual rate.** Calculate

$$\lambda_{\geq k} = -\ln\left[1 - P(C \geq k)\right], \qquad k = 1, \ldots, 5 \tag{2.11}$$

where $\lambda_{\geq k}$ is the annual Poisson rate for hurricanes at category k or higher; C is hurricane category; k identifies a threshold from Category 1 through Category 5; and $P(C \geq k)$ is the annual exceedance probability assigned in Step 2.

6. **Calculate the annual rate for each exact category.** Subtract the rate for the next-higher category from the exceedance rate:

$$\lambda_k = \lambda_{\geq k} - \lambda_{\geq k+1}, \qquad k = 1, 2, 3, 4 \tag{2.12}$$

where λ_k is the annual rate for exact category k; $\lambda_{\geq k}$ is the exceedance rate for category k or higher; and $\lambda_{\geq k+1}$ is the exceedance rate for the next-higher category. For Category 5, use $\lambda_5 = \lambda_{\geq 5}$.

7. **Convert each exact-category rate to an annual probability.** Calculate

$$p_k = 1 - \exp(-\lambda_k) \tag{2.13}$$

where p_k is the annual probability of exact category k and λ_k is the exact-category rate from Equation 2.12.

8. **Determine whether each category reaches the bridge low chord.** Compare the SLOSH surge depth with the effective low-chord clearance:

$$I_k = \begin{cases} 1, & H_k \geq H_{\text{LC}} \\ 0, & H_k < H_{\text{LC}} \end{cases} \tag{2.14}$$

where I_k is the surge-exposure indicator for category k; H_k is the SLOSH surge depth assigned in Step 3; and H_{LC} is the effective bridge low-chord clearance from Step 4. A value of $I_k = 1$ means that the mapped surge reaches or exceeds the low chord.

9. **Combine the rates for categories that reach the bridge.** Calculate

$$\lambda_{\text{surge}} = \sum_{k=1}^{5} \lambda_k I_k \tag{2.15}$$

where λ_{surge} is the combined annual rate of hurricane categories whose surge reaches the bridge low chord; λ_k is the exact-category rate; and I_k is the exposure indicator. A category with $I_k = 0$ adds nothing to the combined rate.

10. **Calculate the annual surge-exposure probability.** Convert the combined rate to probability using

$$P_{\text{surge}} = 1 - \exp(-\lambda_{\text{surge}}) \tag{2.16}$$

where P_{surge} is the annual probability that hurricane surge reaches the effective bridge low chord and λ_{surge} is the combined exposure rate from Equation 2.15.

11. **Calculate the probability-weighted exposure margin.** For every category that reaches the bridge, multiply the vertical amount above the low chord by the exact-category probability:

$$M_{\text{surge}} = \sum_{k=1}^{5} p_k I_k (H_k - H_{\text{LC}}) \tag{2.17}$$

where M_{surge} is the probability-weighted vertical exposure margin in feet; p_k is the annual probability of exact category k; I_k identifies whether that category reaches the bridge; H_k is the mapped surge depth; and H_{LC} is the effective low-chord clearance. Report this value to describe the expected vertical amount of surge exposure.

12. **Rank the bridges.** Retain bridges with $P_{\text{surge}} > 0$. Round P_{surge} to six decimal places for ranking so that sub-millionth numerical differences are treated as equal. Sort from the largest to the smallest rounded probability. For equal rounded probabilities, rank the bridge with the larger NBI AADT first. Resolve any remaining ties with the structure identifier.

2.5 Heavy-Vehicle Bridge Pier Collision

1. **Extract the heavy-vehicle crash records.** Begin with 2016–2025 Texas CRIS records. Retain crashes involving a truck or another heavy vehicle and carrying information needed to determine the object struck, crash location, roadway relationship, and vehicle type.

2. **Identify physical impacts with bridge supports.** Review the object-struck description and roadway relationship for each candidate crash. Use the crash coordinates and bridge geometry to determine whether the vehicle struck an underpass pier or another bridge support. Remove crashes that occurred near a bridge but did not involve a physical support impact.

3. **Assign each accepted crash to a bridge site.** Match the accepted crash coordinates to the physical bridge-support location. Use the facility carried, roadway below, direction of travel, and available site review to select the bridge site. Assign one bridge-site identifier to each accepted crash so that a single crash is counted once.

4. **Combine records at the same physical site.** Treat parallel bridge records and related inventory records as one site when they share the same underpass support location. Retain the associated structure identifiers so that the site result can be connected to the bridge inventory.

5. **Count accepted crashes at each bridge site.** The final published dataset contains 3,778 accepted bridge-support crashes assigned to 2,159 physical bridge sites. Count the crashes assigned to each site using

$$N_b = \sum_{i=1}^{3778} \mathbf{1}(a_i = b) \tag{2.18}$$

where N_b is the observed 2016–2025 crash count at bridge site b; i identifies one of the 3,778 accepted and assigned crashes; a_i is the bridge site assigned to crash i; and $\mathbf{1}(a_i = b)$ equals 1 when crash i belongs to site b and 0 otherwise.

6. **Calculate the observed annual crash frequency.** Divide the ten-year count by the length of the observation period:

$$\lambda_b = \frac{N_b}{10} \tag{2.19}$$

where λ_b is the observed number of bridge-support crashes per year at site b; N_b is the accepted crash count from Equation 2.18; and 10 is the number of years from 2016 through 2025.

7. **Assign bridge traffic for tie-breaking.** Match each physical bridge site to its TxDOT or NBI traffic record and retain AADT. When a site contains parallel bridge records, use the traffic value associated with the roadway carried at that site.

8. **Rank the bridge sites.** Sort sites from the largest to the smallest value of N_b. For sites with the same accepted crash count, rank the site with the larger AADT first. Resolve any remaining ties with the bridge-site or structure-identifier order.

2.6 Overload

1. **Select the bridges for analysis.** Begin with the 21,349 TxDOT-managed, nonculvert bridge records in the final statewide inventory. Require a positive NBI Item 64 operating rating, a positive NBI Item 48 maximum span, bridge-level truck traffic, and an assigned Texas axle-load distribution.

2. **Calculate daily truck traffic.** Obtain AADT and truck percentage from the matched TxDOT traffic record or NBI record. Multiply AADT by the truck percentage to estimate the average number of trucks crossing the bridge each day. This result becomes TruckAADT.

3. **Prepare the Texas truck-load observations.** Use STARS II weigh-in-motion stations and the other Texas weigh-in-motion and vehicle-classification stations in the final statewide assignment. Retain FHWA vehicle Classes 5–13. Divide each vehicle class into gross-vehicle-weight groups and retain 24 probability-weighted representative axle configurations for each class and weight group.

4. **Assign monitoring data to each bridge.** First use observations from the same route and travel direction. When an exact match is not available, use functional class, truck-volume pattern, and network distance to select a representative station. The assigned observations provide the bridge-specific vehicle-class mix and axle-load distribution.

5. **Calculate the HS20 reference load effect.** Round the NBI Item 48 maximum span to the nearest 0.5 m. Represent the bridge by a simple span of that length. Move the 8-, 32-, and 32-kip HS20 reference axles across the midspan-moment influence line. Evaluate rear-axle spacings from 14 to 30 ft and retain the largest moment as E_{HS20}.

6. **Calculate the bridge-specific load-effect threshold.** Scale the HS20 moment by the bridge operating rating:

$$E_{\text{threshold}} = E_{\text{HS20}} \frac{R_{\text{op}}}{32.4} \tag{2.20}$$

   where $E_{\text{threshold}}$ is the bridge-specific threshold moment; E_{HS20} is the HS20 reference moment from Step 5; R_{op} is the NBI Item 64 operating rating converted to metric tons; and 32.4 metric tons is the adopted HS20 reference rating. A larger operating rating produces a larger threshold.

7. **Calculate the load effect of each representative truck.** For representative truck j, move its axle weights and spacings across the same simple-span influence line and retain the maximum midspan moment E_j. Express that moment as an Item-64-equivalent rating:

$$R_{\text{eq},j} = 32.4 \frac{E_j}{E_{\text{HS20}}} \tag{2.21}$$

   where $R_{\text{eq},j}$ is the equivalent rating in metric tons for representative truck j; 32.4 metric tons is the HS20 reference rating; E_j is the truck's maximum simple-span midspan moment; and E_{HS20} is the HS20 reference moment from Step 5. Truck j exceeds the bridge threshold when $R_{\text{eq},j} > R_{\text{op}}$.

8. **Calculate the exceedance probability within each vehicle class.** Add the probability weights of the representative trucks for which $R_{\text{eq},j} > R_{\text{op}}$. Repeat this calculation for each of FHWA vehicle Classes 5–13.

9. **Combine the vehicle classes.** Multiply each class-specific exceedance probability by that class's share of the bridge's assigned truck stream. Equivalently, sum over all representative trucks:

$$p_{\text{exceed}} = \sum_j p_j \mathbf{1}(R_{\text{eq},j} > R_{\text{op}}) \tag{2.22}$$

   where p_{exceed} is the probability that a truck in the assigned Texas vehicle distribution exceeds the bridge's Item 64-equivalent moment threshold; j identifies a representative truck; p_j is its combined vehicle-class and within-class probability; $R_{\text{eq},j}$ is the equivalent rating from Equation 2.21; R_{op} is the bridge operating rating; and $\mathbf{1}(\cdot)$ equals 1 when the stated condition is true.

10. **Calculate annual overload exposure.** Multiply daily truck demand by the exceedance probability and the number of days in a year:

$$N_{\text{overload,annual}} = 365 \, \text{TruckAADT} \, p_{\text{exceed}} \tag{2.23}$$

where $N_{\text{overload,annual}}$ is the expected number of truck passages per year that exceed the bridge-specific load-effect threshold; TruckAADT is the average daily truck traffic from Step 2; p_{exceed} is the exceedance probability from Equation 2.22; and 365 converts daily passages to annual passages.

11. **Rank the bridges.** Sort eligible bridges from the largest to the smallest value of $N_{\text{overload,annual}}$. A bridge with a larger expected number of annual threshold-exceeding passages receives a higher statewide overload priority.

2.7 Overheight Collision

1. **Select grade-separation bridges.** Begin with the Texas bridge inventory. Retain TxDOT-maintained, nonculvert bridges that pass over a roadway. Require a valid bridge location, positive structure length, and a reported vertical underclearance of at least 2.5 m. A total of 9,223 bridges met the initial candidate definition.

2. **Identify the road below each bridge.** Match each bridge to the TxDOT roadway segment passing beneath it. Use the bridge coordinates, roadway geometry, facility names, and crossing direction to identify the under-road segment. Retain matches classified as high or medium confidence. The matched roadway provides under-road AADT, truck percentage, number of lanes, and rural or urban functional class.

3. **Count observed overheight strikes.** Review 2016–2025 CRIS records coded as a vehicle hitting the top of an underpass or tunnel. Match each accepted crash to one bridge by coordinates, roadway names, travel direction, and bridge geometry. The final model dataset contains 425 accepted crashes assigned to 252 bridges. For every candidate bridge, count the accepted crashes during the ten-year period. This count is $Y_b = N_{\text{obs},b}$.

4. **Prepare the continuous predictors.** For bridge b, calculate seven continuous predictors: the natural logarithm of under-road AADT; vertical underclearance in meters; bridge age in 2025; the natural logarithm of one plus maximum span in meters; the natural logarithm of one plus horizontal clearance in meters; under-road truck percentage; and number of under-road lanes. Standardize each predictor using

$$z_{bj} = \frac{g_j(x_{bj}) - \bar{x}_j}{s_j} \tag{2.24}$$

where z_{bj} is the standardized value of predictor j for bridge b; x_{bj} is the original predictor value; $g_j(\cdot)$ is the transformation shown in Table 2.2; $\bar{x}_j$ is the model-sample mean after transformation; and s_j is the corresponding model-sample standard deviation.

5. **Prepare the functional-class predictors.** Use the rural or urban functional class of the road below the bridge as a categorical predictor. The fitted sample contains rural groups R1–R7 and urban groups U1–U7. Use U5 as the reference category. Create one indicator for every other category; the indicator equals 1 when the under-road belongs to that category and 0 otherwise.

6. **Form the complete estimation sample.** Require valid values for all seven continuous predictors, functional class, and structure length. The complete estimation sample contains 6,647 bridges. Among these bridges, 252 have at least one accepted crash and the total observed crash count is 425.

7. **Fit the Texas negative-binomial model.** Model the ten-year crash count with an NB2 distribution:

$$Y_b \sim \text{NB2}(\mu_{10,b}, \alpha), \qquad \text{Var}(Y_b) = \mu_{10,b} + \alpha\mu_{10,b}^2 \tag{2.25}$$

where Y_b is the observed 2016–2025 crash count for bridge b; $\mu_{10,b}$ is its expected ten-year crash count; and α is the NB2 dispersion parameter. Estimate the model with iteratively reweighted least squares and apply a ridge penalty of 0.10 to all nonintercept coefficients.

8. **Calculate the fitted ten-year mean.** Use bridge length as the exposure offset and calculate

$$\mu_{10,b} = 10L_b \exp\left(\beta_0 + \sum_{j=1}^{7} \beta_j z_{bj} + \sum_{c \neq \text{U5}} \gamma_c I_{bc}\right) \tag{2.26}$$

where L_b is bridge structure length in meters; 10 is the number of years in the model period; β_0 is the fitted intercept; z_{bj} is standardized continuous predictor j from Equation 2.24; β_j is its fitted coefficient; I_{bc} is the indicator for functional class c; and γ_c is the coefficient for class c relative to U5.

9. **Apply the final intercept calibration.** Multiply the initial fitted means by 0.8579 by adding $\ln(0.8579)$ to the fitted intercept. This makes the sum of the fitted ten-year crash counts equal the 425 accepted crashes in the estimation sample. The final intercept is -10.2716.

10. **Use the fitted model parameters.** The estimation converged after 16 outer iterations. The fitted dispersion parameter is $\alpha = 5.1713$. Table 2.2 gives the continuous-predictor estimates and the values used for standardization. Table 2.3 gives the functional-class estimates. The rate ratio is exp(coefficient). For a continuous predictor, it represents the multiplicative change in predicted strike frequency for a one-standard-deviation increase while other predictors remain fixed.

Table 2.2: Estimated continuous-predictor parameters for the Texas overheight-collision model

Predictor	Value entered before standardization	Mean	Standard deviation	Coefficient (rate ratio)
Under-road AADT	ln(AADT)	9.1532	1.8701	0.3913 (1.479)
Vertical underclearance	Meters	5.2578	0.8074	−1.4530 (0.234)
Bridge age	Years in 2025	37.1378	20.1734	0.2978 (1.347)
Maximum span	ln(1 + span in m)	3.3693	0.3269	−0.1693 (0.844)
Horizontal clearance	ln(1 + clearance in m)	2.6960	0.3500	0.0845 (1.088)
Under-road truck percentage	Percentage points	10.4666	11.2060	0.1066 (1.113)
Number of under-road lanes	Lanes	3.1026	1.4716	−0.2697 (0.764)

Table 2.3: Estimated functional-class parameters for the Texas overheight-collision model

Class	Coefficient	Rate ratio	Class	Coefficient	Rate ratio
R1	0.1060	1.112	U1	0.4220	1.525
R2	0.4598	1.584	U2	0.3086	1.362
R3	0.5297	1.698	U3	−0.1788	0.836
R4	0.0215	1.022	U4	−1.3643	0.256
R5	−0.7059	0.494	U5	Reference	1.000
R6	−2.5320	0.079	U6	−1.9402	0.144
R7	−3.0131	0.049	U7	−2.6720	0.069

11. **Calculate the empirical-Bayes weight.** Use the fitted ten-year mean and estimated dispersion parameter:

$$w_b = \frac{1}{1 + \alpha \mu_{10,b}} \tag{2.27}$$

where w_b is the empirical-Bayes weight assigned to the model prediction for bridge b; $\alpha = 5.1713$ is the fitted dispersion parameter; and $\mu_{10,b}$ is the fitted ten-year strike mean from Equation 2.26.

12. **Combine the fitted and observed strike counts.** Calculate

$$\widehat{N}_{10,b} = w_b \mu_{10,b} + (1 - w_b) N_{\text{obs},b} \tag{2.28}$$

where $\widehat{N}_{10,b}$ is the empirical-Bayes expected strike count for bridge b during ten years; w_b is the weight from Equation 2.27; $\mu_{10,b}$ is the fitted model mean; and $N_{\text{obs},b}$ is the accepted 2016–2025 crash count. The first term contributes the model estimate and the second term contributes the bridge's observed crash history.

13. **Convert the ten-year estimate to an annual frequency.** Calculate

$$\widehat{N}_{\text{annual},b} = \frac{\widehat{N}_{10,b}}{10} \tag{2.29}$$

where $\widehat{N}_{\text{annual},b}$ is the expected number of overheight strikes per year for bridge b; $\widehat{N}_{10,b}$ is the empirical-Bayes ten-year estimate from Equation 2.28; and 10 is the number of years in the model period.

14. **Rank the bridges.** Sort the 6,647 model-complete bridges from the largest to the smallest value of $\widehat{N}_{\text{annual},b}$. For equal empirical-Bayes frequencies, sort by decreasing model-predicted annual frequency and then by structure identifier.

2.8 Tornado

1. **Select the bridges for analysis.** Begin with 21,349 Texas bridge records having valid locations. Exclude the 15 movable ferry ramps and landings from the statewide ranking. The remaining 21,334 conventional roadway bridges enter the tornado calculation.

2. **Assign each bridge to a FEMA Census tract.** Overlay the bridge coordinates on the Census-tract polygons used in the FEMA National Risk Index. Assign the tract containing the bridge. For a bridge

immediately outside a polygon because of a boundary or coordinate difference, use the nearest valid tract and record the assignment distance.

3. **Obtain tract tornado information.** For the assigned tract, retain FEMA's annual tornado frequency and the tract polygon area in square kilometers. Use the same National Risk Index version for every bridge in the statewide comparison.

4. **Calculate tract tornado-frequency density.** Divide annual tornado frequency by tract area and express the result per 1,000 square kilometers:

$$d_t = \frac{f_t}{A_t} \times 1000 \tag{2.30}$$

where d_t is the annual tornado-frequency density for tract t, expressed per 1,000 square kilometers; f_t is FEMA's annual tornado frequency for tract t; and A_t is the tract area in square kilometers. Dividing by area prevents a large tract from receiving a higher bridge hazard value only because it covers more land.

5. **Calculate the design and clearance component.** Assign raw design category D_b from NBI superstructure type and vertical underclearance using Table 2.4. When reported clearance is greater than 6.1 m, use the high-clearance category; otherwise, use the standard-clearance category. Convert the raw category to the 0–1 component using

$$S_{\text{design}} = \frac{D_b - 1}{4} \tag{2.31}$$

where S_{design} is the design and clearance component and D_b is the raw category from Table 2.4. Raw categories 1, 2, 3, 4, and 5 produce component values 0, 0.25, 0.50, 0.75, and 1.0.

Table 2.4: Tornado design categories from NBI superstructure type and vertical underclearance

NBI superstructure type	Clearance 6.1 m or less, missing, or not applicable	Clearance greater than 6.1 m
1–8, 11–12, 21–22	1	2
9–10	4	5
13–14	4	4
15–17	3	4
18	2	4
19	2	3
20	2	2
Other or missing	1	1

6. **Calculate the span component.** Use maximum span length $L_{\text{max},b}$ from NBI Item 48 to assign raw span category K_b:

$$K_b = \begin{cases} 1, & L_{\text{max},b} \leq 45.7 \text{ m or missing} \\ 2, & 45.7 < L_{\text{max},b} \leq 90 \text{ m} \\ 3, & 90 < L_{\text{max},b} \leq 150 \text{ m} \\ 4, & L_{\text{max},b} > 150 \text{ m} \end{cases} \tag{2.32}$$

where K_b is the raw span category and $L_{\text{max},b}$ is the maximum span length in meters. Convert the category to

$$S_{\text{span}} = \frac{K_b - 1}{3} \tag{2.33}$$

where S_{span} is the 0–1 span component. Categories 1, 2, 3, and 4 produce component values 0, 0.333, 0.667, and 1.0.

7. **Calculate the condition component.** Obtain the deck, superstructure, and substructure condition ratings from NBI Items 58, 59, and 60. Let $C_{\text{min},b}$ be the lowest available rating for bridge b. Calculate

$$S_{\text{condition}} = \frac{9 - C_{\text{min},b}}{8} \tag{2.34}$$

where $S_{\text{condition}}$ is the 0–1 condition component and $C_{\text{min},b}$ is the lowest rating after limiting valid ratings to 1–9. Rating 9 produces 0, rating 8 produces 0.125, and rating 1 produces 1.0. Use 0.5 when all three component ratings are missing.

8. **Calculate structural vulnerability.** Combine the three components using

$$V_b = 0.45S_{\text{design}} + 0.30S_{\text{span}} + 0.25S_{\text{condition}} \tag{2.35}$$

where V_b is the structural-vulnerability component for bridge b; S_{design} is the design and clearance component from Step 5; S_{span} is the maximum-span component from Step 6; and $S_{\text{condition}}$ is the condition component from Step 7. The adopted weights assign 45 percent of structural vulnerability to design and clearance, 30 percent to maximum span, and 25 percent to observed condition.

9. **Convert tornado-frequency density to a statewide percentile.** Order the Texas tracts by d_t and convert each density to a 0–1 percentile, H_t. Assign the tract percentile to every bridge located in that tract. A larger value of H_t represents a greater tornado-frequency density relative to other Texas tracts.

10. **Calculate the final tornado priority score.** Multiply the hazard percentile by bridge vulnerability:

$$R_{\text{tornado}} = 100H_tV_b \tag{2.36}$$

where R_{tornado} is the tornado priority score on a 0–100 scale; H_t is the statewide percentile of tract tornado-frequency density from Step 9; and V_b is the structural-vulnerability component from Equation 2.35. The largest scores occur where relatively high tornado-frequency density coincides with a more vulnerable bridge.

11. **Rank the bridges.** Sort eligible bridges from the largest to the smallest value of R_{tornado}. Resolve equal scores with the fixed structure-identifier order.

2.9 Vessel Collision

1. **Select bridges over active navigable waterways.** Begin with the Texas bridge inventory and retain bridges that cross an active navigable waterway. Use the navigable-waterway layer, bridge feature description, and accepted AIS crossing evidence to confirm the site. The final calculation contains 86 physical crossing sites.

2. **Create one record for each physical crossing site.** Group parallel structures when they cross the same navigation channel at the same location. Retain all associated TxDOT bridge identifiers, the facility carried, the waterway crossed, and the crossing coordinates. This prevents vessel passages at one crossing from being counted separately for parallel bridge records.

3. **Count 2025 vessel passages.** Draw a bridge cross section through the navigation channel. Intersect the cross section with 2025 NOAA Automatic Identification System trajectories. Count one passage each time an accepted vessel trajectory crosses the section. Include all accepted vessel classes. Separate barge-tow passages from ship or other-vessel passages and use the exact annual count for each class as N_i.

4. **Calculate the vessel-traffic density factor.** Let N_{all} be the exact number of accepted 2025 all-vessel passages at the crossing site. Convert N_{all} to the adopted traffic-density factor:

$$R_D = \begin{cases} 1.0, & N_{\text{all}} \leq 2122 \\ 1.3, & 2123 \leq N_{\text{all}} \leq 6558 \\ 1.6, & N_{\text{all}} \geq 6559 \end{cases} \tag{2.37}$$

   where R_D is the vessel-traffic density factor used in the aberrancy calculation and N_{all} is the exact 2025 all-vessel passage count. The thresholds are the project categories derived from TxDOT/CTR 0-4650 Table 1.6.

5. **Calculate vessel aberrancy probability.** For each vessel class i, calculate

$$P_{A,i} = BR_i R_B R_C R_{XC} R_D \tag{2.38}$$

   where $P_{A,i}$ is the aberrancy probability for vessel class i; BR_i is the class-specific base rate, equal to 0.6×10^{-4} for ships or other vessels and 1.2×10^{-4} for barge tows; R_B is the bridge-location factor; R_C is the parallel-current factor; R_{XC} is the cross-current factor; and R_D is the traffic-density factor from Equation 2.37. Calculate $R_B = 1 + \theta/45$, where θ is the channel bend angle in degrees over the 3,000-ft USACE channel-centerline window. Calculate $R_C = 1 + v_{\parallel,95}/10$ and $R_{XC} = 1 + v_{\perp,95}$ from the 95th-percentile parallel and cross-current components assigned from the available current data.

6. **Calculate the geometry and direction factor for the 36 detailed-geometry sites.** For each vessel class, travel direction, and candidate support, calculate

$$P_G = \Phi\left(\frac{y + (B+W)/2}{\text{LOA}}\right) - \Phi\left(\frac{y - (B+W)/2}{\text{LOA}}\right) \tag{2.39}$$

   where P_G is the conditional probability that the aberrant vessel track enters the support impact interval; $\Phi(\cdot)$ is the standard normal cumulative distribution function; y is the support's lateral position relative to the mean vessel track for the applicable class and direction; B is vessel or tow beam; W is support width; and LOA is the effective vessel or tow length used as the lateral standard deviation. Weight the support probabilities by the observed class-and-direction passage counts and retain the controlling-support value for the bridge.

7. **Calculate the geometry factor for the remaining 50 sites.** Use a uniform two-support normal-CDF model when verified support geometry is unavailable. Place the two supports at the edges of the minimum

positive NBI Item 40 horizontal navigation clearance; when Item 40 is unavailable, use the minimum positive NBI Item 48 maximum span as the clearance proxy. Use a 5-m support width. Use a representative ship length of 180 m and beam of 30 m. Use a representative barge-tow length of 86.868 m and beam of 10.668 m. Apply Equation 2.39 to both supports and combine overlapping impact intervals without double counting.

8. **Assign the bridge-protection factor.** Use the controlling NBI Item 111 value across the grouped bridge records. Assign $P_F = 0.10$ for code 2, protection in place and functioning; $P_F = 0.70$ for code 3, protection deteriorated; $P_F = 0.90$ for code 4, reevaluation recommended; and $P_F = 1.00$ for code 5, no protection and reevaluation recommended. Also use $P_F = 1.00$ for code 1 or for an unknown or not-applicable protection code.

9. **Assign the conditional-collision factor.** Use $P_{C,i} = 0.010$ for a ship or other-vessel passage and $P_{C,i} = 0.001$ for a barge-tow passage. Apply the value that corresponds to the vessel class in each term of the annual-frequency calculation.

10. **Calculate annual screening collision frequency.** Combine the class-specific vessel count, aberrancy probability, geometry factor, protection factor, and conditional-collision factor using

$$\mathrm{AFC}_{\mathrm{screen}} = \sum_i N_i P_{A,i} P_{G,i} P_F P_{C,i} \tag{2.40}$$

where $\mathrm{AFC}_{\mathrm{screen}}$ is the expected number of vessel collisions per year; i identifies vessel class and, for the detailed sites, travel direction; N_i is the exact number of accepted 2025 AIS passages represented by term i; $P_{A,i}$ is the aberrancy probability from Equation 2.38; $P_{G,i}$ is the geometry and direction factor from Equation 2.39; P_F is the bridge-protection factor; and $P_{C,i}$ is the class-specific conditional-collision factor. Calculate one product for each applicable class and direction and add the products.

11. **Convert annual frequency to a one-year probability.** Calculate

$$P_{1\mathrm{yr}} = 1 - \exp(-\mathrm{AFC}_{\mathrm{screen}}) \tag{2.41}$$

where $P_{1\mathrm{yr}}$ is the probability of one or more vessel collisions during one year and $\mathrm{AFC}_{\mathrm{screen}}$ is the annual screening collision frequency from Equation 2.40.

12. **Calculate the low and high geometry cases.** For the detailed-geometry sites, calculate the raw AIS tug/tow, nominal one-barge-plus-towboat, and multi-barge-tow geometry cases. For the two-support proxy sites, use support widths of 0, 5, and 10 m for the low, central, and high cases. Recalculate annual frequency for each case and use the central case for statewide ranking.

13. **Rank the crossing sites.** Sort all 86 completed crossing-site records from the largest to the smallest central value of $\mathrm{AFC}_{\mathrm{screen}}$. Use the crossing-site identifier to order equal values.

2.10 Wildfire

1. **Locate the bridge.** Use the reported bridge coordinates to place the bridge on the map. Use the mapped bridge line or the direction of the roadway at the bridge to determine the bridge orientation. This step establishes the bridge center and the direction in which the bridge extends.

2. **Build the bridge footprint.** Create a polygon that represents the full bridge deck. Use the NBI/SNBI bridge length for the long dimension and the reported or estimated deck width for the short dimension. Align the polygon with the bridge orientation established in Step 1. This footprint is more representative of the bridge than a single coordinate point.

3. **Create the 15-m analysis zone.** Extend the bridge footprint outward by 15 m on every side. The resulting zone includes the bridge deck and the immediately surrounding land. All wildfire-hazard and fuel calculations are limited to this zone so that the score reflects conditions close to the bridge.

4. **Obtain wildfire-hazard values.** Overlay the 15-m analysis zone on the 30-m USDA Wildfire Hazard Potential (WHP) raster. Identify every WHP pixel that overlaps the zone and retain the WHP value for each pixel. A larger WHP value indicates a greater potential for wildfire at that location.

5. **Normalize the hazard value for each WHP pixel.** Convert each WHP value to a number between 0 and 1 using

$$h_k = \min\left(\frac{W_k}{492}, 1\right) \tag{2.42}$$

where h_k is the normalized wildfire hazard for WHP pixel k; W_k is the USDA WHP value for that pixel; and 492 is the adopted Texas reference value. For example, a WHP value of 246 produces $h_k = 0.5$. A WHP value of 492 or greater produces $h_k = 1.0$.

6. **Identify burnable fuel from NAIP imagery.** Classify the NAIP aerial imagery inside the 15-m zone as tree cover, low vegetation, wetland, water, roadway or other impervious surface, or other land cover. Tree cover and low vegetation, including dry grass, are treated as burnable fuel. Water, roadways, and other impervious surfaces are not treated as fuel.

7. **Measure the fuel area within each WHP pixel.** For every WHP pixel k, measure only the area that is both inside the 15-m analysis zone and classified as burnable fuel. If only part of a WHP pixel overlaps the zone, use only the overlapping portion. This produces the fuel area associated with each normalized hazard value.

8. **Calculate the effective fuel-weighted hazard.** Combine the normalized WHP values with the mapped fuel areas using

$$H_{\text{effective}} = \frac{\sum_k h_k A_{\text{fuel},k}}{A_{15\text{m}}} \tag{2.43}$$

where $H_{\text{effective}}$ is the effective wildfire hazard at the bridge; h_k is the normalized wildfire hazard for pixel k from Equation 2.42; $A_{\text{fuel},k}$ is the area of pixel k that lies inside the bridge zone and is classified from NAIP as burnable fuel; and $A_{15\text{m}}$ is the full area of the 15-m analysis zone. This calculation gives a larger value when high-WHP locations occupy a larger fuel-covered portion of the bridge surroundings. If the zone contains no mapped fuel, $H_{\text{effective}} = 0$.

9. **Assign the bridge-material vulnerability factor.** Use the NBI/SNBI bridge material and structure-type descriptions to assign V_b. The final mapping assigns $V_b = 0.4$ to concrete, prestressed-concrete, and masonry bridges. It assigns $V_b = 0.7$ to steel, aluminum or iron, fiber-reinforced-polymer, and wood or timber bridges. This factor represents the relative vulnerability of the bridge material to wildfire exposure.

10. **Calculate the final wildfire priority score.** Calculate

$$R_{\text{wildfire}} = 100 H_{\text{effective}} V_b \tag{2.44}$$

where R_{wildfire} is the wildfire priority score on a 0–100 scale; $H_{\text{effective}}$ is the effective fuel-weighted hazard from Equation 2.43; and V_b is the bridge-material vulnerability factor assigned in Step 9. Multiplication by 100 expresses the result on the reporting scale.

11. **Rank the bridges.** Include bridges for which the NAIP analysis is complete. Sort them from the largest to the smallest value of R_{wildfire}. A bridge with a larger score receives a higher statewide wildfire priority. If two bridges have the same score, rank the bridge with the larger average annual daily traffic first.

2.11 Final Multi-Hazard Recurrence Method

The final recurrence calculation uses the adopted statewide risk lists that contribute to the multi-hazard analysis. Let $\mathcal{R}$ denote this set of contributing lists.

1. **Standardize bridge identifiers and join the contributing lists.** Preserve leading zeros in bridge structure identifiers and join the statewide risk-rank files by the standardized identifier. A bridge absent from a risk list remains unranked for that risk.
2. **Assign rank-band points.** Calculate

$$s(q_{br}) = \begin{cases} 3, & 1 \le q_{br} \le 20 \\ 2, & 21 \le q_{br} \le 50 \\ 1, & 51 \le q_{br} \le 100 \\ 0, & q_{br} > 100 \text{ or unranked} \end{cases} \tag{2.45}$$

where b identifies a bridge; r identifies a risk list in $\mathcal{R}$; q_{br} is bridge b's statewide rank in risk r; and $s(q_{br})$ is the recurrence-point value assigned to that rank band.

3. **Calculate the recurrence index.** Calculate

$$R_{\text{MH},b} = \sum_{r \in \mathcal{R}} s(q_{br}) \tag{2.46}$$

where $R_{\text{MH},b}$ is the multi-hazard recurrence index for bridge b. An unranked risk contributes zero points.

4. **Record the mutually exclusive rank-band counts.** Calculate

$$\begin{aligned} N_{20,b} &= \sum_{r \in \mathcal{R}} \mathbf{1}(1 \le q_{br} \le 20) \\ N_{50,b} &= \sum_{r \in \mathcal{R}} \mathbf{1}(21 \le q_{br} \le 50) \\ N_{100,b} &= \sum_{r \in \mathcal{R}} \mathbf{1}(51 \le q_{br} \le 100) \end{aligned} \tag{2.47}$$

where $\mathbf{1}(\cdot)$ equals 1 when the stated rank condition is true; $N_{20,b}$, $N_{50,b}$, and $N_{100,b}$ count bridge b's appearances in ranks 1–20, 21–50, and 51–100. Equivalently,

$$R_{\mathrm{MH},b} = 3N_{20,b} + 2N_{50,b} + N_{100,b} \tag{2.48}$$

where all terms are defined above.

5. **Assign the final statewide order.** Rank bridges by decreasing $R_{\mathrm{MH},b}$, followed by the fixed tie-break sequence. Flag for focused review any bridge that appears in the Top 10 of at least one contributing risk. Retain the contributing risk, rank band, and point value for every nonzero contribution.

2.12 Limitations of the Screening Method

The statewide framework is a screening method, and four features govern how its ranks should be interpreted. First, the weights and thresholds come from the cited guidance, published methods, or the project scoring choices stated in each subsection. For example, Table 2.1 retains the AHP weights from NCHRP 20-07/Task 378, whereas the fatigue and tornado subsections state project weights for combining their exposure and vulnerability components.

Second, a tie-breaking rule creates a fixed table order but does not create a difference in the underlying risk measure. The hurricane-surge Top 10 illustrates this point because all ten probabilities round to the same six-decimal value. Staff should review tied bridges as a group when the reported measure is equal.

Third, eligibility depends on the required source data. A bridge with missing clearance, low-chord, stream-match, element, traffic, or geometry information can remain unranked for the affected module. Section 3 reports the ranked and unranked populations so that absence from a Top 10 is not read as evidence of low risk.

Fourth, raw scores are not comparable across risk categories because the modules represent different hazards, eligible populations, units, and scoring scales. The recurrence method therefore combines rank-band appearances rather than adding the raw risk scores. Bridge review begins with the risk-specific inputs that produced each rank.

SECTION 3. Risk-Specific Statewide Results

3.1 Introduction

The statewide results show that the ten adopted risk modules select different bridge populations and produce different geographic and numerical patterns. Each section defines the ranked population, identifies the leading bridges, and interprets the pattern, score distribution, or data gap that most affects use of the ranking. A Top 10 map and bridge table accompany every module.

Eligibility differs by module because each hazard requires a different combination of inventory, exposure, condition, traffic, and incident records. Therefore, a bridge that does not appear in a Top 10 may either rank below the displayed cutoff or lack one of the inputs required by that module. TxDOT district boundaries provide geographic context on the maps, and rank labels correspond directly to the accompanying tables.

3.2 Explosive/Security Criticality

The statewide analysis contains 21,349 explosive/security criticality screening records. Scores are tightly grouped at the top: Structure 091610020901034, carrying LP 2 eastbound/18th Street across UPRR, Mary Avenue, and Waco Creek in Waco District, ranks first at 55.556, whereas rank 10 is 50.353. This five-point range shows that no single structure dominates the statewide list; instead, the leading group combines comparable measures of occurrence, vulnerability, and consequence.

Houston District contains five of the Top 10, including three structures on SH 332. Pharr contains two, and Austin, Dallas, and Waco each contain one. The Houston concentration reflects repeated combinations of major transportation corridors, industrial or transportation assets, and network consequences rather than one input acting alone.

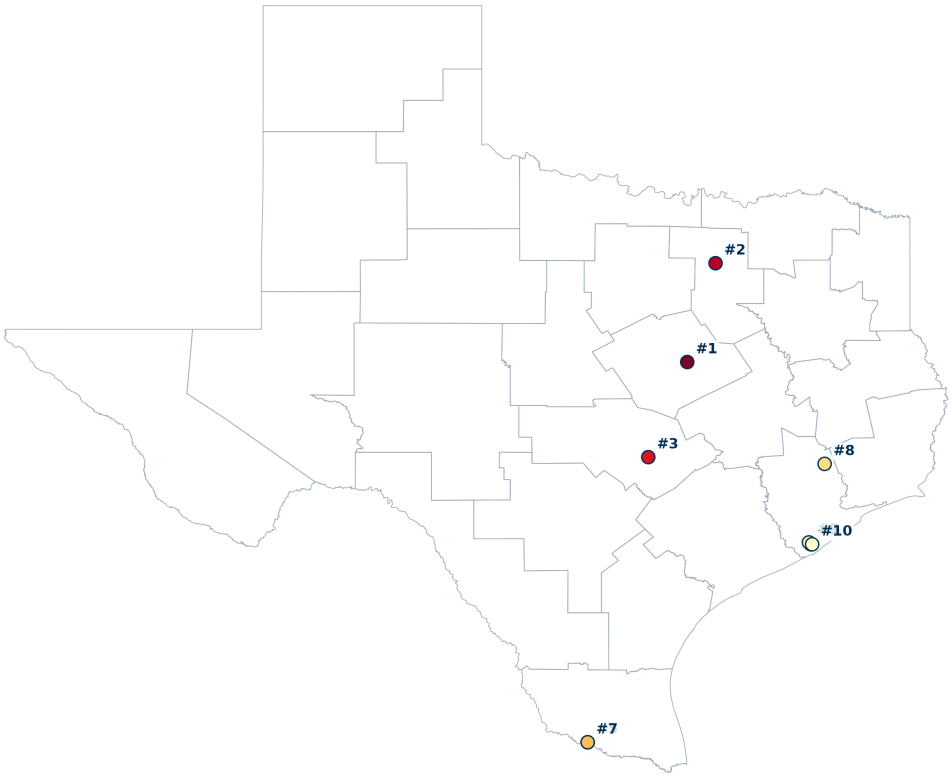


Figure 3.1: Statewide Top 10 for explosive/security criticality. Rank labels correspond to Table 3.1.

Table 3.1: Final Statewide Top 10 for Explosive/Security Criticality.

Rank	Structure ID	Facility/location	Feature crossed	District	Criticality factor
1	091610020901034	LP 2 EB / 18TH ST	UPRR MARY AVE WACO CRK	Waco	55.556
2	180570237401471	RAMP W-GREEN	TI BLVD; DART; EB RMP SE	Dallas	54.798
3	142270015201079	SH 71 EBML	US 183; Carson Creek	Austin	53.825
4	120200152401078	SH 332 EB	DIXIE DRIVE	Houston	52.594
5	211090003902469	US 83 W-WB	Bus US 83 WB	Pharr	52.550
6	120200152401077	SH 332 WB	DIXIE DRIVE	Houston	52.445
7	211090003902441	US 83 W-EB	Bus US 83 WB	Pharr	52.395
8	121700017705552	IH 69 NB TO SH99WB	IH 69 WHITE OAK CREEK	Houston	52.041
9	120200152401074	SH 332 WB	OAK DRIVE	Houston	51.185
10	120200152401079	SH 332 WB	MAIN STREET	Houston	50.353

3.3 Fatigue

The statewide analysis ranks 778 eligible fatigue bridge sites. The leading site is Structure 121020027115377, which carries IH 610 across the Houston Ship Channel in Houston District and has a fatigue score of 90.000; the rank 10 score is 74.770. The comparatively narrow upper range identifies a group of bridges where high truck demand and fatigue-sensitive structural characteristics occur together.

Houston District contains three of the Top 10, Beaumont and Paris contain two each, and Austin, Dallas, and

Yoakum contain one each. Freight demand does not produce one metropolitan cluster; the ten sites extend from coastal Houston and Beaumont to inland Paris, Austin, Dallas, and Yoakum. Parallel bridge records at the same crossing are grouped as one fatigue site, which prevents a divided highway crossing from occupying multiple positions because of duplicated exposure.

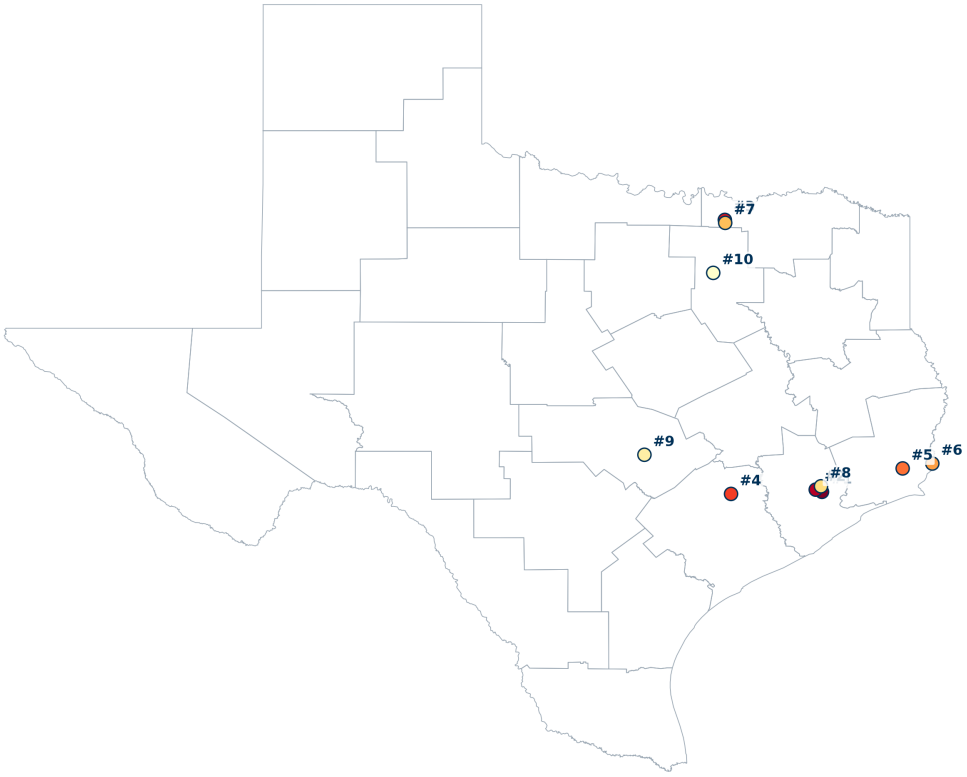


Figure 3.2: Statewide Top 10 for fatigue. Rank labels correspond to Table 3.2.

Table 3.2: Final Statewide Top 10 for Fatigue.

Rank	Structure ID	Facility/location	Feature crossed	District	Fatigue score
1	121020027115377	IH 610	HOUSTON SHIP CHANNEL	Houston	90.000
2	121020017711091	IH 69	CAPITOL ST. ETC.	Houston	82.827
3	010920004713174	US 75	SPUR 381	Paris	82.731
4	130450053508005	IH 10 WB / EB	Colorado River	Yoakum	82.539
5	201240073902120	IH 10 WB / IH 10 EB	IH 10 WB TO US 69 SB	Beaumont	81.521
6	201810002814102	IH 10 WB	US 90 BUS	Beaumont	79.929
7	010920004713176	US 75	W FARMING-TON/BLYTHE RD	Paris	77.648
8	121020027114120	IH 610 NB / IH 610 SB	UP RR & OLD LIB RD	Houston	77.164
9	142270001513104	IH 35	WOODLAND AVE	Austin	76.884
10	180570009214198	IH 345 NB / SB	IH 30 US 75 DART etc	Dallas	74.770

3.4 Flood Overtopping and Scour

The statewide analysis contains 12,479 waterway and flood-relief bridge records. Of these, 8,909 have both component ranks required for the combined result, whereas 3,570 records, or 28.6 percent, have incomplete combined-ranking data. Because the final measure averages the overtopping and scour ranks, both inputs are required before a bridge can be placed in the statewide combined order.

Structure 150100173001001, carrying FM 1283 across Red Bluff Creek in San Antonio District, ranks first with a mean component rank of 15.0. Rank 10 has a mean component rank of 84.0, with smaller values indicating greater combined priority. Eight of the Top 10 carry FM or RM routes, and the district distribution is five in San Antonio, three in Austin, and two in Waco. The result therefore emphasizes creek crossings on farm-to-market and ranch-to-market routes in central Texas rather than interstate structures, because those sites combine low hydraulic margins with high modeled scour priority.

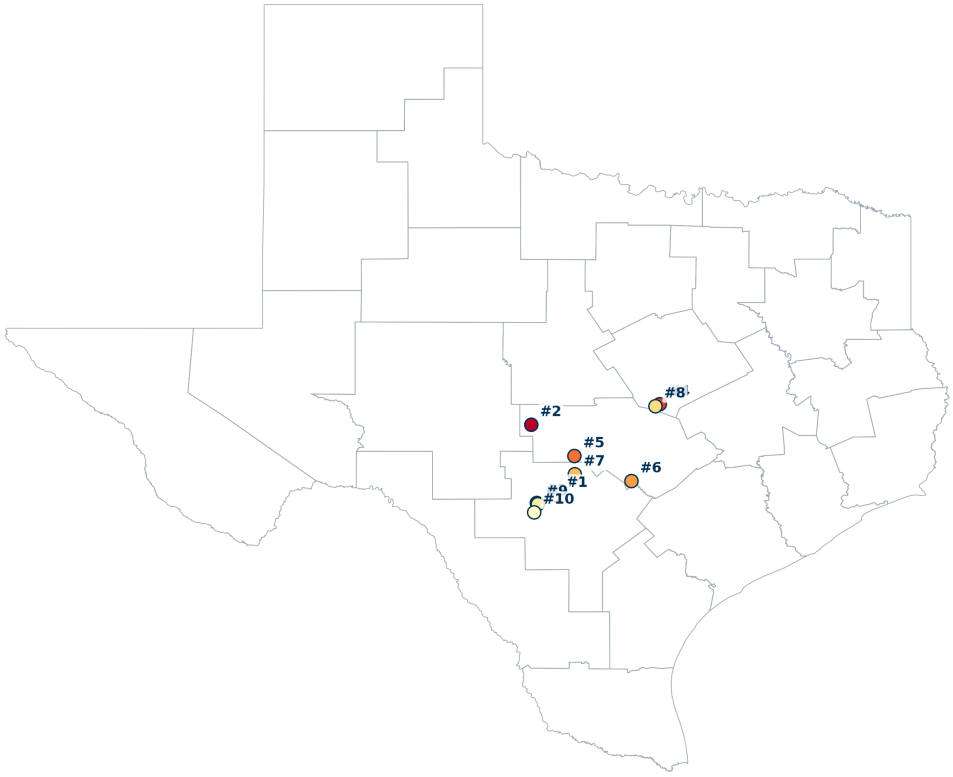


Figure 3.3: Statewide Top 10 for flood overtopping and scour. Rank labels correspond to Table 3.3.

Table 3.3: Final Statewide Top 10 for Flood Overtopping and Scour.

Rank	Structure ID	Facility/location	Feature crossed	District	Mean component rank
1	150100173001001	FM 1283	RED BLUFF CREEK	San Antonio	15.0
2	141570175301005	RM 1871	Llano River	Austin	24.5
3	151630084804023	FM 462	HONDO CREEK NO 1	San Antonio	36.5
4	090140213601005	FM 2268	SALADO CREEK	Waco	38.5
5	140870011302468	US 290	SOUTH GRAPE CREEK	Austin	48.5
6	141060001603465	IH35 NBFR	San Marcos River	Austin	51.0

Rank	Structure ID	Facility/location	Feature crossed	District	Mean component rank
7	151310014209156	FM 473	EAST SISTER CREEK	San Antonio	68.0
8	090140287001003	FM 2843	SALADO CREEK	Waco	72.0
9	151630084804025	FM 462	HONDO CREEK NO 3	San Antonio	72.5
10	151630059502001	FM 1796	SECO CREEK	San Antonio	84.0

3.5 Hurricane Surge Exposure

The statewide analysis contains 13,193 coastal-screening records. Positive modeled surge exposure occurs at 789 bridges, 12,305 records have zero modeled exposure, and 99 records, or 0.8 percent, lack the clearance data required for the calculation. The principal statewide distinction is therefore between bridges with positive surge exposure and those with none.

All ten leading bridges lie in Houston District and have annual exposure probabilities that round to 0.048771. Structure 120850050001286, carrying IH 45 across the Tidal Interchange, is listed first. Because differences among these probabilities are smaller than the six-decimal reporting precision, their table order follows the Section 2 tie-breaking rules rather than a meaningful difference in modeled exposure. District review should treat ranks 1–10 as a leading group and use bridge-specific clearance and route conditions to set the order of field review.

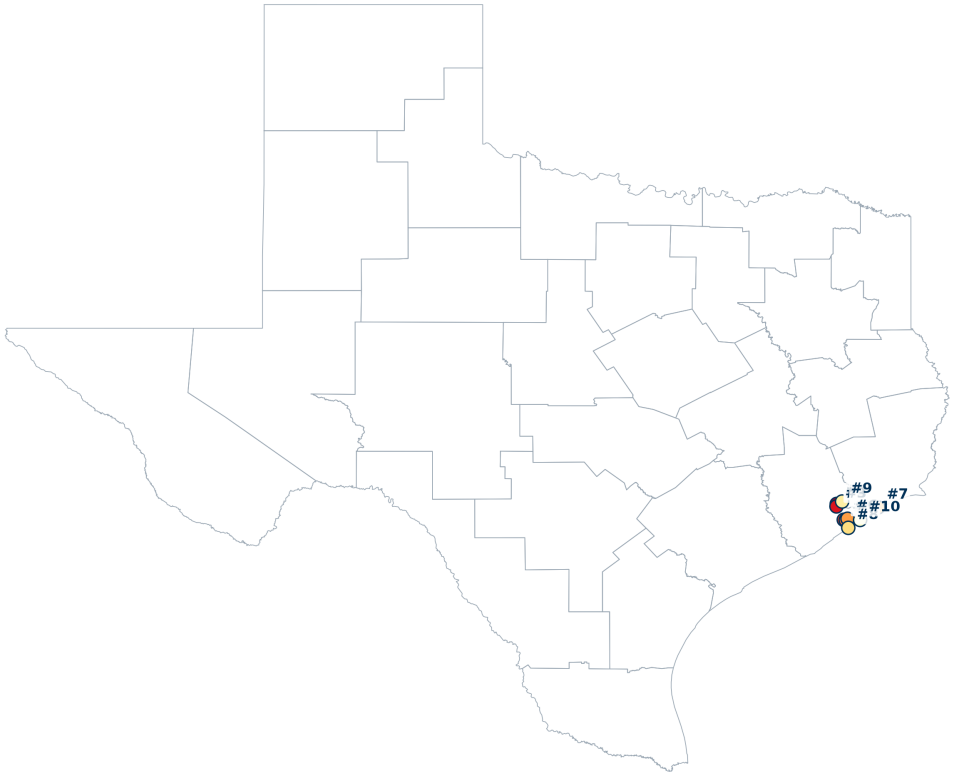


Figure 3.4: Statewide Top 10 for hurricane surge exposure. Rank labels correspond to Table 3.4.

Table 3.4: Final Statewide Top 10 for Hurricane Surge Exposure.

Rank	Structure ID	Facility/location	Feature crossed	District	Annual surge-exposure probability
1	120850050001286	IH 45	TIDAL INTERCHANGE	Houston	0.048771
2	121020098101018	NASA RD 1	HL & P CANAL	Houston	0.048771
3	120850331202003	FM 270	ROBINSON BAYOU	Houston	0.048771
4	120850019204020	SH 6	DRAINAGE DITCH	Houston	0.048771
5	120850019204022	SH 6	DRAINAGE DITCH	Houston	0.048771
6	120850038911085	LP 197	DRAINAGE DITCH	Houston	0.048771
7	120850036703029	SH 87	ROLLOVER PASS	Houston	0.048771
8	120850B00825001	STEWART RD	MENBELL BYU	Houston	0.048771
9	121020AA6469001	TODVILLE RD	PINE GULLY	Houston	0.048771
10	120850B00270079	BODDEKER RD.	CLOUD BAYOU	Houston	0.048771

3.6 Heavy-Vehicle Bridge Pier Collision

The statewide analysis ranks 2,159 bridge-pier or bridge-support collision sites identified in the 2016–2025 CRIS review. Structure 150150052104285, carrying IH 410 westbound across Ingram Road in San Antonio District, ranks first with 46 assigned crashes. The second-ranked site has 16 assigned crashes and rank 10 has 11, so the IH 410/Ingram Road site is a clear statewide outlier rather than the upper end of a gradual distribution.

The Top 10 include two sites each in Fort Worth, Houston, and San Antonio districts, one each in Dallas and Pharr districts, and two bridge-support collision sites represented by non-NBI identifiers. Those two locations remain in the results because the crash records establish a bridge-support impact location even though an NBI structure identifier was not assigned. The large separation at rank 1 supports reviewing that location first, followed by the tighter group represented by ranks 2–10.

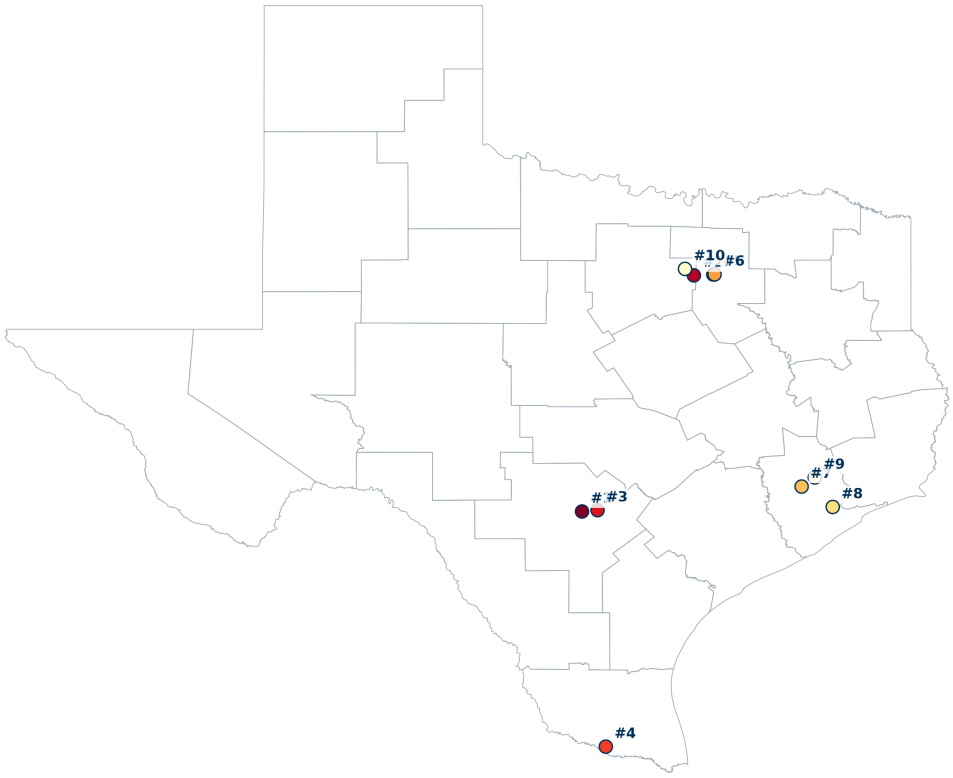


Figure 3.5: Statewide Top 10 for heavy-vehicle bridge pier collision. Rank labels correspond to Table 3.5.

Table 3.5: Final Statewide Top 10 for Heavy-Vehicle Pier Collision.

Rank	Structure ID	Facility/location	Feature crossed	District	Assigned crashes, 2016–2025
1	150150052104285	IH 410 WB	Ingram Rd	San Antonio	46
2	022200226602023	SH 360 SBL	RANDOL MILL ROAD	Fort Worth	16
3	150150001710188	IH 35 NB ML	RITTIMAN RD	San Antonio	15
4	211090003917323	US 83 EB	FM 494 (SHARY RD)	Pharr	15
5	SITE_DALLAS_CORINTH_ST_RAIL_YARD_UNDERPASS	Dallas Corinth St Rail Yard Underpass	Confirmed underpass pier/support collision location	Not assigned	13
6	180570009214198	IH 345 NB	IH 30 US 75 DART etc	Dallas	12
7	OSM_WAY_126987165	STATE LOOP 0008	Confirmed underpass pier/support collision location	Not assigned	12
8	121020050003427	IH 45	FM 528 NASA PKWY	Houston	11
9	121020325602004	BW 8	HARDY TOLL RD & UPRR	Houston	11
10	022200036401671	SH 121/183 WB GP	FM 3029/Precinct Line Rd	Fort Worth	11

3.7 Overload

The statewide analysis contains 21,349 bridge records and ranks 15,574 records with a finite positive annual load-effect exceedance value. Two adjacent Dallas District structures, 180570237403192 and

180570237403193, share the leading estimate of approximately 515,962 trucks per year whose calculated load effect exceeds the Item 64-equivalent threshold. The rank 10 estimate is approximately 70,378, making the leading pair more than seven times higher than the lower end of the Top 10.

Dallas District contains four of the Top 10 and Austin District contains three; Beaumont, Corpus Christi, and Houston each contain one. The paired Dallas result shows why the structure number must remain the controlling identifier: adjacent bridge records can share the same traffic stream and capacity condition while representing separate structures. Freight demand and lower bridge-specific rating thresholds coincide most often among the Dallas and Austin entries; the remaining three districts show that this loading condition also occurs outside those two networks.

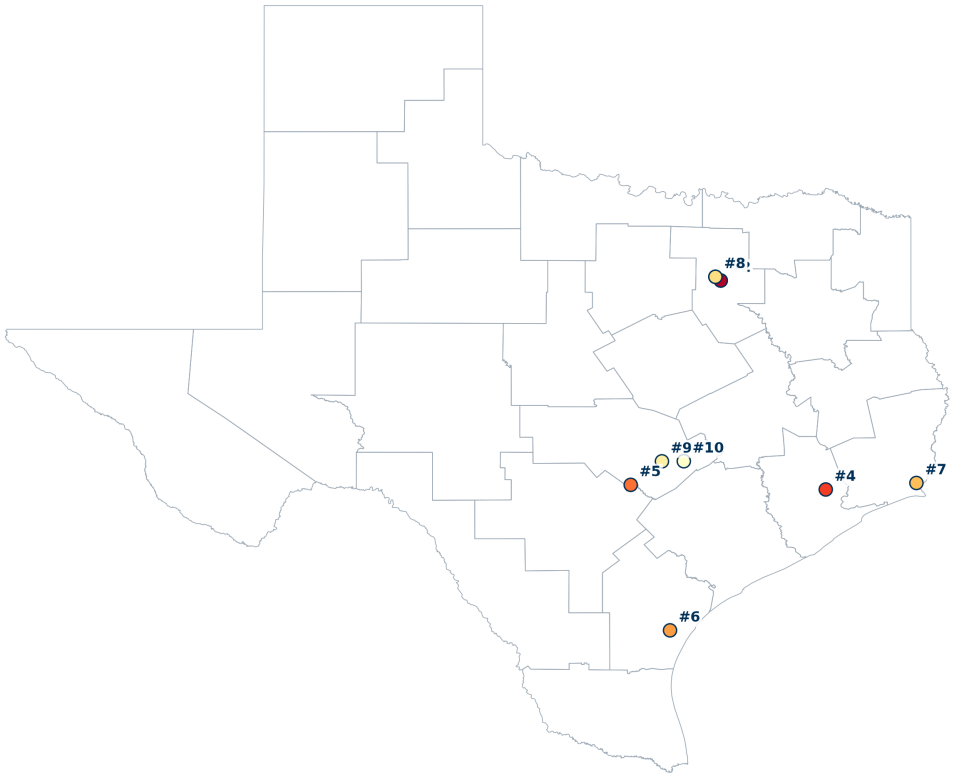


Figure 3.6: Statewide Top 10 for overload. Rank labels correspond to Table 3.6.

Table 3.6: Final Statewide Top 10 for Overload.

Rank	Structure ID	Facility/location	Feature crossed	District	Annual trucks above threshold
1	180570237403192	2.5 MI E OF IH 45	NA	Dallas	515,962.4
2	180570237403193	2.5 MI E OF IH 45	NA	Dallas	515,962.4
3	180570009214256	1.8 MI S OF IH 30	NA	Dallas	417,259.8
4	121020050801023	2.0 MI E OF IH 610	NA	Houston	202,963.1
5	141060036601002	2.1 MI S OF IH 35	NA	Austin	189,646.2
6	161780010106041	3.16 M SW OF NUECES CO LN	NA	Corpus Christi	104,290.4
7	201240030701009	0.4 MI SW OF SH 82	NA	Beaumont	94,008.3
8	180570009214257	1.8 MI S OF IH 30	NA	Dallas	83,312.1
9	140110026503046	5.3 MI W OF FM 1209	NA	Austin	74,684.9

Rank	Structure ID	Facility/location	Feature crossed	District	Annual trucks above threshold
10	140110047201002	1.6 MI E OF FM 1441	NA	Austin	70,377.9

3.8 Overheight Collision

The statewide analysis contains 9,223 candidate under-road bridge records. Complete model inputs are available for 6,647 records, whereas 2,576 records, or 27.9 percent, remain unranked because one or more required inputs are absent. This gap is large enough that clearance and roadway data completion should accompany use of the ranking.

Structure 121020027107240, carrying Houston Avenue across IH 10/White Oak Bayou in Houston District, ranks first with an empirical-Bayes expected strike frequency of 3.457 per year. The second-ranked value is 0.389 and rank 10 is 0.224; thus, the leading location is nearly nine times the second-ranked value. Houston contains four of the Top 10, while Atlanta, Bryan, Dallas, Fort Worth, Laredo, and Odessa each contain one. The geographic spread is wider than the four Houston entries alone suggest: six additional districts contribute one bridge each. Their shared feature is the joint influence of strike history, clearance, roadway geometry, and truck exposure; the Houston Avenue crossing remains a separate outlier requiring location-specific review.

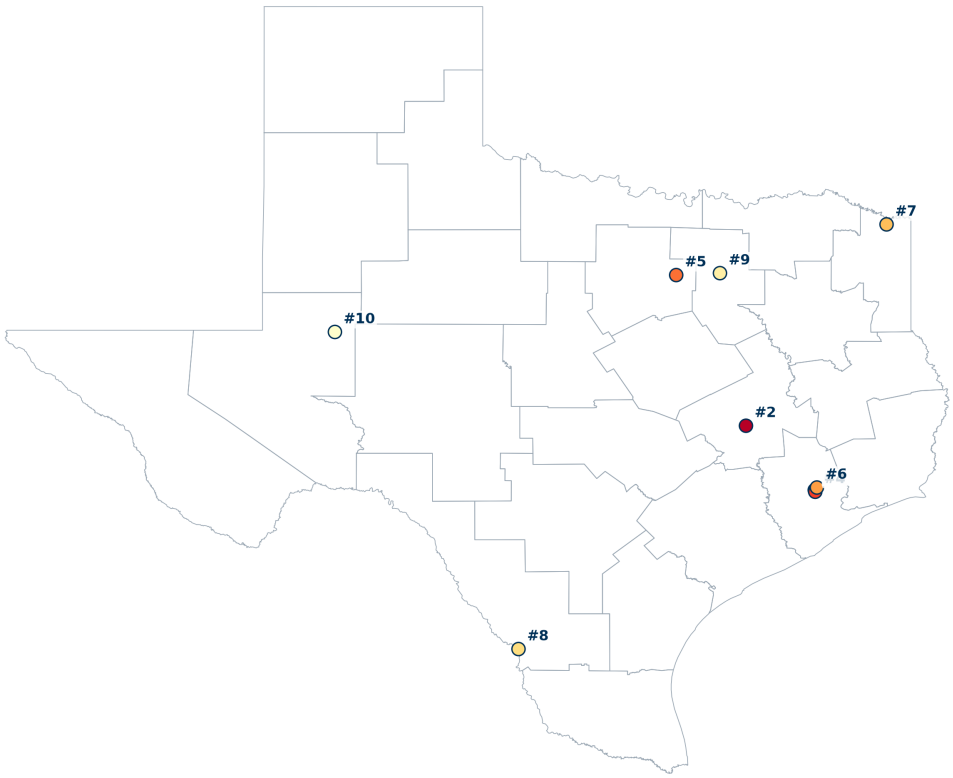


Figure 3.7: Statewide Top 10 for overheight collision. Rank labels correspond to Table 3.7.

Table 3.7: Final Statewide Top 10 for Overheight Collision.

Rank	Structure ID	Facility/location	Feature crossed	District	EB expected annual strikes
1	121020027107240	HOUSTON AVE	IH 10 / WHITE OAK BAYOU	Houston	3.457
2	170210004912089	FM 158	SH 6	Bryan	0.389
3	121020027107191	TAYLOR ST	IH 10	Houston	0.368
4	121020050003068	JEFFERSON ST EXIT	CALHOUN ST TO IH 45 NB	Houston	0.350
5	022200036303014	N SYLVANIA AVE	SH 121	Fort Worth	0.310
6	121020248301007	SPUR 548 CONN C	KELLEY ST & IH 610	Houston	0.276
7	190190061006114	CR 2003(RED BAYOU)	IH 30	Atlanta	0.259
8	222400001806085	IH 35 ML	DEL MAR BLVD	Laredo	0.258
9	180570000911201	Jim Miller Rd	IH 30	Dallas	0.224
10	061650000515192	Lamesa Rd	IH 20	Odessa	0.224

3.9 Tornado

The statewide analysis contains 21,349 inventory records and ranks 21,334 conventional bridge records after removing 15 records outside the module definition. Structure 121020038912090, carrying SH 146 southbound across the Houston Ship Channel in Houston District, ranks first with a tornado risk index of 72.913. Its tract hazard percentile is 0.997094, while the second-ranked parallel SH 146 structure has a percentile of 0.855296; both share the same 381-m maximum span, design score, and vulnerability value of 0.73125. The difference between the first two ranks therefore comes from modeled tornado exposure rather than structural form.

Houston and Dallas districts each contain three of the Top 10, and Corpus Christi, Fort Worth, Pharr, and Waco each contain one. Long-span Houston Ship Channel bridges rise to the top because high structural vulnerability coincides with high local tornado exposure. Inland bridges enter the same list when a different combination of exposure and structural factors yields a comparable product, which explains the statewide distribution beyond Houston.

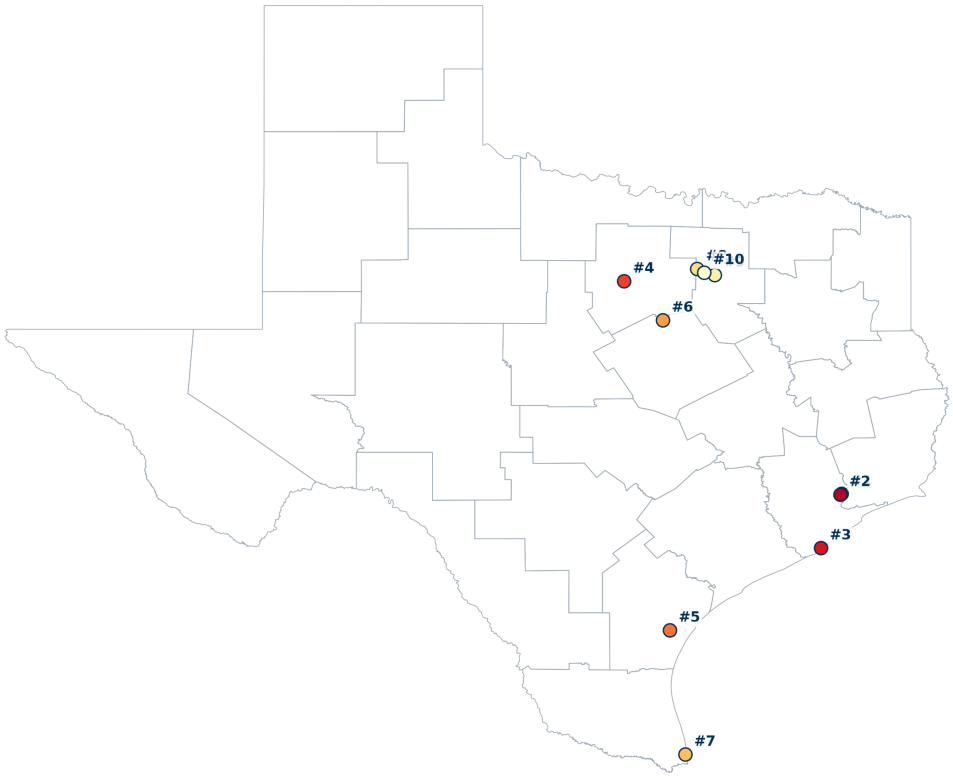


Figure 3.8: Statewide Top 10 for tornado risk. Rank labels correspond to Table 3.8.

Table 3.8: Final Statewide Top 10 for Tornado.

Rank	Structure ID	Facility/location	Feature crossed	District	Tornado risk index
1	121020038912090	SH 146 SB	HOUSTON SHIP CHANNEL	Houston	72.913
2	121020038912089	SH 146 NB	HOUSTON SHIP CHANNEL	Houston	62.543
3	120200058601013	SH 332	INTRACOASTAL CANAL	Houston	40.513
4	021840031401006	IH 20 NFR	BRAZOS RIVER	Fort Worth	40.009
5	161780010106041	US 181	CC SHIP CHANNEL	Corpus Christi	35.162
6	091100051902005	SH 174	BRAZOS RIVER	Waco	33.825
7	210310033104005	PR 100	INTRACOASTAL WATERWAY	Pharr	32.458
8	180570296401461	SH 161 NB CONN G	SH 183	Dallas	31.338
9	180570009214407	IH 45 SB ON RAMP	LAMAR STREET & UPRR	Dallas	30.935
10	180570058102073	LP 12 SB	West Fork Trinity River	Dallas	30.526

3.10 Vessel Collision

The statewide analysis ranks 86 bridge-crossing sites with vessel-collision screening results. Structure 121020038905081, carrying SH 146 across Clear Creek and Shipyard Drive in Houston District, ranks first with a one-year screening probability of 0.001449; rank 10 is 0.000281. The leading probability is more

than five times the rank 10 value, which identifies a meaningful upper tier even within the limited navigable-waterway population.

Seven of the Top 10 sites are in Houston District, two are in Yoakum, and one is in Beaumont. This coastal concentration follows the combination of vessel activity, navigation geometry, current, and bridge configuration used by the module. Some sites list paired structure identifiers because parallel bridge records cross the same navigation channel; grouping them as one site prevents the same vessel approach from being counted twice in the statewide ranking.

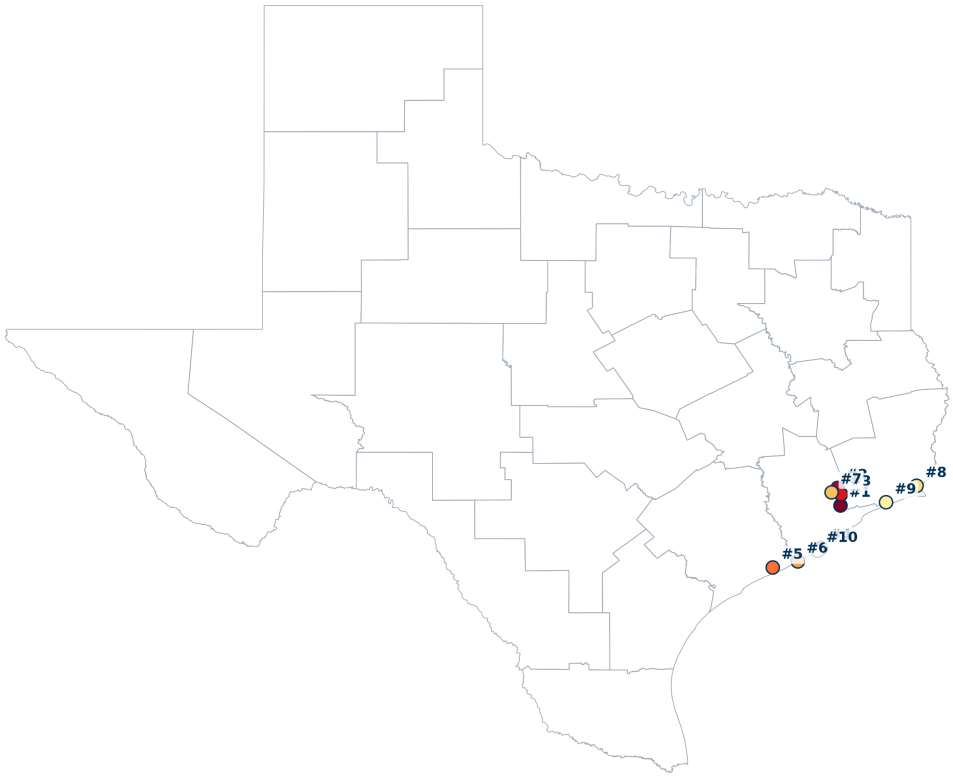


Figure 3.9: Statewide Top 10 for vessel collision. Rank labels correspond to Table 3.9.

Table 3.9: Final Statewide Top 10 for Vessel Collision.

Rank	Structure ID	Facility/location	Feature crossed	District	One-year screening probability
1	121020038905081	SH 146	CLEAR CRK & SHIPYARD DR	Houston	0.001449
2	121020050801222; 121020050801317	IH 10 WB	SAN JACINTO RIVER	Houston	0.001288
3	121020038912090; 121020038912089	SH 146 NB/SB	HOUSTON SHIP CHANNEL	Houston	0.000902
4	120200058601013	SH 332	INTRACOASTAL CANAL	Houston	0.000776
5	131580060401004	FM 2031	INTRACOASTAL WATERWAY	Yoakum	0.000697
6	131580060501080	FM 457	INTRACOASTAL WATERWAY	Yoakum	0.000604

Rank	Structure ID	Facility/location	Feature crossed	District	One-year screening probability
7	121020325603075	BELTWAY 8	HOUSTON SHIP CHANNEL	Houston	0.000594
8	201240030701014	SH 87	Intracoastal Canal	Beaumont	0.000458
9	120850036702023	SH 124	INTRACOASTAL CANAL	Houston	0.000320
10	120200058701005	FM 1495	GULF-INTRACOASTAL WTRWAY	Houston	0.000281

3.11 Wildfire

The statewide analysis contains 21,349 bridge records. Positive wildfire scores are available for 11,777 bridges, 9,567 have no modeled exposure, and five lack the road match needed to complete the calculation. The result separates locations where mapped hazard overlaps burnable fuel in the bridge analysis zone from locations with no modeled overlap.

Structure 041180055702003, carrying SH 152 across the East Fork of Big Creek in Amarillo District, ranks first with a wildfire priority score of 24.347. Its effective hazard is 0.3478, 86.2 percent of the analysis zone is classified as vegetation, and its material-vulnerability factor is 0.7. Amarillo contains ranks 1, 3, 5, and 9 because several Panhandle bridges combine high Wildfire Hazard Potential with continuous fuel near vulnerable structural materials. Houston contains two Top 10 bridges, while Abilene, Austin, Bryan, and Wichita Falls each contain one, showing that high priority can also result from localized fuel and material conditions outside the Panhandle.

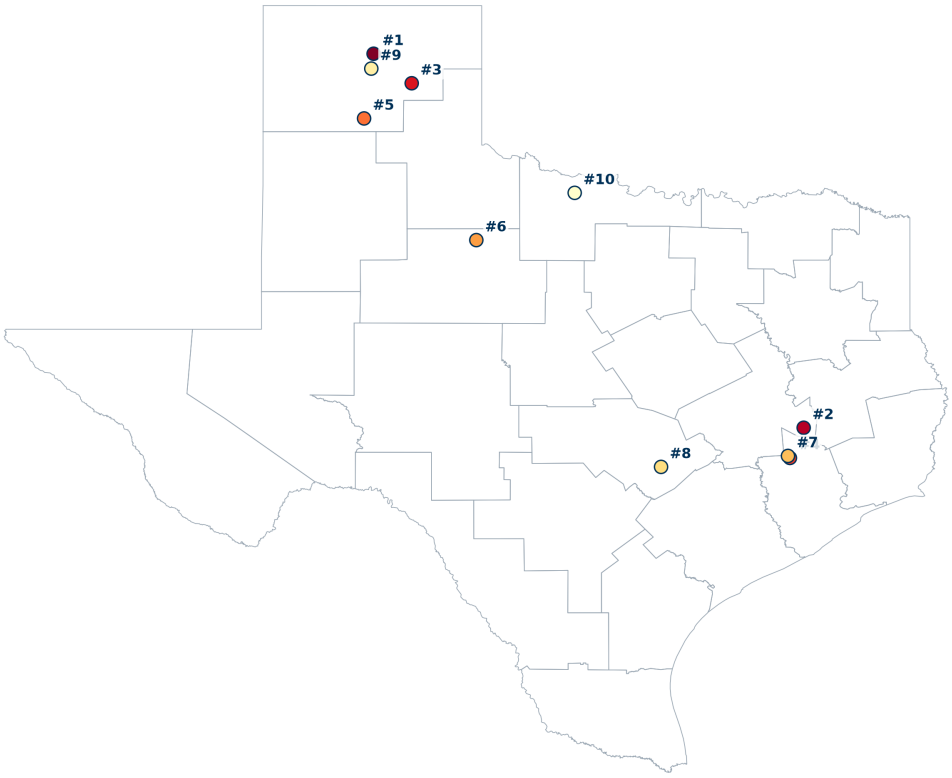


Figure 3.10: Statewide Top 10 for wildfire risk. Rank labels correspond to Table 3.10.

Table 3.10: Final Statewide Top 10 for Wildfire.

Rank	Structure ID	Facility/location	Feature crossed	District	Wildfire priority score
1	041180055702003	SH 152	East Fork Big Creek	Amarillo	24.347
2	172360226701001	PR 40	BIG CHINQUAPIN CREEK	Bryan	22.740
3	040910030901003	SH 70	North Fork of Red River	Amarillo	16.870
4	121700363501021	SH 249 SB FR	Mill Creek Trib. 4	Houston	16.438
5	041910053401002	PR 5	Prairie Dog Frk Red Rv	Amarillo	14.934
6	082170330601001	FM 1835	SALT FORK BRAZOS RIVER	Abilene	14.809
7	121700363501015	SH 249 SB	MILL CREEK	Houston	14.736
8	140110047105045	SH 21	MAYHAW CREEK	Austin	14.653
9	041180037901027	SH 136	Antelope Creek	Amarillo	14.217
10	032430080401016	FM 368	WICHITA RIVER	Wichita Falls	14.037

3.12 Overall Risk Results

The final multi-hazard result contains 21,349 bridge records and ranks bridges by repeated appearance near the top of the included risk lists. Table 3.11 identifies the risk-list contributions to each recurrence score. Equal totals do not imply the same risk profile because different modules can produce the same number of recurrence points.

Table 3.11: Final multi-hazard statewide Top 10 and contributing risk rankings.

Rank	Structure ID	Facility	District	Score	Contributing risk rankings
1	120200058601013	SH 332	Houston	9	Vessel Collision Risk (Top 20), Tornado Risk (Top 20), Hurricane Surge Exposure (Top 20)
2	120850036702023	SH 124	Houston	9	Wildfire Risk (Top 20), Vessel Collision Risk (Top 20), Hurricane Surge Exposure (Top 20)
3	121020027115437	SH 225 WB/IH 610WB	Houston	7	Overload Risk (Top 20), Fatigue Risk (Top 20), Tornado Risk (Top 100)
4	121020002710063	US 90A SB	Houston	7	Flood + Scour Risk (Top 20), Vessel Collision Risk (Top 50), Tornado Risk (Top 50)
5	121020027115377	IH 610	Houston	6	Vessel Collision Risk (Top 20), Tornado Risk (Top 20)
6	121020038912090	SH 146 SB	Houston	6	Vessel Collision Risk (Top 20), Tornado Risk (Top 20)
7	201240030603030	SH 73/SH 87 EB	Beaumont	6	Vessel Collision Risk (Top 20), Tornado Risk (Top 20)
8	201240236701001	SH 82	Beaumont	6	Vessel Collision Risk (Top 20), Tornado Risk (Top 20)
9	120200058701005	FM 1495	Houston	6	Vessel Collision Risk (Top 20), Hurricane Surge Exposure (Top 50), Tornado Risk (Top 100)
10	150150001710080	SP 536 (ALAMO ST)	San Antonio	5	Heavy-Vehicle Bridge Pier Collision (Top 20), Overheight Collision Risk (Top 50)

The two leading bridges have recurrence scores of 9. Structure 120200058601013 in Houston District carries SH 332 across the Intracoastal Canal and appears in the Top 20 for vessel collision, tornado, and hurricane surge exposure. Structure 120850036702023, also in Houston District, carries SH 124 across the Intracoastal Canal and appears in the Top 20 for wildfire, vessel collision, and hurricane surge exposure. Although their totals match, the second bridge adds wildfire exposure while the first is distinguished by its SH 332 corridor context.

Structures ranked 3 and 4 have recurrence scores of 7 but require different follow-up. Structure 121020027115437 combines Top 20 overload and fatigue appearances with a Top 100 tornado appearance, whereas Structure 121020002710063 combines a Top 20 flood/scour appearance with Top 50 vessel-collision and tornado appearances. Ranks 5–9 are led by coastal vessel, tornado, and hurricane combinations. Rank 10, Structure 150150001710080 in San Antonio District, combines heavy-vehicle bridge-pier and overheight collision. These patterns demonstrate why TxDOT should open the contributing risk modules before selecting a response.

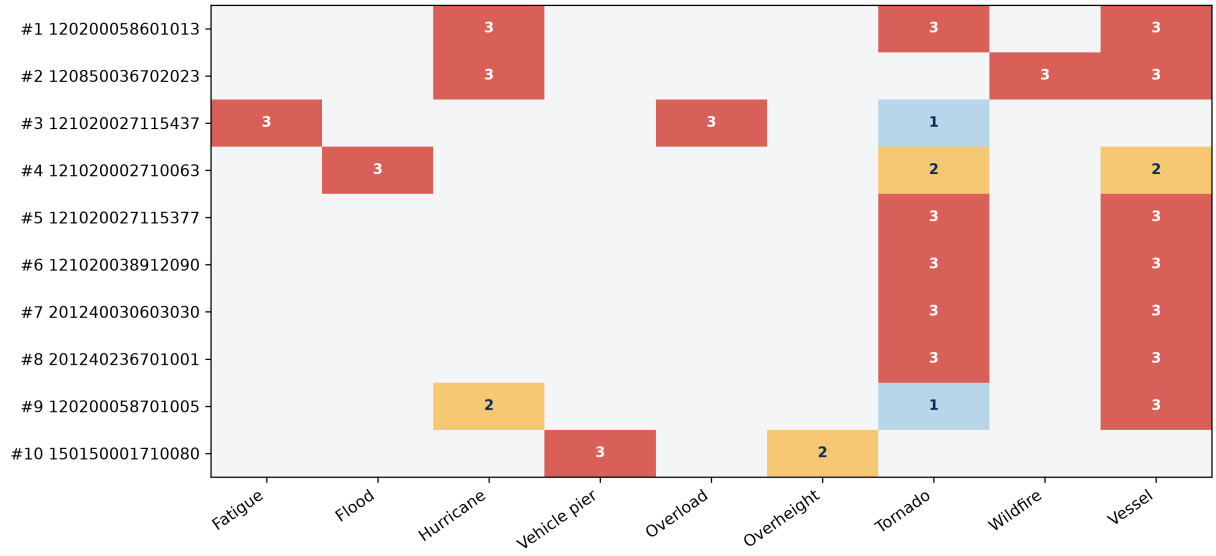


Figure 3.11: Risk-list contributions to the final statewide multi-hazard Top 10. Cell values are rank-band points: 3 for Top 20, 2 for ranks 21–50, and 1 for ranks 51–100.

Figure 3.12 maps the statewide Top 100. Houston District contains 51 bridges, followed by Amarillo with 17 and Fort Worth with 9. Waco contains 4; Dallas, Lufkin, and Odessa contain 3 each; Beaumont and San Antonio contain 2 each; and Abilene, Corpus Christi, Paris, Pharr, Tyler, and Yoakum contain 1 each. Houston leads because its coastal, navigation, industrial, and high-traffic network produces recurrence across several lists; the count reflects repeated eligibility and high placement across modules, not a common raw-risk scale.

Only 12 structure identifiers appear in both the coastal Top 100 population formed by hurricane and vessel collision and the inland Top 100 population formed by flood, tornado, and wildfire. This small overlap confirms that the individual rankings identify distinct hazard populations, whereas the recurrence result highlights the bridges where those populations intersect.

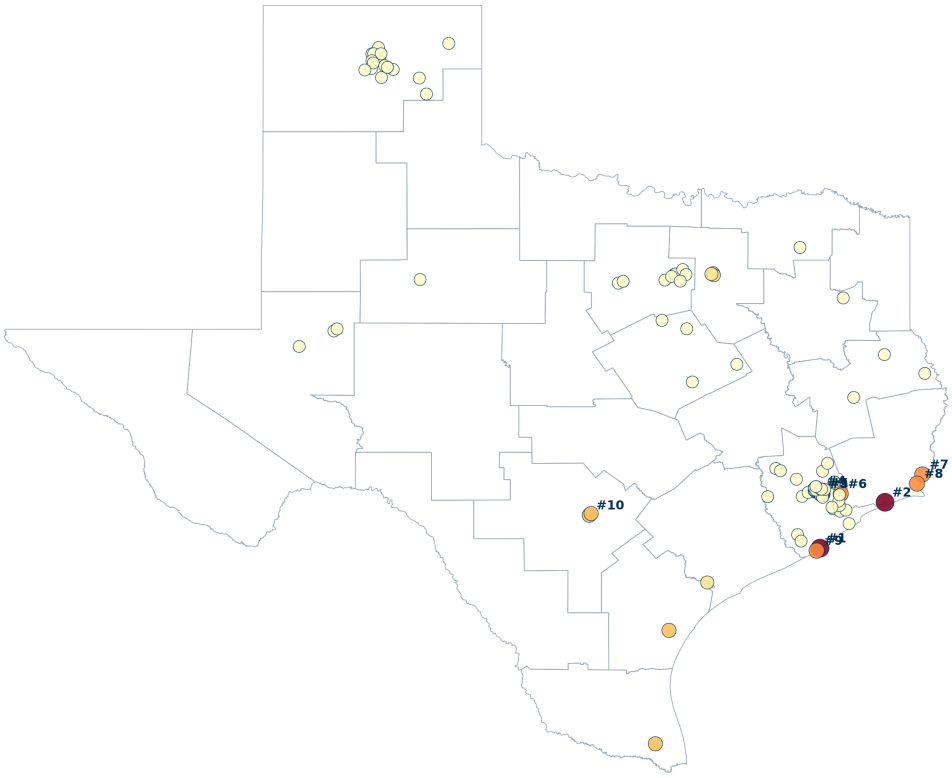


Figure 3.12: Final multi-hazard statewide Top 100. Marker size and color indicate the recurrence score.

Table 3.12: District representation in the final multi-hazard Top 100.

TxDOT district	Number of bridges	Share of Top 100
Houston	51	51%
Amarillo	17	17%
Fort Worth	9	9%
Waco	4	4%
Dallas	3	3%
Lufkin	3	3%
Odessa	3	3%
Beaumont	2	2%
San Antonio	2	2%
Abilene	1	1%
Corpus Christi	1	1%
Paris	1	1%
Pharr	1	1%
Tyler	1	1%
Yoakum	1	1%

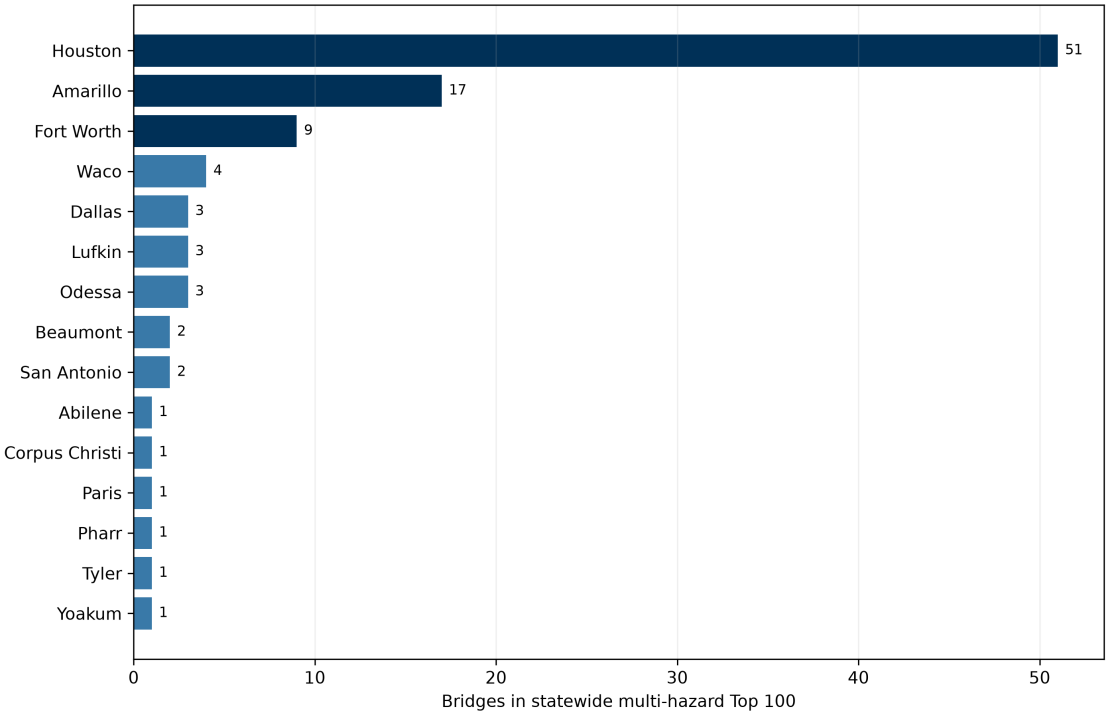


Figure 3.13: TxDOT district representation in the final multi-hazard Top 100.

SECTION 4. Use of Statewide Bridge Risk Rankings

4.1 Purpose and Statewide Application

The statewide rankings translate the final risk-assessment methods and bridge-level results into a consistent framework for priority review. They provide TxDOT with a statewide set of candidate bridges for further evaluation and establish the order in which bridge-level review begins. The subsequent bridge-level evaluation identifies the applicable mitigation, maintenance, inspection, operational, or project response.

TxDOT should retain one centralized statewide methodology for calculating and updating the rankings. The same eligibility rules, data sources, calculations, and ranking definitions should apply to all bridges included in the analysis. This consistency supports comparison across districts, risk categories, and successive updates and prevents calculated priority from varying solely because a bridge is located in a different district.

District staff should use district-filtered subsets of the statewide results to organize local review. These subsets are geographic views of the statewide results, not separate district ranking methods, and should retain each bridge's statewide risk-specific and overall ranks. District staff should compare the statewide inputs with recent inspections, completed work, and current bridge records and correct outdated or mismatched information.

Statewide staff can use the complete results to identify recurring concerns, compare the distribution of ranked bridges across districts, and identify areas requiring additional technical review or data improvement. District staff can use the same results to sequence bridge reviews within their areas. Both applications rely on the same statewide calculations and differ only in geographic scope and review purpose.

4.2 Use of the Rankings

The risk-specific rankings and overall multi-hazard recurrence ranking serve distinct but complementary purposes. Risk-specific rankings identify bridges with the highest relative screening priority for individual risk mechanisms and indicate which bridge information requires review. For example, a high flood ranking directs attention to flood exposure, low-chord information, modeled velocity, and scour-related records, whereas a high fatigue ranking directs attention to traffic loading, structural details, condition, inspection findings, and fatigue-related distress. Risk-specific rankings therefore determine what information should be examined during subsequent technical review.

The overall multi-hazard recurrence ranking identifies bridges that repeatedly appear near the top of multiple risk-specific lists and directs broader review where several risk mechanisms affect the same bridge or corridor. Staff read the recurrence ranking together with the contributing risk categories and their underlying results. The recurrence score measures repeated high placement across the risk-specific lists, while each risk-specific analysis supplies the applicable hazard mechanism and bridge-level calculation.

For statewide planning, the overall ranking identifies bridges warranting broader review, while the risk-specific rankings direct hydraulic, structural, collision, navigation, wildfire, security-related, or other applicable technical evaluations. A bridge ranked highly overall warrants consideration of its contributing risk mechanisms. A bridge ranked highly for a single hazard remains a candidate for prompt risk-specific review independently of its overall recurrence position.

District-filtered subsets allow each district to sequence its review workload without changing the statewide calculation. Retaining the statewide rank allows district staff to distinguish bridges that are leading statewide candidates from bridges that are more prominent within the district subset. Bridge-level review confirms the factor producing the ranking, identifies input-data corrections, determines whether recent work has addressed that factor, and identifies conditions not represented in the statewide data.

Screening priority and final project priority should remain distinct. After reviewing the bridge record and current conditions, staff should consider life-safety and mobility consequences, detour and freight effects, urgency, treatment effectiveness, constructability, cost, project readiness, funding eligibility, and opportunities to coordinate work with existing projects. These considerations determine the priority of an inspection, study, maintenance action, operational change, retrofit, rehabilitation, or capital project. The ranking determines where review begins; current engineering and program information determine the subsequent action.

Statewide programs can use the rankings to identify recurring technical needs, data gaps, and groups of bridges for common evaluation. Districts can use their filtered subsets to sequence bridge reviews and coordinate candidate actions with existing inspection, maintenance, and project-development activities.

4.3 From Ranking to Action

Statewide and district staff can apply the rankings through the six-step closed-loop process shown in Figure 4.1. The process connects the calculated rankings to bridge-record review, identification of the ranking factor, response selection, data updates, and subsequent statewide reassessment.

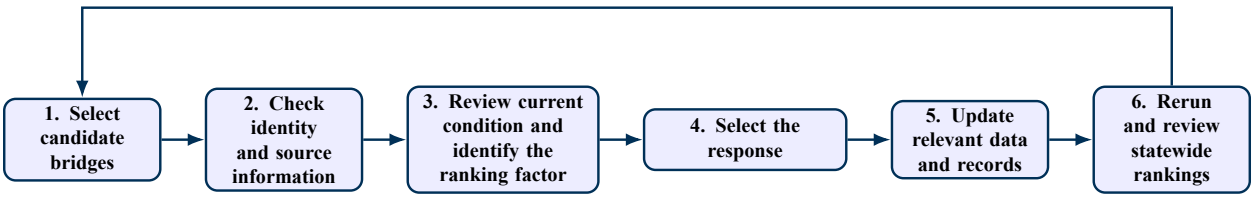


Figure 4.1: Closed-loop use of statewide bridge risk rankings.

1. **Select candidate bridges.** Select candidates from the statewide results, including the overall multi-hazard recurrence ranking, risk-specific rankings, and district-filtered subsets. Candidate information should retain the structure number, statewide rank, district, contributing risk categories, and principal factors driving the ranking.
2. **Check bridge identity and source information.** Match the ranked record to the bridge and location. Review the source information used in the calculation, including applicable inventory, inspection, condition, traffic, event, hydraulic, navigation, rating, and hazard-exposure data. Replace incomplete, outdated, or mismatched inputs with current information.
3. **Review current condition and identify the ranking factor.** Review recent inspections, incident histories, completed repairs, maintenance activities, relevant studies, and site conditions. Identify the condition or exposure that produced the high ranking and determine whether it remains present.
4. **Select the response.** Use the hazard-specific review and response recommendations in Section 5 to select the next action for the observed condition. The response may include continued monitoring, a focused inspection or study, an operational or maintenance action, a protective measure, or consideration

in project programming. If the source data produced an incorrect ranking, correct the data and return the bridge to the data-update pathway.

5. **Update relevant data and records.** Enter corrections and information on completed evaluations or actions into the source data or records used in subsequent assessments. Future rankings will then use current bridge information rather than outdated inputs or conditions already addressed by completed work.
6. **Rerun and review the statewide rankings.** Recalculate the affected rankings during the next statewide update or following a material event or data revision, as appropriate. Review significant changes and use the updated statewide results and district-filtered subsets to begin the next review cycle.

This process closes the loop between statewide screening and subsequent TxDOT action. It maintains a single statewide analytical framework while using bridge-level information and engineering judgment for response selection and future ranking updates.

SECTION 5. Hazard-Specific Review and Response Recommendations

5.1 Introduction

This chapter explains the actions TxDOT can take after a bridge is selected from the statewide rankings. It implements Steps 2–4 of the six-step process in Section 4.3: check bridge identity and source information, review current condition and identify the ranking factor, and select the response. For practical use, those three process steps are organized into the following four review tasks for every risk:

1. check that the ranked record and assigned ranking factors belong to the correct bridge;
2. review current bridge, site, and supporting technical information;
3. identify the bridge condition or exposure that produced the ranking and correct mismatched or outdated source data; and
4. select the applicable response for the observed condition or exposure.

A high ranking identifies where review should begin. The applicable bridge-specific inspection, analysis, maintenance, operational, design, or project-development process determines the response. Table 5.1 summarizes the records to review and the available response paths; the following sections present the four steps for each of the ten final risk categories.

Table 5.1: Quick reference from ranked risk to bridge-level review and response.

Risk	Check first	Main response paths
Explosive/security criticality	Bridge importance, access, structural configuration, and existing protection	Controlled security review, access control, monitoring, physical protection, and emergency planning
Fatigue	Steel details, truck loading, condition, cracks, and repair history	Focused inspection, nondestructive evaluation, monitoring, repair, strengthening, or rehabilitation
Flood overtopping and scour	Low chord, flood level, velocity, scour, foundations, and channel condition	Hydraulic review, underwater inspection, monitoring, scour protection, operational action, or bridge improvement
Hurricane surge exposure	Bridge and approach elevations, surge exposure, erosion, and prior storm effects	Coastal analysis, post-storm inspection, operational planning, protection, strengthening, raising, or reconstruction
Heavy-vehicle bridge pier collision	Recorded support impacts, crash assignment, bent/pier setback, truck traffic, roadway geometry, and existing protection	Damage inspection, roadway or traffic modification, TxDOT-compliant load redirection, structural resistance, relocation, or reconstruction

Risk	Check first	Main response paths
Overload	Current load rating, bridge condition, truck loads, posting, and route function	Rating update, enforcement, posting or routing, monitoring, strengthening, rehabilitation, or replacement
Overheight collision	Measured clearance, under-road match, truck exposure, and accepted strike history	Signing, routing, detection, protection, geometric change, or structural repair
Tornado	Hazard location, bridge type, condition, vulnerable components, and network function	Inspection readiness, post-event inspection, traffic control, component protection, or project-level improvement
Vessel collision	Navigable-waterway eligibility, vessel tracks, channel and pier geometry, and protection	Bridge-specific collision analysis, navigation measures, protection systems, strengthening, retrofit, or replacement
Wildfire	Wildfire hazard, current fuels, bridge materials, access, and site condition	Fuel management, access improvement, monitoring, operational planning, component protection, repair, or rehabilitation

5.2 Explosive/Security Criticality

1. **Step 1–Check the ranked record.** Match the structure number to the physical site and review the assigned traffic and detour importance, emergency or evacuation function, access indicators, and structural configuration through the applicable controlled TxDOT process.
2. **Step 2–Review current conditions.** Review access near critical components, parking or staging opportunities, barriers, pier protection, lighting, monitoring, recent construction, and changes in bridge or site operations.
3. **Step 3–Identify the ranking factor.** Identify the current bridge importance, access, vulnerability, and protection conditions that produced the ranking. Correct incorrect, outdated, or incomplete source data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.1 to select the bridge-specific response.

Figure 5.1 converts the explosive/security review into a controlled decision path. The tree uses the threat scenario, standoff distance, detection and response time, and structural performance objective established through the applicable bridge-specific security assessment [3, 191].

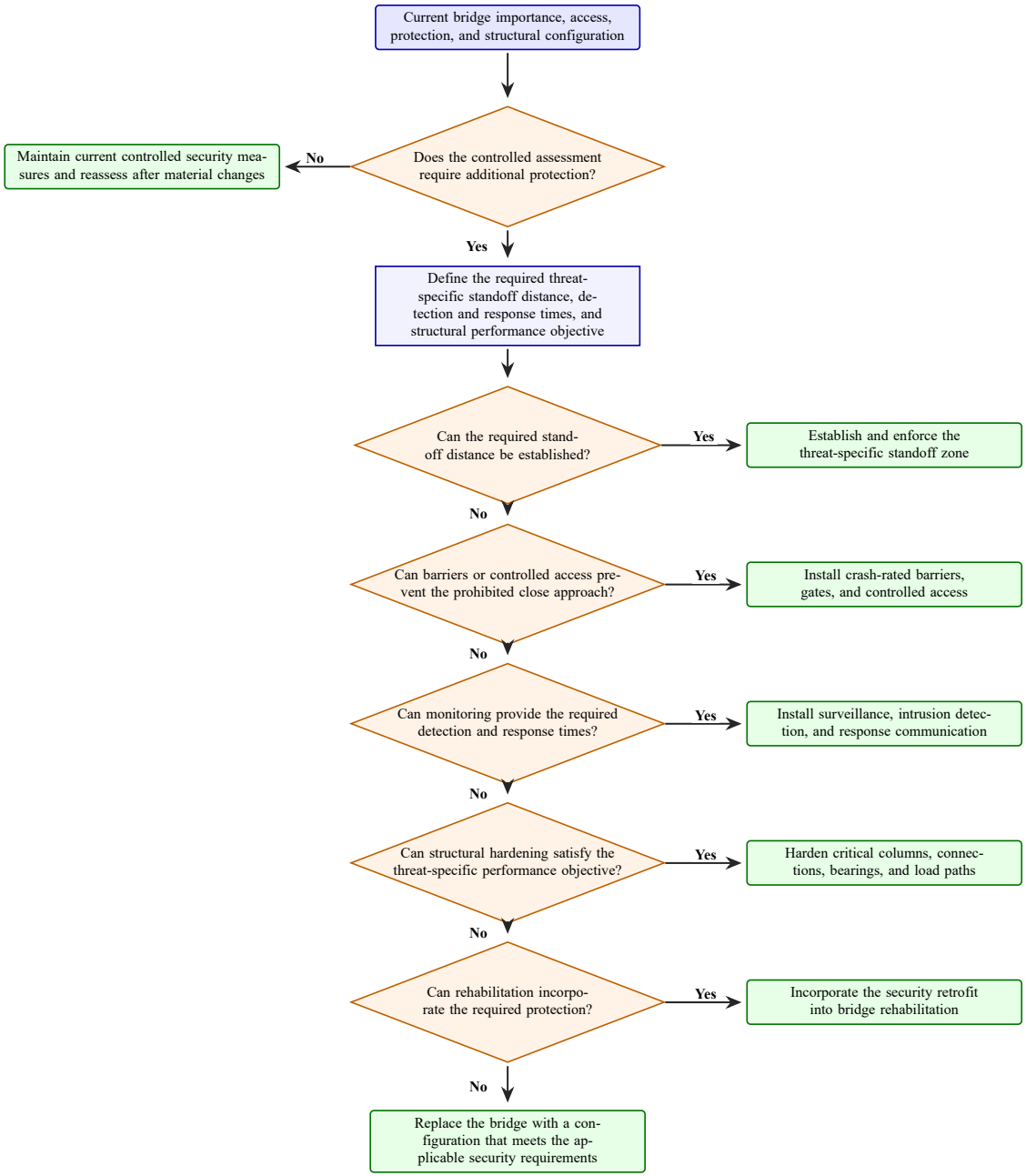


Figure Note. Required standoff, detection time, response time, and structural performance are bridge- and threat-specific values established in the controlled security assessment; they are not taken from the statewide screening score.

Figure 5.1: Decision tree for selecting the primary TxDOT response to explosive/security criticality.

5.3 Fatigue

1. **Step 1–Check the ranked record.** Check that the bridge has an eligible steel superstructure and that the truck demand, governing span, condition information, and element data were assigned to the correct structure.
2. **Step 2–Review current conditions.** Review fatigue-prone details, nonredundant steel tension members (NSTMs) where applicable, connections, welded attachments, crack maps, inspection photographs, nondestructive-evaluation results, prior cracks, repairs, and current truck loading.
3. **Step 3–Identify the ranking factor.** Identify susceptible details exposed to substantial cyclic loading, relevant distress, or a bridge-specific fatigue-life result that requires action. Correct incorrect or outdated material, traffic, element, or repair data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.2 to select the bridge-specific response.

Figure 5.2 selects a primary fatigue response after inspection identifies the fatigue-prone detail and the bridge-specific evaluation establishes the remaining fatigue life. Immediate closure, restriction, or temporary support is applied before this selection when fracture consequence or crack condition requires it [192, 193].

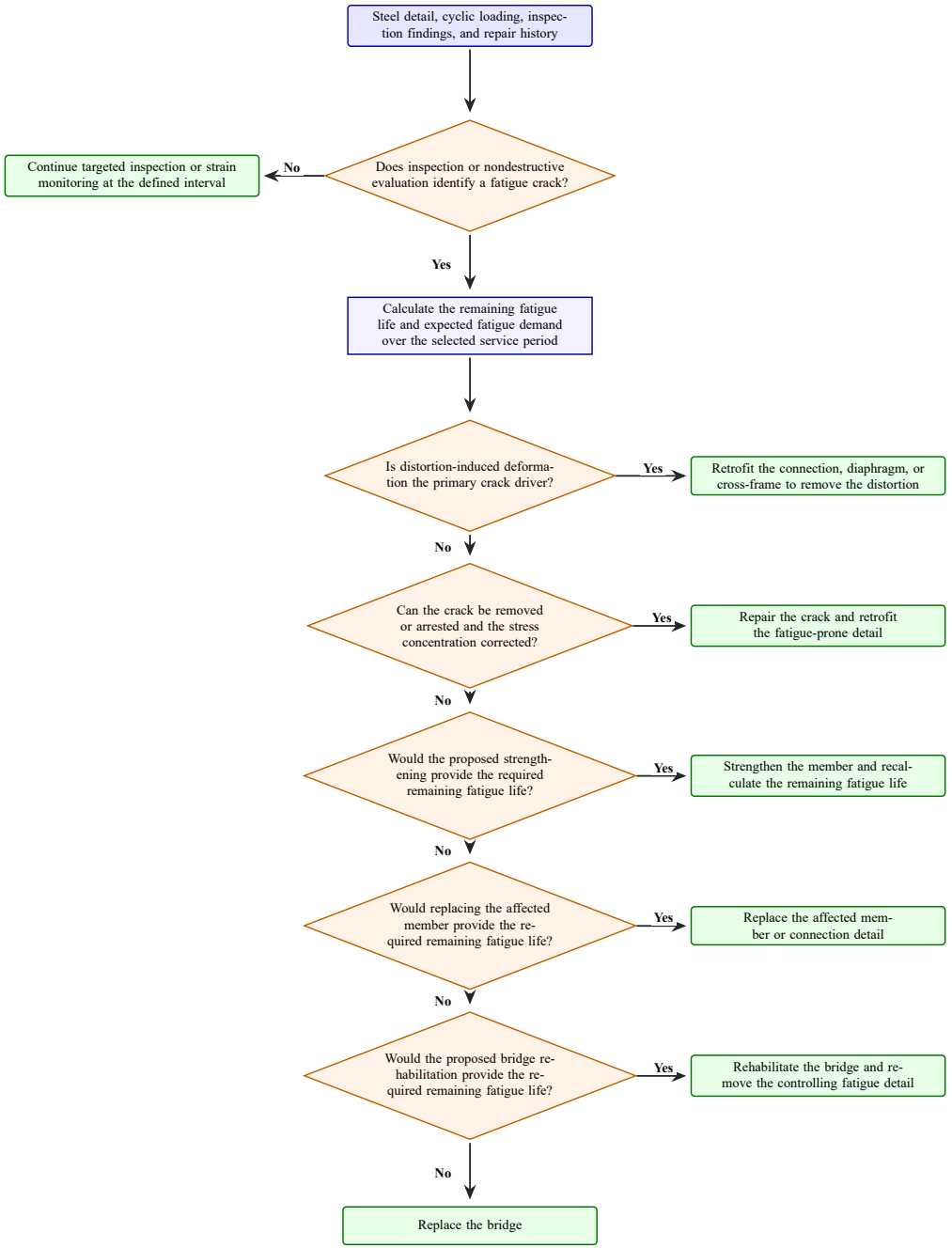


Figure Note. For each candidate repair or retrofit, recalculate the expected remaining fatigue cycles and compare them with the cycles expected during the selected service period under the bridge-specific truck stress-range spectrum.

Figure 5.2: Decision tree for selecting the primary TxDOT response to fatigue risk.

5.4 Flood Overtopping and Scour

1. **Step 1–Check the ranked record.** Match the bridge to the waterway and review the assigned low-chord elevation, mapped flood levels, stream reach, modeled velocity, and match quality.
2. **Step 2–Review current conditions.** Review hydraulic studies, scour appraisal, foundation information, Scour Plan of Action, underwater inspection, channel and bank condition, debris, approach erosion, recent floods, closures, and completed countermeasures.
3. **Step 3–Identify the ranking factor.** Identify overtopping when bridge-specific information shows water reaching the low chord, deck, or vulnerable approaches. Identify scour when hydraulic exposure coincides with vulnerable foundations, observed scour, channel instability, debris, or an active plan-of-action concern. Correct mismatched or outdated elevation, stream-reach, structure, or hydraulic inputs and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.3 to select the bridge-specific response.

Figure 5.3 separates overtopping, scour, channel-instability, and debris responses using bridge-specific elevations and foundation embedment. The hydraulic variables are obtained from the survey and hydrologic, hydraulic, and scour analyses described in the preceding steps [196, 197].

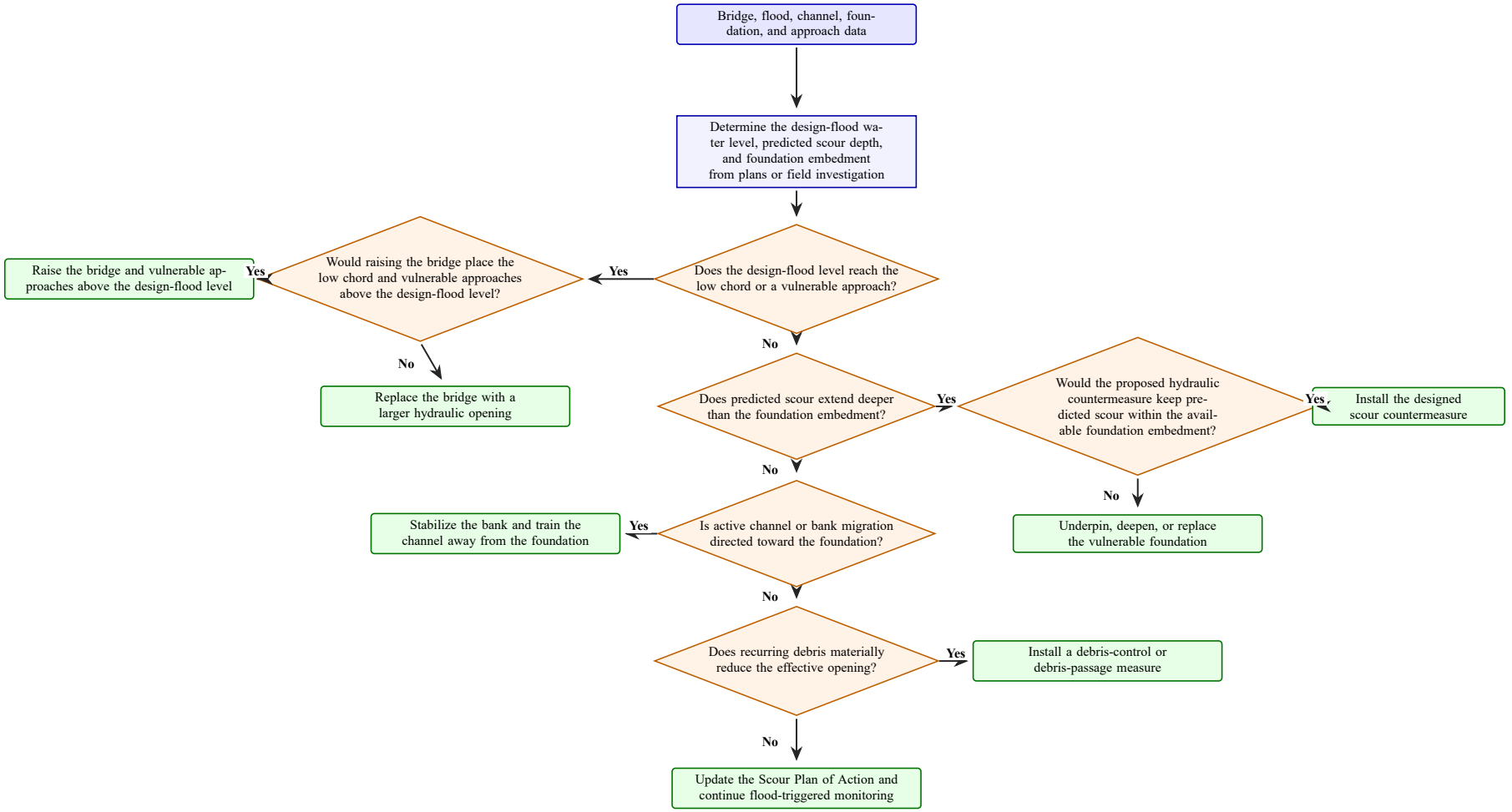


Figure Note. Use the bridge-specific design-flood water level, controlling low-chord elevation, lowest vulnerable approach elevation, predicted total scour depth, and foundation embedment below the current bed obtained from plans or field investigation. Operational warnings and closure follow the observable triggers in the Scour Plan of Action.

Figure 5.3: Decision tree for selecting the primary TxDOT response to flood-overtopping and scour risk.

5.5 Hurricane Surge Exposure

1. **Step 1–Check the ranked record.** Match the bridge to its coastal location and review the low-chord source, bridge and approach elevations, surge exposure, and available coastal hydraulic or wave analysis.
2. **Step 2–Review current conditions.** Review shoreline and channel conditions, erosion and scour, approach vulnerability, wave and debris exposure, previous storm damage, closure history, evacuation function, and completed protection or reconstruction.
3. **Step 3–Identify the ranking factor.** Identify surge exposure when current elevation and coastal-hazard information shows that surge can reach the bridge or its approaches. Treat wave, debris, erosion, and scour as bridge-specific consequences or secondary factors. If the bridge location, surge cell, or elevation is incorrect or outdated, correct the source data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.4 to select the bridge-specific response.

Figure 5.4 uses the bridge-specific surge-and-wave elevation and the calculated wave-force demand to select the primary coastal response. When the analysis identifies more than one deficiency, TxDOT applies the corresponding measures together rather than treating the terminal boxes as mutually exclusive [198, 199].

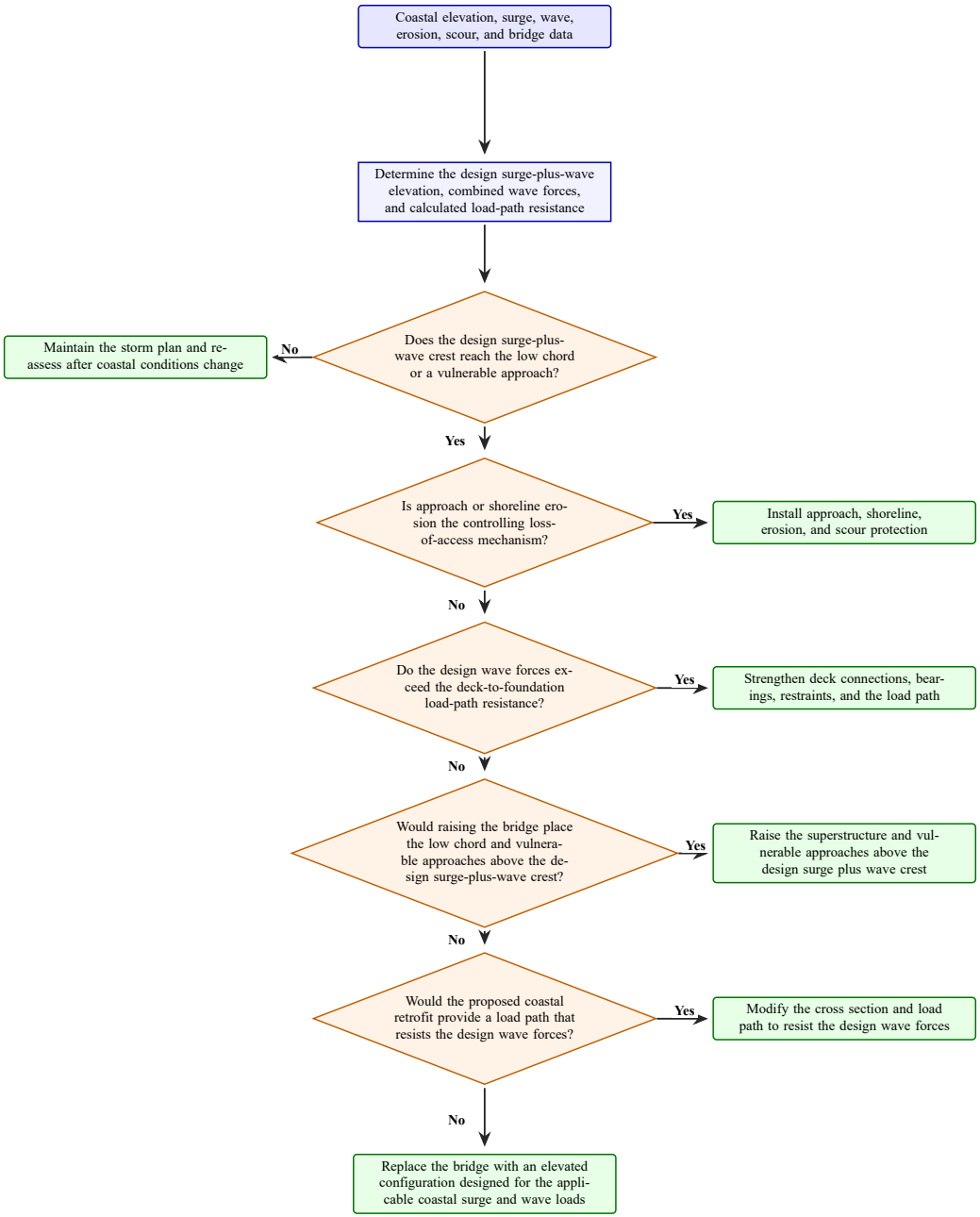


Figure Note. Use the bridge-specific design surge plus the applicable wave crest, the bridge low-chord elevation, the lowest vulnerable approach elevation, the calculated uplift and lateral wave forces, and the calculated resistance of the complete deck-to-foundation load path.

Figure 5.4: Decision tree for selecting the primary TxDOT response to hurricane-surge exposure.

5.6 Heavy-Vehicle Bridge Pier Collision

1. **Step 1–Check the ranked record.** Review the contributing crash records and retain events in which a heavy vehicle physically struck a pier or bridge support at the bridge site.
2. **Step 2–Review current conditions.** Measure the shortest setback in the final configuration from each roadway edge to the bent or pier. Review adjacent bridge decks, roadway type and alignment, shoulders and medians, ramps and intersections, one-direction truck traffic, prior impacts and repairs, and the type, placement, foundation, height, test level, and dynamic displacement of existing protection.
3. **Step 3–Apply the TxDOT collision-design screening.** Investigate a bent or pier when it is within 30.0 ft of a roadway edge; a bridge deck adjacent to a column is treated as an adjacent roadway. For a bent or pier within that distance, calculate the annual frequency of a heavy-vehicle impact using the TxDOT procedure, current one-direction truck traffic, and roadway geometry. An annual frequency below 0.001 per year completes the screening without a vehicular-collision design requirement. The current annual-frequency calculation governs the design screening, and a recorded prior impact separately triggers damage inspection and incident response. If the ranked crash records are duplicated, unrelated, or assigned to the wrong bridge, correct the source data and return the bridge to the update and reranking process in Section 4.3 [214].
4. **Step 4–Select the response.** Use the decision tree in Figure 5.5 to select the bridge-specific response.

Figure 5.5 selects the long-term response using the measured bent/pier setback, the calculated annual heavy-vehicle impact frequency, and the load-redirection or structural-resistance design checks. Damage inspection, stabilization, and traffic control occur immediately after an impact and precede completion of this long-term selection process [202, 203, 214].

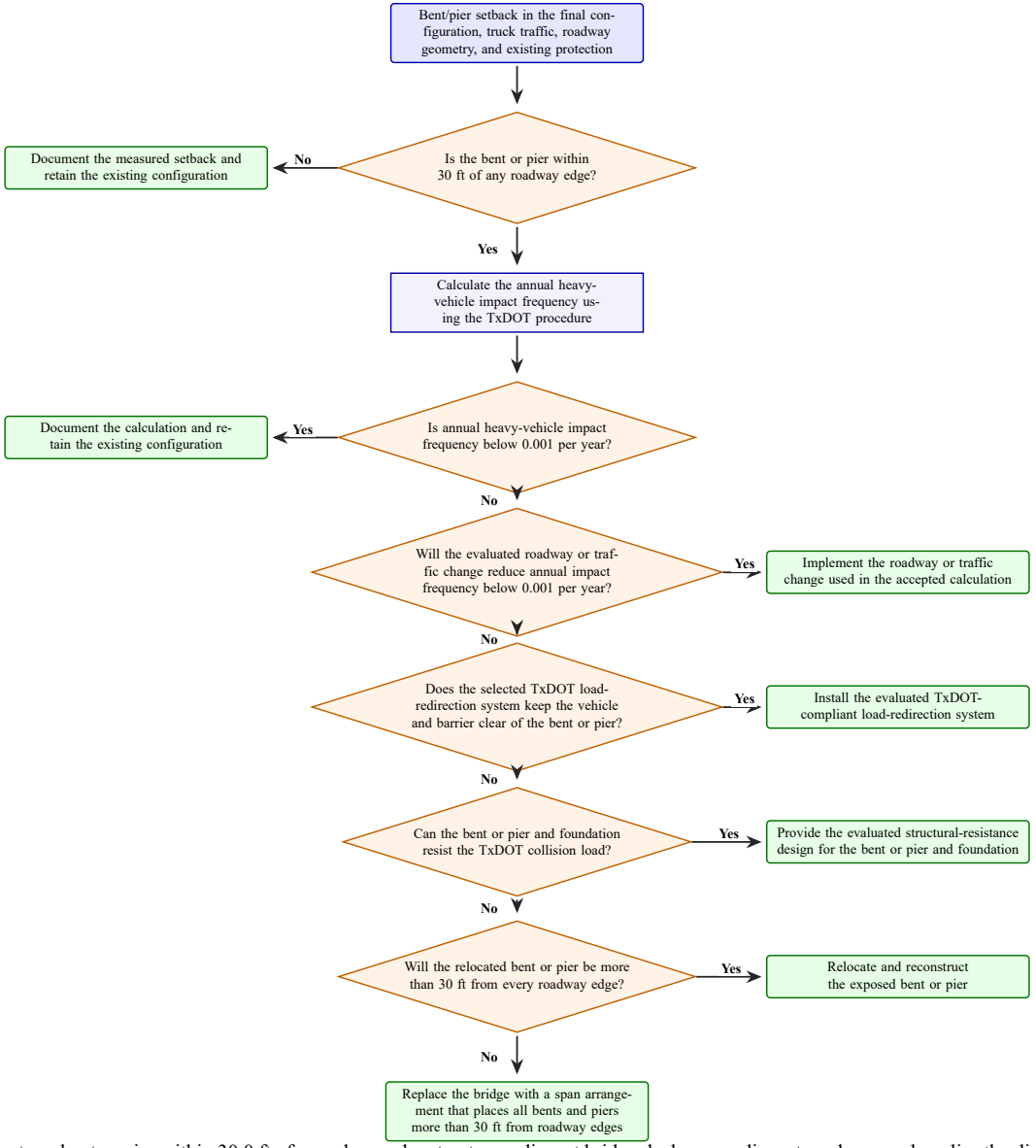


Figure Note 1. TxDOT investigates a bent or pier within 30.0 ft of a roadway edge, treats an adjacent bridge deck as an adjacent roadway, and applies the distance to the final configuration [214]. *Figure Note 2.* For those bents or piers, calculate annual heavy-vehicle impact frequency from current one-direction truck traffic and roadway characteristics. A result below 0.001 per year completes the screening without collision design; a result at or above 0.001 per year requires TxDOT-compliant load redirection or structural resistance [214].

Figure 5.5: Decision tree for selecting the primary TxDOT response to heavy-vehicle bridge-pier collision exposure.

5.7 Overload

1. **Step 1–Check the ranked record.** Check that the assigned load rating, controlling member and limit state, span and bridge configuration, posting, route function, truck profile, and WIM station belong to the correct bridge and analysis record.
2. **Step 2–Review current conditions.** Review the current load rating and bridge condition, repairs or strengthening, truck volume and axle-load information, permit and overweight activity, enforcement information, freight operations, and whether the assigned traffic station remains representative.
3. **Step 3–Identify the ranking factor.** Compare the current load rating with truck-load information and identify exposure above the screening threshold. Correct incorrect or outdated rating, span, traffic-station, posting, or route data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.6 to select the bridge-specific response.

Figure 5.6 uses the current bridge load rating to select operational or structural responses. A rating factor of 1.0 or greater at the applicable legal or permit load-rating level indicates sufficient calculated capacity for the evaluated vehicle and conditions [204].

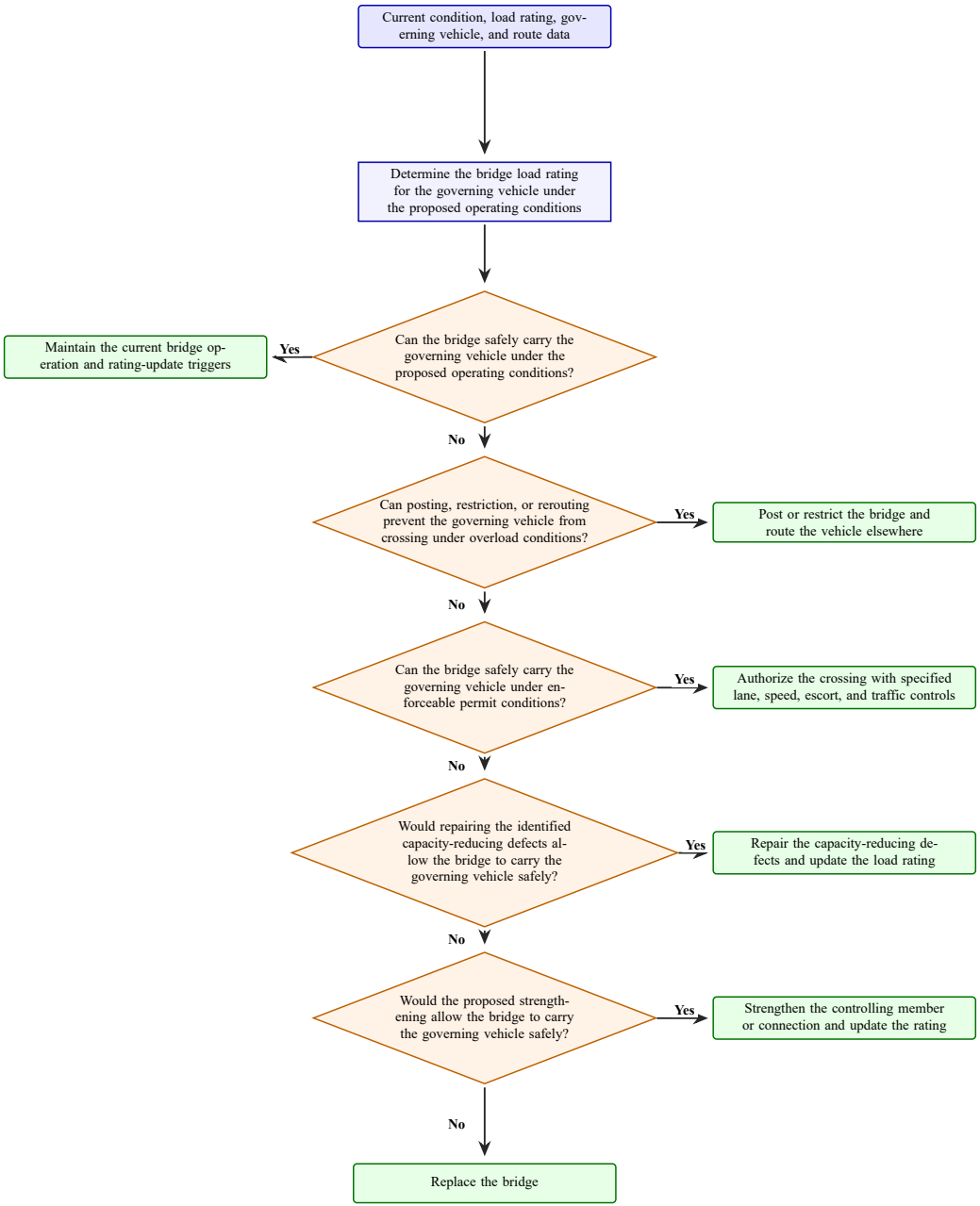


Figure Note. Perform the load-rating analysis for the bridge in its current inspected condition, the governing legal or permit vehicle, and the applicable rating level. Reanalyze the bridge after applying candidate permit controls, repairs, or strengthening.

Figure 5.6: Decision tree for selecting the primary TxDOT response to overload risk.

5.8 Overheight Collision

1. **Step 1–Check the ranked record.** Match the bridge to the road below and review the assigned clearance, measurement date, truck demand, roadway geometry, and accepted strike records.
2. **Step 2–Review current conditions.** Remeasure the controlling clearance when needed and review lane and shoulder clearances, resurfacing or construction, route restrictions, signs, detection or warning systems, impact locations, structural damage, repairs, and district strike information.
3. **Step 3–Identify the ranking factor.** Compare the current measured minimum clearance with the maximum vehicle-and-load height plus the operational clearance allowance required by the applicable TxDOT oversize-permit and routing procedure. Retain correctly assigned strike records. If the clearance, roadway match, event assignment, or geometry data are incorrect or outdated, correct the source data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.7 to select the bridge-specific response.

Figure 5.7 compares the maximum vehicle-and-load height with the measured controlling clearance and then tests progressively more intensive prevention and protection measures [205, 206].

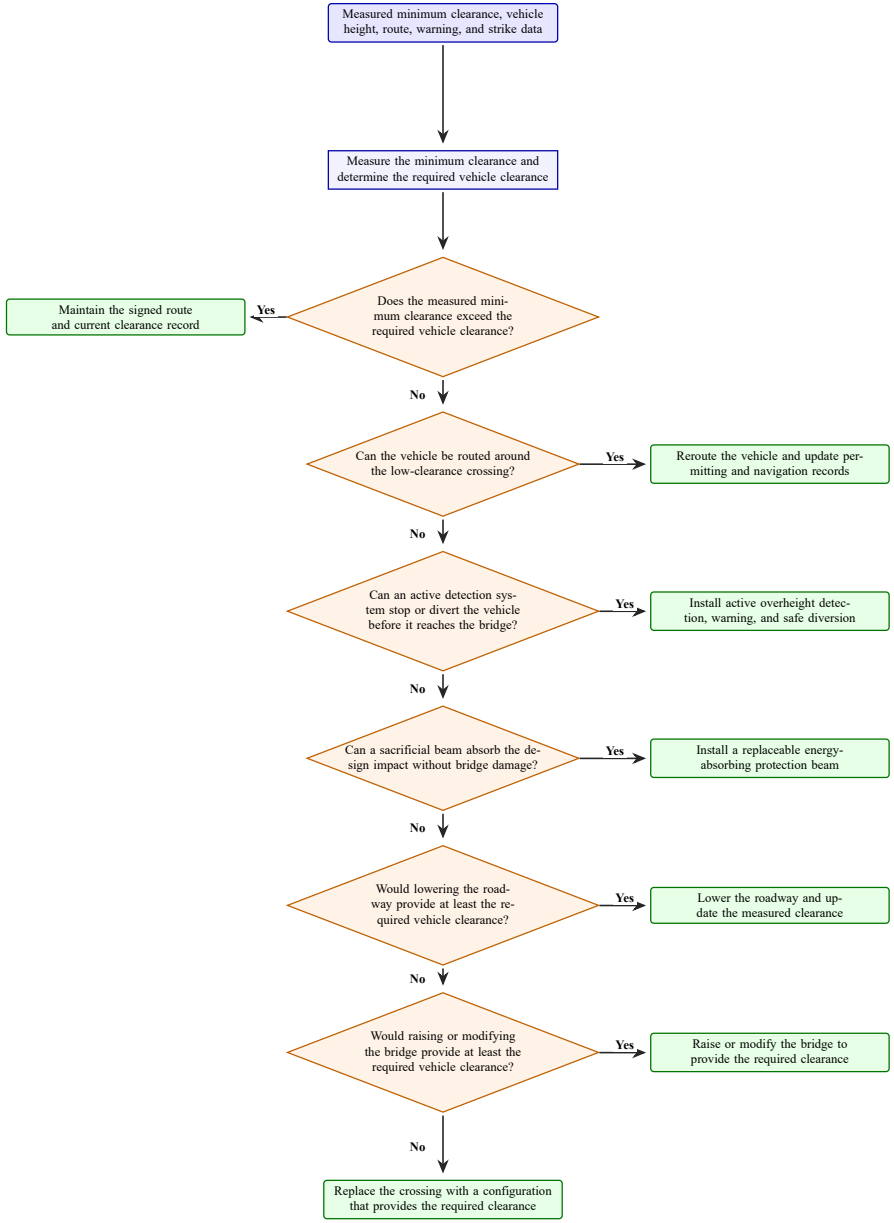


Figure Note. Required vehicle clearance is the maximum vehicle-and-load height plus the operational clearance allowance required by the applicable TxDOT oversize-permit and routing procedure. Compare it with the current measured minimum clearance along the proposed vehicle path, and repeat the measurements after roadway or bridge modification.

Figure 5.7: Decision tree for selecting the primary TxDOT response to overheight-collision risk.

5.9 Tornado

1. **Step 1–Check the ranked record.** Review the bridge-to-hazard-area assignment, bridge type and material, span, assigned clearance information, and emergency-route function.
2. **Step 2–Review current conditions.** Review current condition, bearings, joints, connections, attachments, signs, utilities, components vulnerable to windborne debris or displacement, prior severe-weather damage, inspection access, and nearby bridges that can be grouped into post-event routes.
3. **Step 3–Identify the ranking factor.** Identify the bridge components and conditions associated with the assigned tornado-hazard area. Correct incorrect or outdated geographic assignments, structure classifications, clearance records, or condition data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.8 to select the bridge-specific response.

Figure 5.8 separates immediate post-event action from pre-event component, construction-stage, and aerodynamic measures. Site-specific wind demand and bridge-specific resistance are used rather than a single statewide wind value [209, 210].

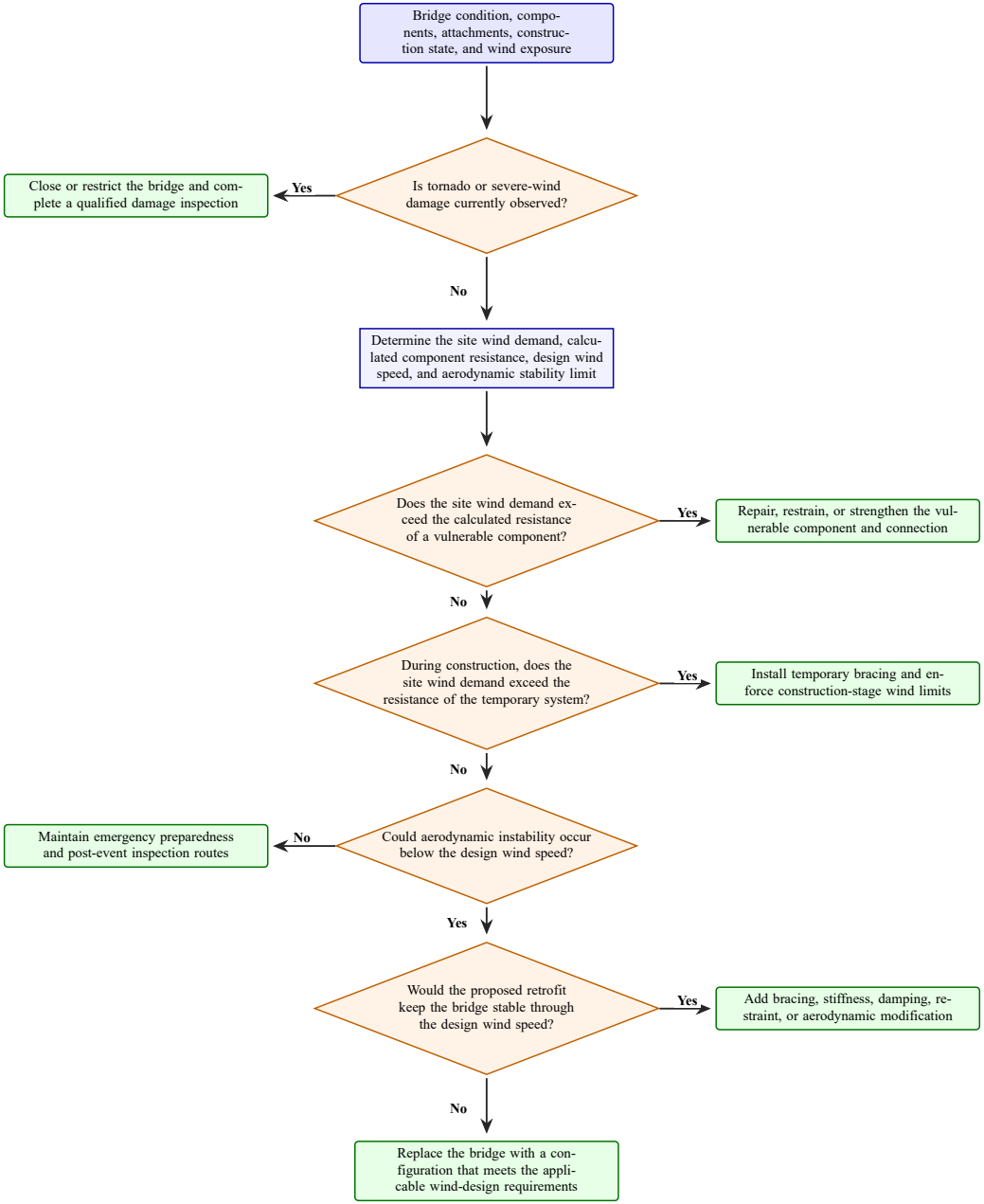


Figure Note. Compare the site-specific wind demand with the calculated resistance of the component or temporary system. Also compare the applicable design wind speed with the calculated wind speed at which divergence, galloping, or flutter becomes unstable.

Figure 5.8: Decision tree for selecting the primary TxDOT response to tornado and severe-wind risk.

5.10 Vessel Collision

1. **Step 1–Check the ranked record.** Check that the bridge crosses an active navigable waterway, that parallel structures are grouped correctly, and that the assigned vessel passages and navigation geometry belong to the correct crossing.
2. **Step 2–Review current conditions.** Review vessel classes and tracks, channel alignment, navigation span, pier location, water depth and current, bridge and roadway demand, fenders, dolphins or other protection, navigation aids, prior allisions, and changes in vessel or channel conditions.
3. **Step 3–Identify the ranking factor.** Determine whether the design vessel path or recorded AIS tracks can reach an exposed pier, support, or span under the current channel geometry and water conditions. If the passage section, structure grouping, navigation exposure, geometry, or protection data are incorrect or outdated, correct the source data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.9 to select the bridge-specific response.

Figure 5.9 translates the vessel-collision review into a continuous TxDOT decision process. The statewide ranking identifies bridges for priority review, and the bridge-specific AASHTO Method II calculation determines the annual frequency of collapse, denoted by AF. TxDOT recalculates AF for each candidate response, and a response reaches a terminal box when the recalculated AF meets the numerical target defined in Figure Notes 1 and 2. Collision detection, motorist warning, emergency traffic control, incident response, and post-collision inspection proceed concurrently when the current exposure requires immediate operational control.

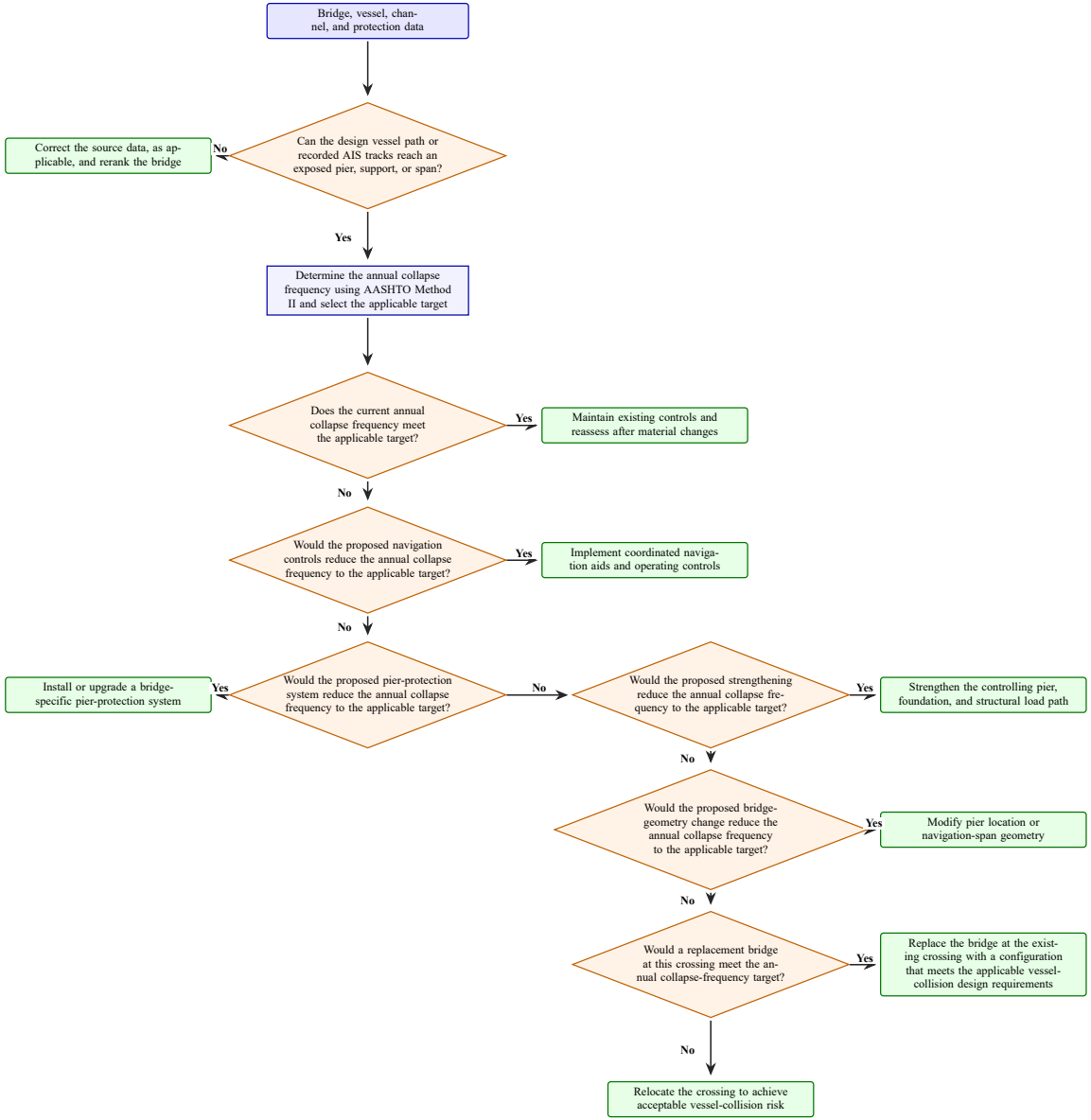


Figure Note 1. For a critical or essential bridge, the target is no more than one expected collapse per 10,000 bridge-years. The AASHTO classification applies to a bridge that serves as an important link in the Strategic Highway Network [211].
Figure Note 2. For a typical bridge, the target is no more than one expected collapse per 1,000 bridge-years [211].
Figure Note 3. The annual collapse frequency is the total frequency of bridge collapse from vessel collision, summed for vulnerable piers and inbound and outbound vessel traffic. Evaluate each candidate response by recalculating this frequency with the response included.

Figure 5.9: Decision tree for selecting the primary TxDOT response to vessel-collision risk.

5.11 Wildfire

1. **Step 1–Check the ranked record.** Review the bridge footprint, assigned wildfire-hazard information, imagery date, fuel classification, bridge and deck materials, and emergency or evacuation function.
2. **Step 2–Review current conditions.** Inspect vegetation and fuel continuity, timber or heat-sensitive components, combustible debris or storage, utilities, drainage features, maintenance history, recent fuel treatment, firefighting and inspection access, water availability, and recent fire history.
3. **Step 3–Identify the ranking factor.** Identify continuous fuels that can transmit heat or flame to vulnerable bridge components in the mapped wildfire-hazard area. Correct incorrect or outdated imagery, fuel, bridge-footprint, or material data and return the bridge to the update and reranking process in Section 4.3.
4. **Step 4–Select the response.** Use the decision tree in Figure 5.10 to select the bridge-specific response.

Figure 5.10 separates post-fire structural action from pre-fire fuel, access, and component-protection measures. Heat exposure and residual resistance are evaluated for the actual bridge materials and fire scenario rather than assigned from the statewide screening rank.

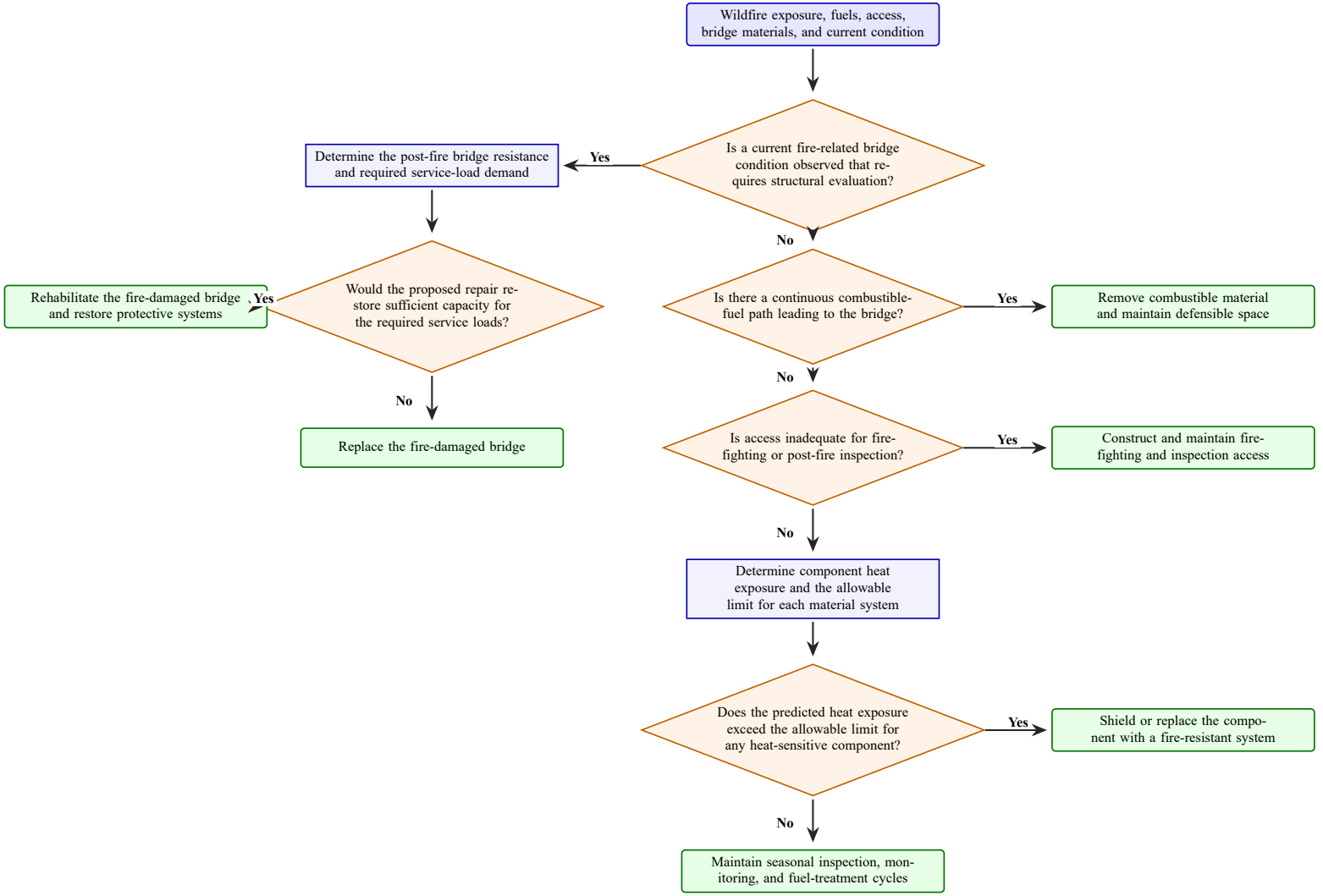


Figure Note. Compare the calculated structural resistance after the proposed repair with the required service-load demand. Also compare the bridge-specific heat exposure from the selected wildfire scenario with the allowable exposure for each component and material system being evaluated. Apply immediate closure and damage inspection before reopening after significant exposure.

Figure 5.10: Decision tree for selecting the primary TxDOT response to wildfire risk.

SECTION 6. Bridge Risk Screening GIS Tool

6.1 Introduction

The Bridge Risk Screening GIS tool converts the statewide rankings into an interactive map for bridge-level review. The risk calculations and statewide results presented in Sections 2 and 3 provide the analytical basis, while the GIS tool provides an interface for selecting a risk, narrowing the displayed bridges, viewing supporting map layers, and examining individual bridge records. The tool also supports the review and response process in Sections 4 and 5 by allowing users to move from a statewide or district-level priority list to information associated with a specific bridge.

The interface includes Multi-Hazard Screening and risk-specific views for fatigue, flood, hurricane surge exposure, heavy-vehicle bridge pier collision, overload, overheight collision, tornado, vessel collision, and wildfire. Each view uses the final results prepared for the corresponding risk and displays the applicable ranking fields, explanatory text, map layers, legends, and bridge popups.

6.2 Local Operation and Interface Layout

The delivered tool runs directly from the project folder without requiring software installation or a web service. The user opens `OPEN_WEBSITE.html`, which loads the bundled application code, mapping libraries, risk data, TxDOT district boundaries, legends, and static bridge imagery from the accompanying `website` folder. The opening page is therefore available on an internal computer without an internet connection or password.

The interface has four principal areas. The control panel on the left contains the risk, district, priority-group, bridge-display, and map-layer controls. The map occupies the center of the screen and displays TxDOT district boundaries, ranked bridge locations, and selected supporting layers. The legend in the upper-right corner explains the colors and symbols used in the active view. Selecting a bridge opens a detailed information panel above the map. Figure 6.1 shows the Multi-Hazard Top 10 view and the primary controls.

6.3 Risk, District, and Priority Selection

The **Risk type** menu changes the active analysis. Selecting a new risk refreshes the displayed bridge records, ranking field, legend, explanation, and risk-specific layers. Multi-Hazard Screening identifies bridges that recur near the top of multiple adopted risk rankings. The risk-specific selections provide direct access to the individual statewide results.

The **TxDOT district** menu limits the active view to one of the 25 TxDOT districts or retains the statewide display through the **All districts** option. District filtering allows a district office to examine its bridges while preserving the statewide and district ranking information used in the report.

The **Ranked bridges** menu controls the displayed ranking range. Users may display the Top 10, Top 50, Top 100, or all bridges available for the selected risk and district. Rank labels identify the positions of the highest-ranked bridges. The **Show bridge locations** control turns the ranked bridge points on or off, allowing supporting hazard or exposure layers to be viewed without the bridge symbols.

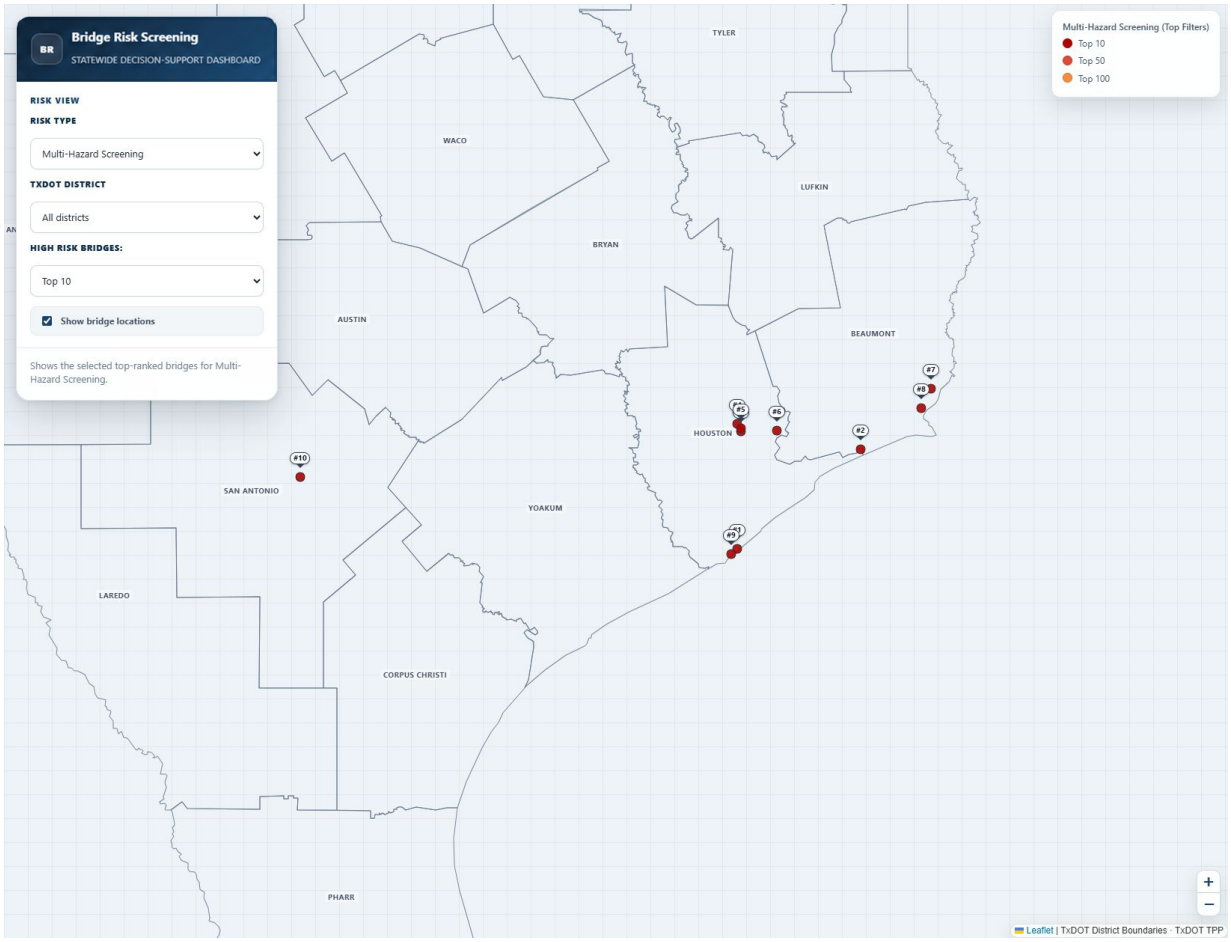


Figure 6.1: Bridge Risk Screening GIS tool showing the Multi-Hazard Top 10 view, TxDOT district boundaries, bridge ranks, filters, and legend.

Table 6.1 summarizes the primary controls and the map response produced by each selection.

Table 6.1: Primary GIS tool controls and displayed results.

Control	Available selections	Map response
Risk type	Multi-Hazard Screening and risk-specific views	Loads the selected ranking, explanatory text, legend, and applicable supporting layers
TxDOT district	All districts or one of 25 districts	Limits the displayed bridges to the selected district and reports district-level ranking information
Ranked bridges	Top 10, Top 50, Top 100, or all bridges	Changes the number of ranked bridge records displayed on the map
Show bridge locations	On or off	Shows or hides the ranked bridge symbols and rank labels
Map layers	Options defined for the selected risk	Adds or removes the selected hazard, exposure, incident, traffic, waterway, or reference layer
Bridge selection	Click a displayed bridge point	Opens bridge identification, location, ranking, risk-specific fields, and bundled imagery

6.4 Risk-Specific Map Layers and Legends

Each risk view provides map layers that help explain the bridge ranking. The available layers correspond to the data used to characterize the selected hazard or exposure. Examples include historical fatigue cases; 10-, 100-, and 500-year flood depths; statewide waterways; National Water Model maximum velocity; hurricane occurrence probabilities, SLOSH surge depths, and bridge low-chord clearance; confirmed heavy-vehicle impacts on bridge supports; weigh-in-motion and vehicle-classification stations; annual truck-volume thresholds; bridge operating ratings; predicted and observed overheight strikes; FEMA tornado frequency; AIS vessel flow and modeled surface currents; previous wildfire road-closure records; and the Wildfire Hazard Potential index.

Layer controls are displayed only when they apply to the selected risk. When a layer is selected, the map displays its symbols, lines, or raster surface beneath the ranked bridge points and TxDOT district reference lines. The legend updates with the active layer, and the explanation at the bottom of the control panel describes what the layer represents. Figure 6.2 illustrates the wildfire view with the Wildfire Hazard Potential layer, road-closure-record controls, ranked bridges, district boundaries, and the associated legend.

6.5 Bridge-Level Information and Static Imagery

Clicking a ranked bridge point opens a bridge-level information panel. Common fields include the bridge structure number, facility or location, statewide rank, district rank, and coordinates. Additional rows present the values used to interpret the selected risk, such as flood overtopping and velocity ranks, traffic and load measures, collision counts, clearance, storm-surge exposure, vessel-flow variables, or wildfire hazard and

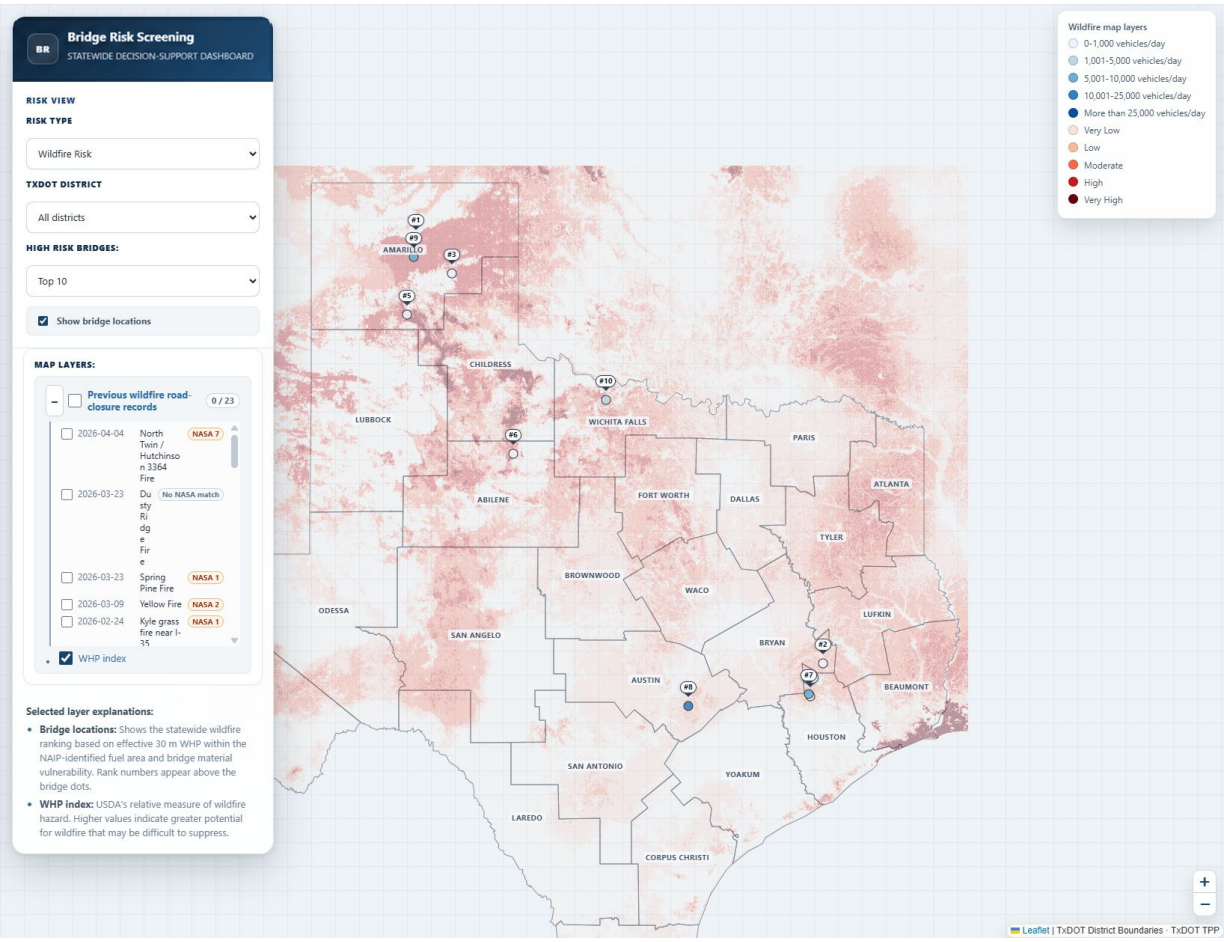


Figure 6.2: Wildfire risk view with the Wildfire Hazard Potential layer, road-closure-record controls, ranked bridges, TxDOT district boundaries, and the active legend.

vulnerability information.

Static bridge imagery is bundled for the statewide Top 10 bridges in each risk view. When imagery is available for the selected bridge, the popup displays a roadway-level or bridge-context image and a satellite image side by side. Each image includes a title, source, and coordinate information and can be opened at full size from the local package. The popup also provides optional links to interactive online views. Figure 6.3 shows a fatigue-risk bridge popup containing the common bridge fields and both bundled image types.

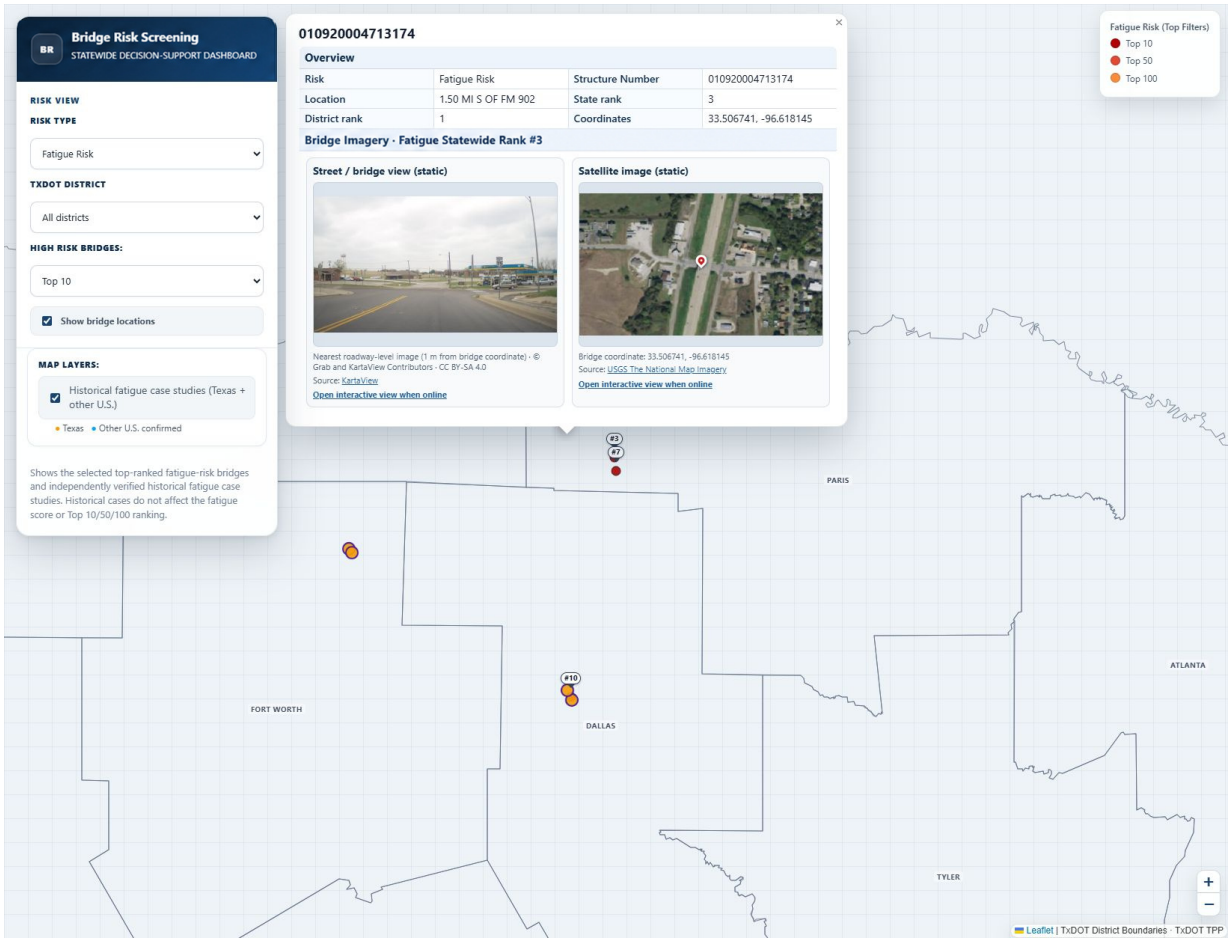


Figure 6.3: Bridge-level fatigue-risk popup showing identification, location, statewide and district ranks, coordinates, static roadway-level imagery, and a static satellite image.

SECTION 7. Recommended TxDOT Manual and Implementation Language

7.1 Manual Integration Framework

Sections 2 and 3 define the final statewide screening methods and results, Section 4 explains how TxDOT can move from a ranked bridge to an action, and Section 5 identifies the technical review and response categories applicable to each ranking factor. This section translates those research products into proposed language for existing TxDOT manuals and guidance. It identifies where the screening, bridge-record review, and application steps fit within current TxDOT practice and provides text for inclusion in TxDOT manuals.

The proposed statewide screening procedure applies to the ten risk categories evaluated through the final statewide ranking framework: explosive/security criticality, fatigue, flood overtopping and scour, hurricane surge exposure, heavy-vehicle bridge pier collision, overload, overheight collision, tornado, vessel collision, and wildfire. The first category remains a separate controlled security review; the other nine also contribute to the overall multi-hazard recurrence ranking. These categories are listed here to define the scope of the proposed manual language. Their data, equations, calculations, and statewide results remain in Sections 2 and 3, while bridge-level review and response selection remain in Section 5.

TxDOT's existing manuals already provide the principal technical and administrative pathways needed to act on an identified bridge concern. The Bridge Inspection Manual addresses routine, damage, special, underwater, and nonredundant steel tension member inspections; critical findings; follow-up actions; posting; and bridge records[213]. The Hydraulic Design Manual addresses hydraulic design, floodplains, scour, waterway changes, and coastal hydraulic conditions[215]. The Bridge Design Manual–LRFD provides bridge design criteria and design-documentation requirements[214]. The Bridge Project Development Manual addresses advanced planning, project selection, scope development, and delivery considerations[216]. The Maintenance Operations Manual provides the bridge-maintenance context for addressing identified needs[217]. The recommended approach is therefore to add compatible language to these existing documents rather than create a separate bridge-risk manual.

Four principles govern the proposed language in this section.

1. **Statewide screening identifies candidates for review.** A high risk-specific or overall rank determines where review begins. The ensuing bridge-record review and applicable technical procedure establish the condition finding, design determination, load restriction, maintenance action, or project decision.
2. **Bridge-record review precedes action.** Staff should check the bridge identity, source information, current condition, and factor producing the ranking through the applicable inspection, engineering, hydraulic, operational, or site-review process before selecting a response.
3. **Existing technical procedures govern the response.** Once bridge review identifies the factor requiring action, the applicable TxDOT procedure for inspection, load rating, hydraulic evaluation, maintenance, emergency response, design, or project development should govern the technical decision. Section 5 identifies the appropriate pathway.
4. **Updated information returns to the next screening cycle.** Corrections to bridge information and material changes resulting from inspection, analysis, maintenance, or completed work should be incorporated

into the applicable existing record so that the next statewide assessment uses current information.

All indented manual-language blocks in Section 7.3 are proposed research-team text for TxDOT review. They are organized by the manual or guidance document in which the concept would be used. The text distinguishes statewide screening from decisions made through existing TxDOT procedures and avoids creating separate priority classes, forms, approval chains, or organizational responsibilities.

7.2 Recommended Manual Locations and Content Crosswalk

Table 7.1 identifies the functional locations in current TxDOT documents where the research products can be incorporated. Functional locations are used because TxDOT can place the proposed text within the corresponding current provision during subject-matter review without relying on a chapter number that may change when a manual is revised.

Table 7.1: Recommended TxDOT manual integration crosswalk.

TxDOT document or record context	Proposed functional location	Research element to integrate	Recommended addition
Bridge Inspection Manual	Damage and special inspection; risk-informed inspection planning; critical findings and follow-up; load posting, vertical clearance, scour, underwater, and NSTM provisions	Use a ranked bridge as a candidate for focused review; check bridge identity, current condition, applicable event or exposure, and supporting records; apply the appropriate existing inspection pathway	New or revised procedural text and cross-references
Hydraulic Design Manual	Bridge hydraulics, scour evaluation, coastal hydraulics, waterway change, and post-event hydraulic review	Use current flood, hurricane-surge, and vessel-exposure information to identify when project-specific hydraulic, scour, coastal, or channel review is appropriate; retain information needed for later inspection	New or revised procedural text and bridge-file handoff language
Bridge Design Manual–LRFD	New design, replacement, widening, rehabilitation, major repair, collision protection, fatigue-sensitive details, and design documentation	Consider the bridge conditions and exposures identified during review when defining design criteria, protective systems, structural details, inspection access, and future maintenance needs	New or revised design-review and documentation text

TxDOT document or record context	Proposed functional location	Research element to integrate	Recommended addition
Bridge Project Development Manual	Advanced planning, project screening, scope development, alternatives, program coordination, and project closeout	Use current bridge and risk information as one input to project scoping and coordination; identify the applicable risk factor and the applicable technical study or treatment without treating rank as project priority	New or revised planning and scoping text
Maintenance Operations Manual	Bridge maintenance needs, inspection follow-up, damage response, recurring site conditions, completed work, and coordination with existing bridge-maintenance processes	Use the observed bridge condition to select the maintenance response and return completed-work information to the relevant bridge record	New or revised bridge-maintenance text
Existing bridge information systems and official bridge files	Existing locations for inventory, inspection, rating, hydraulic, maintenance, event, restriction, repair, and project information	Correct source information used by the statewide screening and retain the current information needed for the next update	Cross-reference to existing record practices; no separate risk database or recordkeeping system

The state and Federal comparison supports this integration approach. The comparison shows that the most transferable practices are focused procedural connections: event or condition information is directed to an appropriate inspection or engineering review; project-specific findings are retained for later use; and bridge management information is used to support subsequent maintenance or project decisions. Table 7.2 condenses the comparison to the practices that directly support the proposed TxDOT language. Appendix A contains the detailed literature and practice context.

Table 7.2: Concise cross-state and Federal practice comparison supporting the proposed manual additions.

Risk or implementation topic	Example practice in other DOT or Federal guidance	Relevant TxDOT document context	Implication for proposed TxDOT language
Damage and overheight impact	Arkansas treats overheight impact as a damage-inspection condition and documents member damage; Alaska uses damage inspection to determine whether restriction, closure, in-depth inspection, or analysis is needed[223, 232]	Bridge Inspection Manual	Connect a recorded impact or observed damage indication to the applicable damage-inspection and engineering-review provisions
Vertical clearance and routing	South Dakota and North Dakota connect measured clearance and construction-related clearance changes to oversize-vehicle routing information[385, 417]	Bridge Inspection Manual; project and maintenance records	Measure the controlling clearance and update applicable records after a field or project change
Flood, scour, and unknown foundations	Federal guidance and state scour practices emphasize scour evaluation, plans of action, foundation information, channel observations, and post-flood review[220, 240, 258, 260]	Hydraulic Design Manual; Bridge Inspection Manual	Connect statewide screening to the existing hydraulic, scour, underwater-inspection, and post-event review pathways
Vessel collision	Federal guidance and Delaware practice connect navigable-waterway exposure, channel and pier geometry, vessel characteristics, and protective systems to bridge-specific evaluation[252, 259]	Bridge Design Manual–LRFD; Bridge Inspection Manual	Limit application to bridges that cross an active navigable waterway and direct exposed piers or spans to collision evaluation or protection review
Load capacity and posting	State load-rating guidance retains the controlling analysis information and basis for posting or removing a posting[324, 343, 439, 450]	Bridge Inspection Manual; Bridge Design Manual–LRFD	Use overload screening as a review input and rely on current load rating, condition, and posting procedures for action

Risk or implementation topic	Example practice in other DOT or Federal guidance	Relevant TxDOT document context	Implication for proposed TxDOT language
Fatigue and NSTM	Other state practices connect fatigue-prone details, close-up inspection, nondestructive evaluation, crack mapping, and repair history to follow-up decisions[224, 243, 250, 263]	Bridge Inspection Manual; Bridge Design Manual–LRFD	Direct fatigue-prone details, cracks, or calculated fatigue-life needs to the existing NSTM, focused-inspection, analysis, repair, or rehabilitation pathway
Coastal storms and post-event review	Coastal-state and Federal practices connect storm exposure, surge or wave effects, approach vulnerability, scour, and post-storm inspection[257, 265, 309]	Hydraulic Design Manual; Bridge Inspection Manual; project development	Use bridge and site elevation data with coastal-hazard information to support hydraulic review, inspection planning, and project scoping
Maintenance and project integration	Florida, Wisconsin, Utah, and Washington connect bridge information to maintenance needs, asset management, and project programming rather than treating inspection as the end of the process[265, 427, 428, 445, 452, 453]	Bridge Project Development Manual; Maintenance Operations Manual	Use current bridge and risk information in existing maintenance and project-development decisions and return completed-work information to current records

The crosswalks connect statewide screening results to existing TxDOT technical processes and return updated information to the next screening cycle.

7.3 Proposed TxDOT Manual Language

This section provides proposed text organized by target TxDOT manual or guidance context.

Bridge Inspection Manual.

Proposed insertion point. Damage and special inspection provisions; inspection planning; critical findings and follow-up actions; load posting and vertical-clearance provisions; scour, underwater, and NSTM inspection provisions.

Purpose. Connect the statewide screening results to bridge-record review and the applicable existing inspection pathway. The proposed language keeps the screening rank separate from an inspection finding and directs the reviewer to the bridge information relevant to the indicated risk mechanism.

Proposed language–use of statewide screening. Statewide bridge-risk screening results iden-

tify candidate bridges for focused review. The reviewer checks the bridge identity, the information contributing to the elevated rank, the current bridge and site condition, and any completed work or intervening event that changed the factor producing the ranking. The applicable inspection finding, load rating, hydraulic evaluation, structural analysis, or other bridge-specific engineering determination then establishes the technical basis for an inspection, restriction, maintenance, or project decision.

Proposed language–selection of the inspection pathway. When bridge-record review identifies the risk mechanism indicated by the statewide screening, the bridge should be evaluated through the applicable existing inspection provision. Depending on the observed condition, the applicable provision may include routine inspection with focused observations, damage inspection following a reported impact or event, special inspection, underwater inspection, scour monitoring or plan-of-action review, NSTM inspection, vertical-clearance measurement, load-posting review, or critical-finding procedures. The inspection scope should address the members, exposure, event information, or site condition that produced the ranking.

Proposed language–bridge impact and clearance information. For a reported or suspected heavy-vehicle, overheight-vehicle, or vessel impact, the inspection record should identify the bridge and impact location; the affected member, support, protective system, or navigation feature; observed displacement, cracking, section loss, connection damage, bearing movement, or other distress; photographs and measurements needed to describe the condition; and the applicable engineering review or traffic action. When measured clearance, signed clearance, roadway work, or route information is inconsistent, staff should measure the controlling clearance and correct the applicable bridge, signing, or routing information through the existing TxDOT process.

Proposed language–flood, scour, and storm review. When a flood, coastal storm, high-water event, or inspection finding indicates possible scour, foundation exposure, countermeasure damage, channel migration, debris blockage, approach washout, settlement, or structural movement, the inspection should document the observed condition and identify the applicable need for underwater inspection, hydraulic review, scour plan-of-action implementation, restriction, closure, or follow-up inspection under existing TxDOT procedures.

Proposed language–fatigue review. When a bridge is selected for fatigue-focused review, the inspector should check that the bridge and members are within the applicable steel and fatigue-detail population; examine the fatigue-prone details identified by the bridge record and inspection plan; compare current findings with prior crack maps, nondestructive-evaluation results, photographs, repairs, and condition observations; and refer an identified crack, crack growth, severe section loss, or other material fatigue concern to the applicable engineering, load-rating, critical-finding, repair, or project-development procedure.

Proposed language–data updates following review. When bridge-level review identifies an incorrect structure match, outdated inventory value, completed repair, changed site condition, or other material difference from the statewide screening input, the corrected information should be entered or transmitted through the applicable existing TxDOT bridge record process so that the next statewide screening update uses current information.

Hydraulic Design Manual.

Proposed insertion point. Bridge hydraulic analysis, scour evaluation, waterway stability, coastal hydraulic analysis, design documentation, and post-event hydraulic review.

Purpose. Connect flood, scour, hurricane-surge, and navigable-waterway concerns identified during bridge review to the appropriate project-specific hydraulic analysis and preserve information needed by later inspectors and project developers.

> **Proposed language–risk-informed hydraulic review.** Statewide flood, hurricane-surge, and vessel-exposure screening identifies bridges for review of existing hydraulic, scour, coastal, channel, and navigation information. Before recommending additional analysis or treatment, the reviewer checks the bridge location and geometry, effective low chord or controlling elevation, foundation and scour information, applicable flood or surge surface, modeled or observed hydraulic conditions, channel and approach condition, and the date and scope of the available analysis. Project-specific hydraulic information governs when it is more current or more detailed than the statewide screening input.
>
> **Proposed language–design information for future inspection.** For a new, replaced, widened, or substantially rehabilitated bridge over a waterway, the hydraulic documentation retained or referenced in the applicable bridge or project record should include the information needed to interpret future flood, scour, channel, and coastal observations. As applicable, this information includes controlling elevations, foundation information used in the scour evaluation, baseline channel or cross-section information, modeled water levels and velocities, applicable surge or wave conditions, scour-countermeasure details, datum or benchmark information, and conditions identified for future inspection or review.
>
> **Proposed language–post-event coordination.** When inspection following a flood, storm, or high-water event identifies a material change in the channel, foundation exposure, scour, countermeasure damage, debris blockage, approach loss, overtopping effects, or other hydraulic condition affecting bridge performance, the applicable hydraulic or scour evaluation should be reviewed before the bridge record, plan of action, repair scope, or reopening basis is finalized.
>
> **Proposed language–navigable-waterway application.** Vessel-collision screening should be applied only to a bridge that crosses a navigable waterway or another waterway with relevant vessel activity. For those bridges, available vessel activity, navigation channel, current, bridge geometry, pier location, navigation clearance, protective-system, and incident information should be provided to the bridge-specific collision or design review required by the applicable TxDOT and AASHTO provisions.

Bridge Design Manual–LRFD.

Proposed insertion point. Design criteria and design notes for new bridges, replacement, widening, rehabilitation, strengthening, major repair, collision protection, and fatigue-sensitive details.

Purpose. Include bridge conditions and exposures relevant to a design or rehabilitation project in the project-specific design basis and retain information needed for later inspection and maintenance.

Proposed language–design applicability review. During design of a new, replacement, widened, substantially rehabilitated, or major-repair bridge project, the designer should review available bridge and site information to identify risk conditions relevant to the project. The review should consider, as applicable, fatigue-sensitive details and truck demand; flood, scour, surge, and foundation conditions; heavy-vehicle or vessel collision exposure; overload and load-rating information; vertical clearance and overheight-strike history; tornado or extreme-wind exposure relevant to the project; wildfire exposure and vulnerable bridge components; and authorized security-criticality information. The applicable design specification and project-specific analysis govern the resulting design decision.

Proposed language–rehabilitation and major repair. For rehabilitation, widening, strengthening, deck replacement, or major repair, the design basis should identify the condition or exposure being addressed, the current inspection and load-rating information, relevant hydraulic or foundation information, prior damage and repairs, fatigue or NSTM concerns, collision-protection or clearance considerations, and inspection or maintenance access needs. The selected strategy should address the project condition and should identify information that must be available for future inspection, rating, hydraulic review, or maintenance.

Proposed language–collision and protection systems. When roadway geometry or vessel tracks allow a heavy vehicle or vessel to reach an exposed bridge component, the designer should evaluate the applicable bridge geometry, traffic or vessel characteristics, exposed structural components, foundation and support configuration, existing protection systems, incident history, and consequence of damage under the governing design provisions. The design record should identify the basis for retaining, repairing, modifying, or adding a protective measure when such a measure is included in the project.

Proposed language–design information handoff. The project record should retain or reference the final information needed to support subsequent bridge inspection, load rating, hydraulic review, maintenance, and repair. This information may include controlling design assumptions, member or detail locations requiring focused inspection, final load-rating information, hydraulic and scour information, foundation assumptions, collision-protection details, vertical-clearance information, access provisions, and as-built changes affecting future review.

Bridge Project Development Manual.

Proposed insertion point. Advanced planning, candidate-project review, scope development, alternatives, project coordination, and project closeout.

Purpose. Use statewide screening information as one project-development input while keeping calculated rank separate from final project priority, scope, and funding decisions.

Proposed language–candidate-project review. A bridge identified through a statewide risk-specific or multi-hazard recurrence ranking enters project-development review after staff check the bridge identity, source information, current condition, and factor producing the ranking. Staff evaluate the screening rank together with current inspection and rating information, life-safety and mobility consequences, detour and freight effects, treatment effectiveness,

constructability, cost, project readiness, funding eligibility, and opportunities to coordinate the response with planned work.

Proposed language–scope development. When a bridge condition or exposure is relevant to a proposed bridge replacement, rehabilitation, widening, strengthening, or major repair, the project scope should identify the condition or exposure to be addressed and the applicable technical coordination. Depending on the project, this may include bridge inspection, structural evaluation, load rating, hydraulic and scour analysis, coastal analysis, collision review, navigation coordination, vertical-clearance review, fatigue or NSTM evaluation, wildfire site review, security review, maintenance access, or emergency-route considerations.

Proposed language–relationship to project priority. Statewide bridge-risk rankings identify candidates for project review. The applicable TxDOT process determines final project priority and programming using the screening rank together with current engineering information, consequences, urgency, feasibility, cost, readiness, and program criteria.

Proposed language–project closeout information. At project completion, final information that materially changes a risk-screening input or the interpretation of a factor contributing to the ranking should be incorporated into or referenced by the applicable existing bridge, inspection, rating, hydraulic, maintenance, or project record. Examples include a corrected bridge attribute, changed low chord or clearance, new load rating, repaired fatigue detail, added collision protection, modified scour countermeasure, altered channel condition, or completed protective work.

Maintenance Operations Manual.

Proposed insertion point. Bridge-maintenance needs, inspection follow-up, recurring site conditions, damage response, maintenance planning, and completed-work information.

Purpose. Connect an observed bridge condition to the applicable existing maintenance response and return completed-work information to later bridge review and statewide rescreening.

Proposed language–use of risk information. Statewide bridge-risk screening identifies candidate locations for maintenance review. Before selecting a maintenance action, district personnel check the bridge, current site and component condition, applicable inspection or engineering finding, and whether completed work has already addressed the indicated condition. The applicable inspection finding, engineering recommendation, maintenance need, or existing TxDOT response procedure provides the basis for the work decision.

Proposed language–maintenance response. When the bridge condition is appropriate for maintenance action, the response should be selected through the existing bridge-maintenance process. Applicable work may include removal of debris or combustible vegetation, drainage or channel-related maintenance within the authorized scope, repair of signs or warning devices, repair of damaged protective components, access improvement, minor structural or nonstructural repair, monitoring support, or coordination of a condition requiring engineering evaluation or project development.

Proposed language–completed work and current condition. Information needed to establish the current bridge or site condition after completed maintenance should be retained or transmitted through the applicable existing record process. When completed work changes an input used by the statewide screening or removes the condition that produced the ranking, that information should be available to the next statewide update.

Existing bridge information systems and official bridge files.

The proposed manual language integrates the screening procedure with existing TxDOT information and record processes. The statewide analysis uses current TxDOT information, and bridge-level review returns corrections and material updates to the process responsible for that information. Security-criticality inputs and findings remain within the applicable controlled TxDOT review and information environment.

The following examples illustrate application of the proposed language without adding a new procedure.

- **Flood example.** A bridge identified through the flood ranking is first checked for the correct structure, low chord, mapped flood exposure, modeled velocity, scour information, and recent inspection or channel change. If those data and current site information support the flood or scour concern, the bridge enters the applicable hydraulic, scour, underwater-inspection, maintenance, or project-development pathway. If the controlling elevation or structure match is incorrect, the relevant source information is corrected before the next statewide update.
- **Overheight example.** A bridge identified through the overheight ranking is checked for the controlling measured clearance, signed clearance, roadway configuration, truck exposure, and accepted strike history. A measured clearance deficiency or recorded strike is addressed through the applicable clearance, signing, damage-inspection, traffic, maintenance, or design process. A superseded measurement or incorrect under-road match is corrected in the applicable record.
- **Fatigue example.** A bridge identified through the fatigue ranking is checked for steel and fatigue-detail eligibility, truck loading, current condition, focused inspection findings, crack history, nondestructive-evaluation results, and completed repairs. A crack, calculated fatigue-life deficiency, or other fatigue condition identified during review proceeds through the applicable NSTM, special-inspection, load-rating, repair, rehabilitation, or design process. A bridge outside the applicable fatigue population or a repaired detail no longer represented by the statewide input is documented through the existing process for use in the next update.

7.4 Supplemental Literature- and Practice-Based Recommendations

The statewide ranking framework did not produce final risk-specific rankings for fire other than wildfire, seismic risk, or landslide/geohazard risk. The recommendations in this section therefore arise from the literature and state-practice review in Appendix A. They are presented separately from the categories with final statewide rankings and can be considered when TxDOT reviews the corresponding existing inspection, design, hydraulic, maintenance, or emergency-response provisions.

Fire other than wildfire. State and agency practices commonly treat a bridge fire or heat exposure as a potential damage-inspection condition because thermal effects can alter material properties, connections, bearings, protective coatings, and structural geometry. Proposed language for consideration is:

When a reported fire or heat event may have affected a bridge component, a damage inspection

should document the fire location and duration information available, the materials and members exposed, visible distortion or displacement, concrete cracking or spalling, steel or connection effects, bearing and joint condition, protective-system damage, and any traffic restriction or engineering evaluation needed under existing TxDOT procedures. The bridge should be returned to normal service through the applicable inspection and engineering process.

Seismic risk. TxDOT's current context uses targeted applicability review and post-event inspection for seismic risk. Practices from Alaska, Montana, Massachusetts, Utah, Washington, Oregon, and New York provide examples of seismic applicability, retrofit screening, and post-earthquake inspection[224, 323, 349, 378, 397, 430, 443]. Proposed language for consideration is:

> Where the applicable design provision, project condition, or reported seismic event warrants review, bridge inspection or design personnel should consider bearings, joints, beam seats, girder movement, columns and foundations, approach settlement, restraint details, and other components relevant to loss of support or structural movement. Observed displacement, loss of bearing support, significant cracking, differential settlement, or related damage should be evaluated through the applicable TxDOT inspection, restriction, structural-evaluation, repair, or reopening process.

Landslide and geohazard risk. Literature and practice indicate that slope movement, rockfall, embankment instability, settlement, erosion, and ground movement can affect bridge approaches, foundations, retaining features, drainage, and access. Proposed language for consideration is:

> When inspection, maintenance, construction, or event information indicates slope movement, rockfall, embankment distress, differential settlement, erosion, ground cracking, retaining-feature movement, or loss of approach support near a bridge, the observed condition should be documented and referred to the applicable geotechnical, hydraulic, structural, maintenance, traffic, or project-development review. Follow-up should be based on the observed site condition and the existing TxDOT procedure appropriate to the affected facility.

These supplemental recommendations provide manual-language directions for seismic, landslide, and geohazard conditions identified in the literature and practice review. Together, Sections 4 through 7 complete the project logic: Section 4 defines how ranked bridges are reviewed, Section 5 identifies the applicable technical response, Section 6 presents the GIS Tool, and Section 7 identifies where the screening, bridge-record review, and application steps can be expressed within existing TxDOT manuals and guidance. Section 8 summarizes the resulting statewide framework and its value to TxDOT.

SECTION 8. Conclusions

8.1 Project Outputs

Project 0-7239 produced ten risk-specific statewide screening methods, corresponding bridge rankings and maps, a multi-hazard recurrence ranking, a GIS Tool, bridge-review and response-selection procedures, ten hazard-specific decision trees, and recommended TxDOT manual language. Together, these products connect statewide screening to bridge-level review: the rankings identify where review begins, the risk-specific results explain why a bridge appears, the decision trees guide the response, and the next data update returns the completed review to the statewide process.

8.2 Findings

- **The risk modules identify different bridge populations.** Only 12 structure identifiers appear in both the coastal Top 100 population formed by hurricane surge and vessel collision and the inland Top 100 population formed by flood, tornado, and wildfire. Individual risk lists are therefore necessary even when TxDOT also uses the multi-hazard recurrence ranking.
- **Houston's overall concentration reflects recurrence across several lists.** Houston District contains 51 of the multi-hazard Top 100, followed by Amarillo with 17 and Fort Worth with 9. The Houston total results from repeated placement across coastal, navigation, wind, traffic, and loading modules; it is a recurrence count, not a comparison of raw scores from unlike hazards.
- **Data completeness changes who can be ranked.** The flood method lacks paired inputs for 3,570 of 12,479 waterway bridges, or 28.6 percent, and the overheight method lacks complete inputs for 2,576 of 9,223 candidates, or 27.9 percent. Hurricane surge lacks clearance data for 99 of 13,193 coastal records, or 0.8 percent. Low-chord, clearance, roadway, and related source-data updates will add the largest number of eligible bridges to the first two modules.
- **Several leading results form groups, but two collision locations are distinct outliers.** The ten hurricane-surge leaders all lie in Houston District and have probabilities that round to 0.048771, so their order depends on tie breakers. In contrast, the leading heavy-vehicle pier-collision site has 46 assigned crashes compared with 16 at rank 2, and the leading overheight site has 3.457 expected strikes per year compared with 0.389 at rank 2. These gaps support immediate location-specific review of the two collision leaders.
- **The recurrence score must be read with its contributing modules.** The two statewide leaders each have a recurrence score of 9, but their risk combinations differ. Likewise, ranks 3 and 4 both score 7 even though one combines overload, fatigue, and tornado while the other combines flood, vessel collision, and tornado. Equal recurrence scores therefore lead to different technical reviews and responses.
- **The six-step process closes the management loop.** TxDOT selects candidate bridges, checks identity and source information, reviews current condition and identifies the ranking factor, selects a response, updates the relevant data and records, and reruns and reviews the statewide rankings. This sequence makes each completed review part of the next statewide screening cycle.

APPENDIX A. Literature and Practice Synthesis

A.1 Introduction

This appendix synthesizes the literature and agency practices used to establish the technical foundation for the statewide bridge hazard and vulnerability screening methods. It distinguishes methods reported by prior studies and agencies from the adopted project calculations presented in Section 2.

- The research team reviewed materials from national databases, other state DOT websites, agency technical reports, and academic sources to cover the hazard categories, data sources, and practices relevant to TxDOT.

- The research team compiled and synthesized bridge risk-assessment workflows across multiple hazard categories, including fires; vessel and vehicle collisions; natural hazards such as flooding and high winds; structural deficiencies related to aging and corrosion; and explosive or intentional-attack scenarios. For each category, the appendix summarizes key definitions, primary drivers, typical impacts, and methods reported in the literature.

- The research team identified the inventory, condition, traffic, exposure, and event data used by the reviewed approaches. These findings informed the adopted statewide methods and implementation recommendations.

A.2 Explosive Risk

A.2.1 Introduction

Definition. Explosive risk refers to the combined likelihood and consequence of an explosive event, intentional or accidental, that can damage bridge components, degrade load-carrying capacity, trigger progressive collapse mechanisms, or cause prolonged loss of service and cascading network disruption. From a bridge owner/operator perspective, the hazard is often framed as an intentional multi-threat security problem involving blast, fire, or sabotage. The performance objective includes resilience: absorbing the defined hazard, recovering, and returning to function within an acceptable time window. Because bridges are open systems with many access points and interdependencies involving utilities, freight corridors, and emergency routes, explosive risk is typically managed as part of a broader critical-infrastructure risk framework that evaluates threat, vulnerability, and consequence jointly to prioritize mitigation investments [18, 19].

Recent incidents. Events reported during the most recent five-year review period include deliberate attacks and explosive incidents during emergency operations Table A.1.

Government guidance and protective approaches. In the United States, federal guidance for protecting bridges against intentional hazards has developed along both engineering and homeland-security lines. FHWA maintains a bridge-and-tunnel safety and security program that explicitly treats blast and sabotage as representative intentional hazards and promotes solutions through R&D, case studies, and training [34]. Shortly after 9/11, FHWA and AASHTO convened a Blue Ribbon Panel to propose short-, mid-, and long-term strategies, including risk-based prioritization, operational security, and design/engineering guidance aimed at reducing vulnerabilities and limiting collapse potential under attack scenarios [35]. At the national policy level, the Department of Homeland Security's NIPP 2013 provides an overarching risk-management and partnership structure for critical infrastructure, within which transportation assets such as bridges are treated as lifeline functions requiring coordinated protection and resilience planning across federal, state, and local stakeholders [18]. Complementing this, the Transportation Systems Sector-Specific Plan tailors the NIPP framework to the transportation domain and is intended to guide partners (including state and local entities) in setting security and resilience priorities and activities for manmade threats and other hazards [19]. For operational planning and response, interagency products such as the NCTC/DHS/FBI "Terrorism Planning and Incident Response Considerations for Bridges" provide practical pre-incident collaboration and incident-management considerations specific to attacks on bridges [36]. Also, government R&D organizations also provide technical tools to support vulnerability assessment and protective design. For example, the U.S. Army ERDC's ATP-Bridge is described as a capability for predicting bridge component damage under explosive threats and enabling rapid vulnerability assessment [37].

Representative Case Studies in Texas

- Although there are cases with controlled demolition using explosives, the Research Team did not find any bridges in Texas that failed due to accidental explosions.

Table A.1: Summary of Bridge Structural Damage and Explosive Events (2022-2025)

Date	Location	Event Type	Consequences (Summary)	Ref.
2022-10-08	Kerch Strait / Crimean Bridge	Truck Bomb/IED	Fire, partial road collapse, rail tanker fire; traffic severed.	[20]
2022-12-24	Boksburg, South Africa	LPG Tanker Blast	Explosion and fireball; high casualties; thermal/blast risk to structure.	[21]
2023-06-11	I-95 Philadelphia, USA	Fuel Tanker Fire	Fire under bridge caused structure collapse; severe I-95 disruption.	[22]
2023-07-17	Crimean Bridge	USV/Explosion	Bridge deck damage, casualties, and traffic disruption/diversion.	[23]
2023-12-21	Glenville, NY, USA	CNG Bridge Strike	Truck struck low bridge and exploded; integrity assessment required.	[24]
2024-05-02	Norwalk, CT, USA	Tanker Fire	Severe thermal damage to overpass; required emergency demolition.	[25]
2024-05-13	Baltimore, MD, USA	Controlled Demo	Explosive removal of bridge wreckage pinning the ship's hull.	[26]
2024-08-17	Kursk, Russia	Missile/Rocket	Destruction of Seym River bridges due to hostilities.	[27]
2025-05-31	Bryansk, Russia	Blast Collapse	Bridge collapse impacting a train, resulting in casualties.	[28]
2025-06-01	Kursk, Russia	Sabotage Blast	Rail bridge collapse during freight transit; driver injured.	[29]
2025-06-03	Crimean Bridge	Underwater Blast	Claimed detonation at bridge piers and reported blast exposure of the foundations.	[30]
2025-06-14	Aiken County, SC, USA	Tanker Fire	Significant damage to I-20 overpass; required emergency rebuild.	[31]
2025-08-25	Shan State, Myanmar	Conflict Blast	Disputed bombing/mining causing structural destruction.	[32]
2025-09-10	Mexico City, MX	LPG Explosion	Tanker blast under bridge; thermal and shock risk to structure.	[33]

A.2.2 Methods Reported in the Literature

A.2.2.1 AASHTO Bridge Security Guidelines [3]

This document describes a two-level workflow for bridge security risk assessment:

1. **Network level (bridge level):** rank all bridges by criticality to decide which bridges merit deeper security analysis and investment.
2. **Project level (component level):** for bridges deemed critical, conduct a component-based risk assessment to prioritize mitigation, strengthening, and resource allocation.

A key principle evaluates "criticality" and "vulnerability" separately. Criticality screening identifies system-level importance, and the subsequent vulnerability assessment evaluates specific components and attack modes.

Step-by-step procedure

Step 1: Bridge-Level Screening via Criticality Factor For a bridge network, the owner/agency first computes a Criticality Factor (CF) for each bridge and sorts bridges in descending order:

$$\text{CF} = S + E \times D \tag{A.1}$$

where,

- S = Symbolic Importance: a 6-level ordinal score from "largely unknown to the public" to "national landmark/icon" (Table A.2).
- E = Economic Importance: a 6-level ordinal score based on Average Daily Traffic (ADT), e.g., < 100, 100–5,000, 5,000–25,000, …, up to $> 200{,}000$ vehicles/day.
- D = Detour Impact: a 6-level ordinal score based on the distance to the nearest comparable detour route/bridge,
- from "multiple alternatives within 10 miles" to "no alternative within 100 miles (or no practical alternative)."

If multiple bridges have identical CF values:

1. use $E \times D$ as the first tie-breaker (higher $E \times D$ ranks higher),
2. then use E as the second tie-breaker.

Exceptional amplification: In rare, extreme circumstances (e.g., the bridge is the sole access to an emergency facility), the owner may apply an amplification factor α to the detour-economic term, i.e., use $\alpha(E \times D)$, but this should be used exceptionally rarely.

This step identifies bridges for detailed security analysis and investment. The subsequent vulnerability assessment evaluates specific components and threat modes.

Step 2: Component-Level Risk Assessment on Critical Bridges For bridges selected as critical, perform a component-level risk assessment. The basic risk logic is:

$$\text{Risk} = \text{Occurrence} \times \text{Vulnerability} \times \text{Importance} \tag{A.2}$$

Table A.2: Criticality Factor Variables

Criteria	Value	Definition
Symbolic Importance, S	1	Largely invisible in the community
	2	Visible within the community
	3	Visible statewide
	4	Visible nationwide
	5	Visible worldwide
	6	Iconic national symbol
Economic Importance, E	1	Fewer than 100 vehicles per day
	2	100 to 5,000 vehicles per day
	3	5,000 to 25,000 vehicles per day
	4	25,000 to 100,000 vehicles per day
	5	100,000 to 200,000 vehicles per day
	6	More than 200,000 vehicles per day
Detour Impact, D	1	Two or more comparable detours within 10 miles
	2	One within 10 miles
	3	None within 10 miles
	4	None within 20 miles
	5	None within 50 miles
	6	None within 100 miles, or no alternate route

Source: [3, Table 1.2-1]

Let j index bridge components (e.g., a specific pier column, a tower, a segment of main cable), and let i index fundamental threat types (e.g., explosives, cutting, arson/fire, vehicular/vessel impact). The total bridge risk indicator is:

$$R = \sum_{j} \left[I_j \sum_{i} (O_{ij} V_{ij}) \right] \tag{A.3}$$

where,
I_j = Component Importance (relative importance of component j to overall bridge performance);
O_{ij} = Threat Occurrence (relative likelihood of threat i being realized against component j);
V_{ij} = Vulnerability (relative susceptibility of component j under threat i).

The resulting R and component contributions R_j are normalized to the bridge under evaluation and support comparison of components within that bridge.

Component Importance I_j Component importance is computed as a weighted sum of multiple attributes, multiplied by a span-ratio factor:

$$I_j = \left(\sum_{k} w_{f,k}\, a_k \right) SR_j \tag{A.4}$$

where,

- $w_{f,k}$ = weighting factor for attribute k (Table A.3);
- a_k = attribute score (Table A.4). Typical attributes include: Structural importance: is it a critical single component whose failure causes global instability? Is there redundancy or alternate load paths? Historic/cultural importance: is the component historically irreplaceable or easily replicated? Relative repair cost: repair cost for this component relative to other bridge components. Time out of service: expected closure duration if this component is damaged.

- SR_j = Span Ratio = $\dfrac{\text{span length associated with component } j}{\text{main span length}}$, which reflects that loss of main-span components is generally more consequential than approach-span components.

Table A.3: Weight factor

Table 1.3-13—Weight Factor for Vehicular Impact, *wf*

Risk Factor	Component Attribute, *a*	Weight Factor, *wf*
Importance	Structural importance	0.5
	Historic importance	0.1
	Repair cost	0.2
	Time out of service	0.2
Occurrence	Threat likelihood	0.1
	Threat likelihood against component	0.3
	Visibility or attractiveness of component	0.1
	Access to component	0.5
Vulnerability	Resistance of component to basic threat, *t*	1.0

Source: [3, Table 1.3-13]

Table A.4: Component Importance Attributes, a_k

Table 1.3-1—Component Structural Importance Attribute, a_i

Value	Definition
1.00	Critical component with no alternate load path nor the additional capacity needed to withstand the threat while maintaining its service load, whose failure is likely and would lead to global structural instability
0.75	Critical component with robustness but no redundancy, whose failure is not likely but would lead to global structural instability
0.50	Important component that has at least one other load path
0.25	Important component with multiple other load paths
0.00	Noncritical to overall stability

(a) Structural Importance

Table 1.3-2—Component Historic Importance Attribute, a_i

Value	Definition
1.00	Historically irreplaceable and not reproducible by any method
0.75	Historically irreplaceable and not easily reproducible
0.50	Historically irreplaceable but easily reproducible
0.25	Historic but replaceable
0.00	None

(b) Historic Importance

Table 1.3-3—Relative Repair Cost Importance Attribute, a_i

Value	Definition
1.00	Very high
0.75	High
0.50	Medium
0.25	Low
0.00	Little to none

(c) Repair Cost

Table 1.3-4—Relative Time Out of Service Attribute, a_i

Value	Definition
1.00	Very long
0.75	Long
0.50	Medium
0.25	Short
0.00	Little to none

(d) Time Out of Service

Source: [3, Table 1.3-1 to 1.3-4]

Threat Occurrence O_{ij} O_{ij} is a screening score for how likely threat type i is to be carried out specifically against component j. It is computed by (i) assigning a rating value to a small set of sub-attributes using the guideline tables, and (ii) combining them with weights (either owner-defined or adopted from the guideline example).

$$O_{ij} = \sum_{\ell} w_\ell \, a_\ell \tag{A.5}$$

where $a_\ell \in [0, 1]$ is the score for sub-attribute ℓ and w_ℓ is its weight (typically $\sum_\ell w_\ell = 1$). The values of a_ℓ can be obtained from Table A.5 and the values of w_ℓ can be obtained from Table A.3.

Table A.5: Likelihood of Threat Analysis: (a) General Occurrence, (b) Against Specific Component, (c) Based on Visibility, and (d) Based on Access.

Table 1.3-5—Likelihood of Threat Occurrence, a_o

Value	Definition
1.00	Very high
0.75	High
0.50	Medium
0.25	Low
0.00	Little to none

(a) Occurrence

Table 1.3-6—Likelihood of Threat Occurrence against a Given Component, a_o

Value	Definition
1.00	Very high
0.75	High
0.50	Medium
0.25	Low
0.00	Little to none

(b) Against Component

Table 1.3-7—Likelihood of Threat against a Component Based on Visibility or Attractiveness, a_o

Value	Definition
1.00	Very high
0.75	High
0.50	Medium
0.25	Low
0.00	Little to none

(c) Visibility

Table 1.3-8—Likelihood of Threat Based on Access to a Component, a_o

Value	Definition
1.00	Completely accessible, RT >> AT
0.75	RT probably > AT
0.50	RT = AT.
0.25	RT probably < AT
0.00	Access completely denied, or AT >> RT

Note: The bridge owner shall approve appropriate values for the Attack Time (AT) and Response Time (RT) based on past events and response times of local emergency services.

(d) Access-based

Source: [3, Table 1.3.5 to 1.3.8]

Vulnerability V_{ij} In the guideline, the vulnerability for a given threat–component pair (i, j) is obtained by looking up a table value denoted a_v (the vulnerability attribute):

$$V_{ij} \equiv a_v(i, j) \tag{A.6}$$

i.e., for each threat type i (explosives, cutting, fire, impact, …), select the appropriate bin in the corresponding vulnerability table and assign that table's value directly to V_{ij}.

1. **Explosives (e.g., VBIED, HEIED)**

For this threat, the report suggests using Table A.6. This table assigns an explosive-vulnerability score $a_v \in [0, 1]$ based on how much explosive (as a multiple of a reference charge x) is needed to destroy the component. A larger a_v (closer to 1.0) indicates higher explosive vulnerability: the component can be destroyed with less than x explosive weight; a smaller a_v (closer to 0.0) indicates lower explosive vulnerability: destruction would require many multiples of x (or is negligible). In bridge security engineering, x is typically defined by the Design Basis Threat (DBT) category. For critical highway bridges, x is often set to a Large VBIED (approx. 4,000 lbs / 1,814 kg TNT) representing a delivery truck, or a Small VBIED (500 lbs / 227 kg TNT) for restricted-access areas.

2. **Cutting**

For this threat, the report suggests using Table A.7. This table assigns a cutting-vulnerability score $a_v \in [0, 1]$ based on how quickly a component would lose enough strength to fail if attacked with plausible cutting tools: $a_v = 1.00$ means failure in less than x minutes (very vulnerable), while smaller values mean it takes longer; $a_v = 0.00$ means cutting is not applicable or effectively not a concern for that component. Here x is an owner-selected reference time (in minutes) representing the critical time window for a cutting attack to succeed before detection/response, chosen using site-specific factors such as component accessibility, the ability to detect cutting, and local emergency response time (RT). The reference time interval x is typically set between 15 to 30 minutes, which represents the estimated emergency response time for the specific bridge location.

3. Fire

For this threat, the report suggests using Table A.8. This table assigns a fire-vulnerability score $a_v \in [0, 1]$ based on how quickly a component would lose sufficient strength to fail under a fire scenario: $a_v = 1.00$ means strength loss to failure in less than x minutes (very vulnerable), while smaller values indicate the component can sustain fire exposure longer (e.g., $< 3x$, $< 6x$, or $> 6x$ minutes); $a_v = 0.00$ means the component is not applicable or effectively not vulnerable to fire. Here x is an owner-selected reference

Table A.6: Vulnerability Attribute for Explosive Threats

Table 1.3-9—Vulnerability Attribute for Explosive Threats, a_v

Value	Definition
1.00	Component destruction with less than x explosive weight
0.67	Component destruction with x to $6x$ explosive weight
0.36	Component destruction with $6x$ to $20x$ explosive weight
0.20	Component destruction with $20x$ to $60x$ explosive weight
0.10	Component destruction with $60x$ to $120x$ explosive weight
0.06	Component destruction with $120x$ explosive weight
0.00	Little to none

Note: The bridge owner shall approve the size and location of threats to be considered for both hand-placed and vehicle-borne explosives.

Source: [3, Table 1.3-9]

Table A.7: Vulnerability Attribute for Cutting

Table 1.3-10—Vulnerability Attribute for Cutting, a_v

Value	Definition
1.00	Strength loss in less than x min
0.75	Strength loss in less than $2x$ min
0.50	Strength loss in less than $6x$ min
0.25	Strength loss in greater than $6x$ min
0.00	Not applicable or not vulnerable to cutting

Note: The bridge owner shall approve the time intervals to be considered based on the accessibility of the components, ability to detect cutting, and the response times of local emergency services.

Source: [3, Table 1.3-10]

time (in minutes) chosen based on site-specific factors such as component accessibility and the response time/capability of local emergency services (e.g., detection, dispatch, arrival, and suppression). In the absence of bridge-specific emergency response data, the reference time interval can be set to be $x \approx 10$ min as a practical "effective intervention window".

Table A.8: Vulnerability Attribute for Fire

Table 1.3-11—Vulnerability Attribute for Fire, a_v

Value	Definition
1.00	Strength loss in less than x min
0.75	Strength loss in less than $3x$ min
0.50	Strength loss in less than $6x$ min
0.25	Strength loss in greater than $6x$ min
0.00	Not applicable or not vulnerable to fire

Note: The bridge owner shall approve the time intervals to be considered based on the accessibility of the components and response times of local emergency services.

Source: [3, Table 1.3-11]

4. **Vehicular Impact**

For this threat, the report suggests using Table A.9. This table assigns a vehicular-impact vulnerability score $a_v \in [0, 1]$ based on the minimum vehicle size or weight class that can cause major damage to the component. A value of $a_v = 1.00$ means a small vehicle impact can cause complete destruction, which indicates the highest vulnerability. Values of $a_v = 0.75$ and $a_v = 0.50$ mean a medium or large vehicle, respectively, is needed to cause complete destruction, which indicates lower vulnerability. A value of $a_v = 0.25$ means that even a large-vehicle impact causes only partial destruction. A value of $a_v = 0.00$ means vehicular impact is not applicable or the component can withstand all vehicular impacts considered in the assessment. The bridge owner defines and approves the small, medium, and large vehicle classes using the local truck fleet, roadway restrictions, and selected design or assessment vehicles.

Table A.9: Vulnerability Attribute for Vehicular Impact

Table 1.3-12—Vulnerability Attribute for Vehicular Impact, a_v

Value	Definition
1.00	Complete destruction from a "small weight vehicle" impact
0.75	Complete destruction from a "medium weight vehicle" impact
0.50	Complete destruction from a "large weight vehicle" impact
0.25	Partial destruction from a "large weight vehicle" impact
0.00	Not applicable or can withstand all vehicular impacts

Source: [3, Table 1.3-12]

Step 3: Using the Indices for Decision-Making The guideline recommends using the risk indices defined above as an iterative risk–investment analysis tool. In particular, the baseline risk

$$R = \sum_j \left[I_j \sum_i (O_{ij} V_{ij}) \right] \tag{A.7}$$

and the component contributions $R_j = I_j \sum_i (O_{ij} V_{ij})$ are used to prioritize actions and to quantify the benefit of countermeasures:

1. Compute the baseline component risks R_j and total risk R for the critical bridge to identify the highest-priority components and dominant threats.
2. Propose candidate mitigation measures, such as increased stand-off distance, protective barriers, confinement reinforcement, anti-climb devices, access control, and surveillance.
3. For each proposed measure, update the affected indices and recompute risk. Measures that hinder access or improve detection typically reduce O_{ij}, measures that harden or shield the component typically reduce V_{ij}, and measures that add redundancy may reduce I_j.
4. Quantify benefit as a risk reduction $\Delta R = R - R'$ and compare it against implementation cost to select cost-efficient measures and stage implementation if needed.
5. Periodically repeat the assessment and update R as conditions change, including traffic, nearby critical facilities, land use, and the threat environment.

A.2.2.2 NCHRP 20-07/Task 378 [2]

In these proposed guidelines, explosive threats are treated as part of man-made extreme events, specifically within the Terrorism/Countering Violent Extremism hazard class. The network-level procedure assigns a uniform extreme-event likelihood across the bridge inventory and differentiates bridges using a bridge-specific likelihood-of-service-disruption score derived from a structured security checklist, such as the Rutgers Simple Bridge Security Checklist. AHP-derived weights combine the checklist criteria into an overall score on a 0–1 scale.

Explosive hazard scenario Potential explosive tactics considered under this hazard class may include vehicle-borne improvised explosive devices (VBIEDs), person-borne IEDs, and placed explosive devices (among other tactics).

Step-by-step procedure **Step 1**. Obtain the bridge inventory (GIS-enabled bridge layer / BMS inventory) as the baseline asset set for risk computation.

Step 2. Define the explosive hazard scenario and the disruption threshold in a way that is explicit, observable, and consistent across the inventory.

(2a) Choose the explosive/attack scenario ("hazard scenario" definition). In the Guidelines, a hazard scenario is an adverse event of a specified type/magnitude that causes a defined impact on transportation service. For terrorism/countering violent extremism, the report notes that many attack combinations are possible, so the analyst should select a small set of representative scenarios. The report lists the following likely

tactics/threats that can be used to define explosive-type scenarios (and closely related damage/disruption mechanisms):

- **Vehicle-borne improvised explosive device (VBIED):** landborne (e.g., truck bomb) and waterborne (e.g., boat bomb) vehicles targeting components reachable by land/water.
- **Hand-emplaced IED (HEIED):** contact explosive devices such as satchel charges and shaped charges used to cut/sever members.
- **Non-explosive cutting device (NECD):** saws, grinders, torches used to cut/sever members.
- **Vehicular impact (VI):** landborne/waterborne impact, analogous to VBIED access pathways.
- **Fire:** e.g., severe fire from a ruptured tanker truck on/near the bridge or adjacent to key components (including near piers/towers).

Select one or more agency screening scenarios, such as a vehicle-borne improvised explosive device at deck level causing closure or a hand-emplaced improvised explosive device against a critical member causing replacement. Apply the same scenario definitions to all bridges.

(2b) Specify the disruption threshold for "recognition" of the scenario. The report emphasizes that an important decision is the level of disruption that triggers the scenario in the model. Options discussed include:

- **Damage threshold:** structure damaged to at least a defined damage level (often aligned with the agency distinction between routine repairs and programmed capital work).
- **Cost threshold:** near-term or long-term life cycle costs are increased.
- **Service-disruption threshold (recommended emphasis):** transportation service is disrupted (loss of safety and/or mobility performance).
- **Environmental/property threshold:** environmental resources or others' property damaged.

The Guidelines recommend primary emphasis on **service disruption** as the defining criterion for hazard scenarios.

(2c) Make the service-disruption threshold operational. Translate "service disruption" into a measurable rule such as:

- **closure state** (full closure vs. partial restriction),
- **minimum duration** (e.g., ≥ 1 day, ≥ 1 week), or
- **action trigger** (e.g., detour required, emergency stabilization, monitoring period).

Step 3. Assign an annual extreme-event likelihood for the explosive hazard class. Because event likelihood is difficult to estimate bridge-by-bridge for terrorism-related explosives, define a uniform annual probability (or score) for the extreme-event occurrence across all bridges:

$$p_{\mathrm{EE}}^{\mathrm{expl}} \in [0, 1] \tag{A.8}$$

where the same value is applied to every bridge in the screened inventory.

In this report, $p_{\mathrm{EE}}^{\mathrm{expl}}$ is an agency-chosen baseline consistent with the extreme-event magnitude implied by the hazard scenario; for example, a 100-year design-level event corresponds to a probability of 1%. For terrorism and explosives, the guidance assigns a uniform extreme-event probability across the network. The

agency may establish the baseline through judgment-based methods such as polling or analogy, or set the extreme-event likelihood to 1.0 so that the bridge-specific service-disruption likelihood provides all network differentiation.

Step 4. Collect bridge security attributes using a structured checklist. For each bridge i, apply a bridge security checklist (e.g., the Rutgers Simple Bridge Security Checklist) to document security-relevant attributes (e.g., access control, surveillance, standoff opportunities, protective features, and operational procedures), using the checklist's defined scoring rubric (Figure A.1).

Step 5. Compute the likelihood of service disruption score from the checklist. Let $\{x_{ik}\}$ denote the scored checklist items for bridge i. Combine the items using the checklist's weighting method (e.g., AHP-derived weights $\{\alpha_k\}$) to produce an overall terrorism/explosive susceptibility (risk) score on a 0-1 scale:

$$S_i^{\text{expl}} = \sum_k \alpha_k \, x_{ik}, \qquad 0 \le S_i^{\text{expl}} \le 1 \tag{A.9}$$

Interpret S_i^{expl} as the bridge-specific likelihood of service disruption conditional on the extreme event, or as a normalized disruption-likelihood index suitable for ranking:

$$p_{\text{SD}}^{\text{expl}}(i) \equiv S_i^{\text{expl}} \tag{A.10}$$

Step 6. Compute an explosive risk index for screening and ranking. For bridge i, compute a screening-level risk index as the product of the extreme-event likelihood and the disruption-likelihood score:

$$\text{Risk}_{\text{expl}}(i) = p_{\text{EE}}^{\text{expl}} \cdot p_{\text{SD}}^{\text{expl}}(i) \tag{A.11}$$

Rank bridges in descending order of $\text{Risk}_{\text{expl}}(i)$ to identify comparatively higher-risk assets for more detailed review.

Step 7. Compute consequence dollars per disruption event (CQ worksheets) and form $C_{\text{expl}}^{A}(i)$ and $C_{\text{expl}}^{U}(i)$.

(a) Agency consequence: $C_{\text{expl}}^{A}(i)$ (Worksheet CQ–Cost). The report's CQ-Cost worksheet decomposes infrastructure/agency consequences into: incident response (e.g., traffic control, HazMat remediation), recovery (e.g., detour support, debris removal, temporary structures), and repair/replacement of the bridge. A common practice is to estimate replacement using a unit cost per square foot.

Define

$$C_{\text{expl}}^{A}(i) = C_i^{\text{IR}} + C_i^{\text{REC}} + C_i^{\text{TEMP}} + C_i^{\text{REP}} \tag{A.12}$$

where (consistent with CQ-Cost worksheet line items)

$$C_i^{\text{IR}} = \text{incident response costs (e.g., traffic control, HazMat remediation)}, \tag{A.13}$$

$$C_i^{\text{REC}} = \text{recovery costs (e.g., detour-related costs, debris removal)}, \tag{A.14}$$

$$C_i^{\text{TEMP}} = \text{temporary bridge cost (if used)}, \tag{A.15}$$

$$C_i^{\text{REP}} = \rho_i \, C_i^{\text{REPL}}, \qquad C_i^{\text{REPL}} = A_i \, c_i \tag{A.16}$$

Here A_i is the square footage of bridge to be repaired/replaced and c_i is the unit replacement cost (\$/sq.ft), which the worksheet allows as a range (low/high); ρ_i is the assessed percentage of replacement cost representing the repair/rebuild level for the scenario.

NCHRP 20-07 (378) Risk Analysis

Sheet LD - Terrorism

Bridge ID: **010001**

Hazard scenario: **FI-100a**

Answer the questions below for each bridge using field data or NBI data.

Questions Related to the Occurrence Factor		Answer	Weight	Score
1	Is there sufficient lighting on the superstructure?	Yes	0.1613	0.0000
2	Is there enough space around the bearings to place a 6"x6"x6" object?	Yes	0.0951	0.0951
3	Can someone park under/on bridge?	Yes	0.1720	0.1720
4	Is there a shoulder on the bridge?	Yes	0.0714	0.0714
5	Is there a sidewalk or a pedestrian walkway?	Yes	0.0605	0.0605
6	Is there easy access to the deck from underneath the bridge?	Yes	0.0923	0.0923
7	Is there an access to the bearings?	Yes	0.0856	0.0856
8	Is there easy access to the pile cap?	Yes	0.0654	0.0654
9	Is there easy access to the abutment and/or the wingwalls?	Yes	0.0450	0.0450
10	Are pipelines located under/over bridge?	Yes	0.0419	0.0419
11	Are gas pipes located under/over bridge?	Yes	0.0775	0.0775
12	Are power lines located under/over bridge?	No	0.0320	0.0000
		Total	**1.00**	**0.81**

Questions Related to the Vulnerability Factor		Answer	Weight	Score
1	Are the bearings securely anchored in place?	Yes	0.1278	0.0000
2	Is the pier/tower a single column, two-column, three-column or more than 3?	single	0.1784	0.1784
3	Do the pier columns have sufficient confinement?	No	0.1776	0.1776
15	Does roadway drain to beneath bridge?(gasoline fire under bridge from truck accident)	Yes	0.1046	0.1046
16	storage of vehicles under bridge or excessive garbage accumulation)	No	0.1324	0.0000
17	Is there a secure perimeter or zone around the bridge? Around certain bridge components?	No	0.1231	0.1231
18	What is the bridge sufficiency rating?	90	0.0655	0.0000
19	Is there a protection around the pier/tower?	No	0.0906	0.0906
		Total	**1.00**	**0.67**

Questions Related to the Importance Factor		Answer	Weight	Score
1	Is the bridge near or on route to high value target?	Yes	0.3624	0.3624
2	Is the bridge over or near chemical/refinery/industrial	No	0.2443	0.0000
3	What is the length of the longest span in feet?	3500	0.1250	0.1250
4	What is the annual average daily traffic of the bridge?	296900	0.1459	0.1459
5	Is the bridge part of a Coastal Evacuation Route?	No	0.1225	0.0000
6	Is the bridge culturally or historically significant? i.e. Is this structure a well known landmark?	Yes	1.0000	1.0000
		Total	**1.00**	**1.00**

Risk = O * V * I = 0.54

Figure A.1: Simple Bridge Security checklist
Source: [2, Appendix B]

(b) User/social consequence: $C^U_{\text{expl}}(i)$ (CQ-Safety, CQ-Mobility, CQ-Environment). The report provides explicit equations for monetized impacts to safety, mobility, and environment, each in \$/event, and these are the natural components of a user/social consequence term. A convenient aggregation is

$$C^U_{\text{expl}}(i) = C_i^{\text{SAF}} + C_i^{\text{MOB}} + C_i^{\text{ENV}} \tag{A.17}$$

Safety (Worksheet CQ-Safety). The report defines the safety consequence as:

$$C_i^{\text{SAF}} = 43{,}694 \times VC_i \tag{A.18}$$

where VC_i is the count of vehicles involved in the crash (per event). The coefficient 43,694 is the report's adopted social cost per vehicle in a crash (ACC\$), taken as an average from the AASHTO Red Book (2010) and updated to 2016 dollars using CPI (excluding insurance reimbursements to avoid double counting). For a worst-case sudden-collapse scenario, the report gives:

$$VC_i = \frac{A_i^{\text{veh}}}{24} \cdot \frac{1}{RS_i} \cdot \frac{L_i}{5280} \tag{A.19}$$

with A_i^{veh} = forecast vehicles/day on the affected roadway, RS_i = speed (mph), and L_i = length of affected roadway (ft).

Mobility (Worksheet CQ-Mobility). The mobility cost per event is given by:

$$C_i^{\text{MOB}} = A_i^{\text{veh}} \cdot \frac{DD_i}{24} \cdot \left(VOC \cdot DL_i \; + \; TT \cdot \frac{DL_i}{DS_i} \cdot VO \right) \tag{A.20}$$

where DD_i is disruption duration (hours), DL_i detour length (miles), DS_i detour speed (mph), VOC vehicle operating cost (\$/mile), TT value of time (\$/hour), and VO vehicle occupancy.

Environment (Worksheet CQ-Environment). The environmental cost per event is given by:

$$C_i^{\text{ENV}} = A_i^{\text{veh}} \cdot \frac{DD_i}{24} \cdot DL_i \cdot ED \tag{A.21}$$

where ED is emission damage cost (\$/vehicle-mile) obtained from tabulated values as a function of functional class and speed.

A.3 Fatigue Risk

A.3.1 Introduction

Bridge fatigue risk refers to the likelihood that a bridge will experience damage or failure due to fatigue, which is the progressive deterioration of structural components caused by repeated cyclic loading over time, rather than a single extreme event.

Representative Case Studies in Texas Fatigue can produce cracking in bridge components under repeated cyclic loading. The reviewed Texas cases document fatigue-related damage without complete bridge collapse.

- Fatigue Damage in IH-20 Bridges - Midland County, TX (1992)
  - The IH-20 Midland County bridge has experienced fatigue related problems due to the unintended interaction between the longitudinal girders and the cross-frame diaphragms. Fatigue cracks developed in web gaps and diaphragm members of IH-20 bridges due to load distribution issues and connection eccentricity [38].
- I-345 Bridge in Dallas - Fatigue Stress Investigations (2009)
  - Field tests and finite element analysis were conducted after fatigue cracks were found in critical sections of the I-345 bridge. The work documented the cracking, established a monitoring program, and evaluated proposed retrofits [39].
- Fatigue Cracking in Steel Bridges (2010)
  - Fatigue cracking was documented in several steel bridges built between the 1950s and 1980s, including the Fair Park bridges in Dallas, the IH-20 bridges in Midland County, the SH-225 bridges at Sims Bayou, the Intracoastal Canal Bridge in Jefferson County, and the IH-345 bridges in Dallas. These cases illustrate fatigue risk in aging steel bridge infrastructure [40].

A.3.2 Methods Reported in the Literature

A.3.2.1 FDOT Assessment Procedure: Fatigue Risk [1]

In this report, fatigue risk is implemented as a probability-based screening metric aimed primarily at steel bridges, where fatigue cracking is a dominant deterioration mechanism and is closely tied to fracture-critical concerns. The fatigue module estimates the likelihood of crack initiation from an AASHTO-based fatigue-life formulation using network-available inputs (bridge age, truck traffic and growth, and an assumed or assigned fatigue detail category and stress-range level). For statewide screening, conservative default assumptions are used when detailed connection information is not available. For the highest-likelihood bridges, the study recommends refining the governing detail category and stress range using more detailed records. Consequences are expressed in monetary terms by combining an agency repair cost model for typical fatigue crack repairs with a user-impact component representing traffic disruption during repairs, applied mainly to non-redundant/fracture-critical cases. The resulting outputs support ranking bridges either by cracking likelihood (to identify candidates for further investigation) or by expected economic impact (to prioritize mitigation and repair).

Step-by-step procedure **Step 1. Identify bridges of interest.** Create the candidate set of bridges for the fatigue screening by selecting, from the Pontis inventory, structures that are potentially susceptible to load-induced fatigue cracking and have the minimum traffic/geometry data required by the model. In practice, this includes (i) bridges with steel superstructures (and/or older steel bridges, which are more likely to contain fatigue-sensitive details), and (ii) any structures flagged for fracture-critical inspection (NBI Item 92), even if the basic material fields do not indicate steel. Exclude non-bridge structures and records with missing key inputs needed for the fatigue calculations (e.g., construction year, ADT/ADTT information, truck percentage, lanes/traffic direction), as well as retired bridges. Using these filters, the report obtained a candidate set of approximately 530 bridges for the fatigue screening analysis.

Step 2. Assign fatigue-detail parameters

Assume an initial (default) critical fatigue detail category (the report uses Category D unless better information is available). A fatigue detail category is the standard AASHTO classification of common steel bridge details for load-induced fatigue. This classification is tabulated in the AASHTO LRFD Bridge Design Specifications [41] (fatigue-detail category table for load-induced fatigue) and is used consistently in the fatigue evaluation procedures of the AASHTO Manual for Bridge Evaluation [42]. The term detail means a local feature such as a welded attachment, the end of a cover plate, a stiffener connection, a gusset-plate connection, or a hole/coped region, places where geometry and welding can create stress concentrations. AASHTO groups these details into categories (A, B, C, D, E, E'), where lower categories generally indicate poorer fatigue resistance (cracks initiate sooner) and higher categories indicate better resistance. Each category is associated with a standard fatigue S-N relationship used to predict fatigue life for that type of detail [41, 42]. At the network level, detailed information needed to identify the controlling fatigue detail is often not available in the Pontis/NBI inventory. The report therefore recommends using a default detail category (Category D), together with a conservative default stress range (e.g., $S = 5$ ksi), to perform an initial screening, and then refining the detail category and stress range for the highest-priority bridges using design/inspection records (e.g., Virtis files).

With the facility detail category selected, the following fatigue model parameters will be obtained from Table A.10:

- R_y^M (mean-life resistance factor): a model coefficient that represents the typical (central) fatigue resistance of the selected detail category. In the fatigue-life equation, it acts as a multiplicative "capacity" term: a larger R_y^M implies a longer predicted fatigue life (all else equal).
- R_y^E (evaluation-life resistance factor): a more conservative resistance coefficient for the same detail category, used to compute the evaluation fatigue life A_E. It is defined so that the resulting life corresponds to a lower-percentile (earlier) cracking time than the mean/median life, reflecting uncertainty and providing a screening-level safety margin.
- K_y (detail-category constant): a constant taken from AASHTO for the selected fatigue detail category. It represents the category's basic fatigue strength in the life equation: for the same traffic and stress range, a category with higher fatigue resistance has a larger K_y and therefore a longer predicted fatigue life.

Table A.10: Fatigue model parameters

Table 9.3. Fatigue model parameters

Fatigue Category	R_y^M	R_y^E	K_y (ksi^3)
A	2.8	1.7	2.50 E+10
B	2.0	1.4	1.20 E+10
B'	2.4	1.5	6.10 E+09
C	1.3	1.2	4.39 E+09
C'	1.3	1.2	4.39 E+09
D	1.6	1.3	2.20 E+09
E	1.6	1.3	1.10 E+09
E'	2.5	1.6	3.90 E+08

Source: [1, Table 9.3]

Also determine: C = stress-range cycles per truck passage and $S = \Delta f_{\text{eff}}$ = effective stress range. In the

fatigue-life equations, $S = \Delta f_{\text{eff}}$ denotes the effective stress range at the critical fatigue detail of a bridge. A stress range is the change in stress caused by a vehicle passage, from the minimum to the maximum value, i.e., $\Delta f = f_{\text{max}} - f_{\text{min}}$, and it is typically expressed in ksi (or MPa). The word "effective" means that, even though different trucks and lane positions can produce different stress ranges, the model uses one representative (equivalent) value S that captures the overall fatigue-damage effect for network-level screening.

The parameter C is the number of stress cycles per truck passage. It converts truck counts into stress-cycle counts because a single truck crossing does not always create only one perfectly clean "up–down" cycle in the stress history. Depending on axle groups, vehicle configuration, bridge dynamics, and multi-span effects, one truck can produce more than one significant stress cycle. Thus, the model approximates the annual number of stress cycles as $365\,C\,(ADTT)_{SL}$, where $(ADTT)_{SL}$ is the average daily truck traffic in the critical (single) lane. In short, S measures how large the repeated stress fluctuation is at the crack-prone detail, while C measures how many fatigue cycles each truck crossing contributes.

Step 3. Compute traffic growth factor and truck volumes

Compute the traffic growth factor g and compute intermediate model quantities (e.g., T_1, and other terms defined in the fatigue likelihood equations). Let ADT_t denote the total average daily traffic in year t, and let t_0 and t_1 be two years for which traffic estimates are available (e.g., a base year and a future year). Define the average annual traffic growth factor as

$$g = \left(\frac{ADT_{t_1}}{ADT_{t_0}}\right)^{\frac{1}{t_1 - t_0}} \tag{A.22}$$

Let t_b be the bridge construction year. Backcast the opening-year traffic as

$$ADT_b = \frac{ADT_{t_0}}{g^{\,t_0 - t_b}} \tag{A.23}$$

Let π_T denote the truck percentage (as a fraction, not percent), and let SLF denote the single-lane factor. The opening-year annual single-lane truck volume (trucks/year) is then

$$T_b = 365\,(ADT_b\,\pi_T)\,SLF \tag{A.24}$$

Step 4. Compute fatigue cracking probability

The base AASHTO fatigue-life equation (years) [42] is

$$Y = \frac{R_y K_y}{365\,C\,(ADTT)_{SL}\,S^3} \tag{A.25}$$

This equation estimates the fatigue life Y (in years) of the controlling steel detail under repeated truck loading. It converts truck traffic and stress range into an expected time to fatigue cracking: higher stress range S or higher truck traffic $(ADTT)_{SL}$ shortens Y, while larger resistance factors R_y and detail constants K_y increase Y.

In fatigue evaluation, the predicted cracking (fatigue) life is uncertain due to variability in the as-built detail geometry and workmanship, material properties, and the actual stress ranges and traffic loading experienced

by the bridge. Rather than modeling each source of uncertainty explicitly, the AASHTO Manual for Bridge Evaluation (MBE) accounts for the overall scatter in fatigue life by providing tabulated fatigue resistance factors R_y for different life levels. The same fatigue-life equation is evaluated using (i) a mean/median-life resistance factor R_y^M to obtain the mean/median fatigue life A_M (about the 50th-percentile life), and (ii) an evaluation-life resistance factor R_y^E to obtain an evaluation fatigue life A_E (a lower-percentile life; often taken as approximately the 16th percentile). These life levels provide a standardized way to express the uncertainty (scatter) in fatigue life within the AASHTO evaluation framework [42].

Using the AASHTO MBE mean/median-life resistance factor R_y^M for the selected fatigue detail category, compute the mean (median) fatigue life A_M as

$$A_M = \begin{cases} \dfrac{R_y^M K_y}{C\, T_1\, S^3}, & g = 1, \\ \dfrac{\ln\left(1 + \dfrac{R_y^M K_y\,(g-1)}{T_1\, C\, S^3}\right)}{\ln(g)}, & g \neq 1. \end{cases} \tag{A.26}$$

Using the AASHTO MBE evaluation-life resistance factor R_y^E for the same detail category, compute the evaluation fatigue life A_E as

$$A_E = \begin{cases} \dfrac{R_y^E K_y}{C\, T_1\, S^3}, & g = 1, \\ \dfrac{\ln\left(1 + \dfrac{R_y^E K_y\,(g-1)}{T_1\, C\, S^3}\right)}{\ln(g)}, & g \neq 1. \end{cases} \tag{A.27}$$

Once the mean/median fatigue life A_M and the evaluation fatigue life A_E are computed, this report then treats the actual crack-initiation age as uncertain and models it probabilistically. Specifically, A_M is taken as the median (50th-percentile) crack-initiation life, while A_E is defined one log-standard-deviation earlier than A_M, i.e., $\ln(A_E) = \ln(A_M) - \sigma$.

Let T denote the bridge age at which a fatigue crack first initiates. Define the cracking probability at evaluation age A as $P_f(A) \triangleq P(T \leq A)$. Assume T is lognormally distributed, i.e. $\ln T \sim \mathcal{N}(\mu, \sigma^2)$. Let A_M be the median (50th percentile) of T, so $P(T \leq A_M) = 0.5$. Then $0.5 = P(T \leq A_M) = P(\ln T \leq \ln A_M)$. For a normal random variable, the median equals the mean, hence $P(\ln T \leq \mu) = 0.5$. Therefore,

$$\mu = \ln A_M \tag{A.28}$$

Let A_E be defined as one log-standard-deviation earlier than A_M, meaning $\ln A_E = \mu - \sigma$. Substituting $\mu = \ln A_M$ gives

$$\ln A_E = \ln A_M - \sigma \quad \Rightarrow \quad \sigma = \ln A_M - \ln A_E = \ln\left(\frac{A_M}{A_E}\right) \tag{A.29}$$

Starting from the definition,

$$P_f(A) = P(T \le A) = P(\ln T \le \ln A) \tag{A.30}$$

$$= P\left(\frac{\ln T - \mu}{\sigma} \le \frac{\ln A - \mu}{\sigma}\right) \tag{A.31}$$

Define the standardized variable $Z \triangleq \frac{\ln T - \mu}{\sigma}$. Since $\ln T \sim \mathcal{N}(\mu, \sigma^2)$, it follows that $Z \sim \mathcal{N}(0,1)$. By definition of the standard normal CDF $\Phi(\cdot)$, $P(Z \le x) = \Phi(x)$. Hence $P_f(A) = \Phi\left(\frac{\ln A - \mu}{\sigma}\right)$. Substitute $\mu = \ln A_M$ and $\sigma = \ln(A_M/A_E)$:

$$P_f(A) = \Phi\left(\frac{\ln A - \ln A_M}{\ln(A_M/A_E)}\right) = \Phi\left(\frac{\ln(A/A_M)}{\ln(A_M/A_E)}\right) \tag{A.32}$$

Step 5. Screen and rank by likelihood. Sort bridges by $P_f(A)$ in descending order to identify high-priority bridges for more detailed investigation.

Step 6. Refine critical detail category and stress range (for top bridges). For the highest-priority bridges, consult Virtis / load rating files to refine: (i) the critical fatigue detail category and (ii) the effective stress range at the critical detail. Also review the fracture criticality indicator, because it controls whether user impacts are applied.

Step 7. Recompute with refined inputs. Repeat Steps 2-5 using the refined fatigue category / stress range data.

Agency consequence (repair cost). Once cracking occurs, FDOT practice is to schedule repair within a reasonable time frame. The network-level procedure represents repair through a typical repair-cost model based on NCHRP 495 guidance and Pontis attributes, including detail category, truss classification, and number of spans. Let $C_{\text{rep}}(i)$ denote the conditional annual agency repair cost given that at least one crack is observed (see Table A.11).

Table A.11: Typical fatigue repairs on steel superstructures from NCHRP 495

Table 9.4. Typical fatigue repairs on steel superstructures from NCHRP 495

Repair	Detail	Detail category	Found on trusses	Found on non-trusses	Repair cost ($/crack)	Range of cost impact factors
B1	Welds of partial-length coverplates	E	N	Y	775	0.71-1.64
B2	Transverse connection plates welded to the tension flange	E	Y	Y	2445	0.75-2.03
B3	Welded transverse stiffener	C	N	Y	340	1.0
B4	Termination of welded longitudinal web stiffener	B	N	Y	360	1.0
B5A	Welded longitudinal connection plates - bottom flange	E	Y	Y	3415	0.75-2.03
B5B	Welded longitudinal connection plates - web	E	Y	Y	4015	0.75-2.03
B6	Riveted truss members	D	Y	N	1075	0.75-1.2
B7	Riveted built-up girder or floorbeam	D	N	Y	1380	0.51-2.7

Source: [1, Table 9.4]

Then the expected annual agency cost is:

$$\text{AgencyCost}_{\text{fatigue}}(i) = p_{\text{fatigue}}(i)\; C_{\text{rep}}(i) \tag{A.33}$$

User impact (traffic disruption cost). The report recommends applying user impacts primarily to fracture-critical bridges: during repairs, trucks are assumed to detour for a duration of 8 days, and user cost is computed using operating cost per mile and value of time, together with detour length and speed from Pontis roadway fields. Define the conditional user cost (given cracking/repair) as

$$C^U_{\text{rep}}(i) = \mathbb{I}_{\text{FC}}(i) \underbrace{ADTT_i \left(c_{\text{op}} \ell_i + c_{\text{time}} \frac{\ell_i}{v_i} \right) \cdot d}_{\text{8-day detour for trucks during repair}} \tag{A.34}$$

where $\mathbb{I}_{\text{FC}}(i)$ is 1 if bridge i is fracture-critical (else 0), ℓ_i is detour length (miles), v_i detour speed (mph), $d = 8$ days, and $ADTT_i$ is average daily truck traffic for the analysis year. Then the expected annual user cost is:

$$\text{UserCost}_{\text{fatigue}}(i) = p_{\text{fatigue}}(i)\ C^U_{\text{rep}}(i) \tag{A.35}$$

Total fatigue risk (social cost) and ranking. The final fatigue risk metric recommended by the report is the expected social cost:

$$\text{TotalCost}_{\text{fatigue}}(i) = p_{\text{fatigue}}(i) \left(C_{\text{rep}}(i) + C^U_{\text{rep}}(i) \right) \tag{A.36}$$

which corresponds to the report's final risk model (Eq. 9-18), using cracking probability, repair cost, and user cost. Bridges can be ranked by (i) $p_{\text{fatigue}}(i)$ for screening priority, or (ii) $\text{TotalCost}_{\text{fatigue}}(i)$ to prioritize by economic impact.

A.3.2.2 MnDOT report [4]

The MnDOT Report 2007-22 proposes a network-level screening procedure to enumerate and gross-rank steel bridges that are susceptible to fatigue (and fracture) cracking. Instead of performing project-level fatigue analysis, the method assigns each bridge a composite rank number based on the presence and geometry of a fixed set of flagged details. The composite rank serves as a ranking-based risk index for prioritization (higher rank $\Rightarrow$ higher susceptibility and/or multiple problematic details).

Core concept. Let $\mathcal{B}$ be the statewide bridge set. Define a fixed library of $K = 18$ flagged detail types, indexed by $k \in \{1, \ldots, K\}$. For bridge $i \in \mathcal{B}$, each detail type k is assigned a rank

$$r_{ik} \in \{0, 1, 2, 3, 4\} \tag{A.37}$$

where $r_{ik} = 0$ indicates that detail type k is not present (or not applicable), and larger values indicate qualitatively higher severity and/or consequence of cracking.

For each detail type k that is present on bridge i, a vector of geometric descriptors $\mathbf{x}_{ik}$ is collected (from plans and/or existing inspection records), and the rank is computed via a rule-based mapping

$$r_{ik} = f_k(\mathbf{x}_{ik}) \tag{A.38}$$

where $f_k(\cdot)$ is defined by the report's classification table and implemented as rule logic in an Excel-based program (nested `IF/ELSE` logic).

Bridge-level fatigue risk index (composite rank). The bridge-level ranking-based fatigue risk index is defined as the sum of the ranks for the worst (most severe) instanceof each detail type:

$$R_i = \sum_{k=1}^{K} r_{ik} \tag{A.39}$$

The theoretical range is $0 \leq R_i \leq 54$, where $R_i = 0$ denotes a bridge with none of the flagged detail types, and $R_i = 54$ denotes a bridge containing the worst-ranked configuration for all 18 detail types.

Step-by-step procedure (network-level screening) Step 1. Bridge inventory. Compile the statewide bridge inventory $\mathcal{B}$ (unique bridge identifiers, location, structure type, and links to available plans / inspection records). Restrict the screening population to steel bridges (or the subset intended for fatigue screening).

Step 2. Flagged detail library (18 types). Adopt the report's fixed list of $K = 18$ flagged fatigue/fracture-susceptible detail types (Figure A.2). For each type k, use the report's classification table that provides: (i) a description and identification aid, (ii) geometric sub-categories, and (iii) the rank assignment for each geometric category.

Step 3. Desk-based data collection. For each bridge $i \in \mathcal{B}$, review available bridge plans and existing inspection records to:

- determine whether each flagged detail type k is present (Yes/No);
- if present, extract the required geometric inputs $\mathbf{x}_{ik}$ for that detail type.

(If field-only details exist in the library, they are treated as "unknown" under a no-field-visit constraint; see Step 10.)

Step 4. Worst-case selection within each detail type. If bridge i contains multiple instances of the same detail type k, identify the most severe geometry (the instance expected to yield the highest rank) and define $\mathbf{x}_{ik}$ from that worst-case instance only. This produces a single rank r_{ik} per detail type.

Step 5. Select detail ranks using classification rules. For each present detail type k, select

$$r_{ik} = f_k(\mathbf{x}_{ik}) \tag{A.40}$$

where $f_k(\cdot)$ is the rule mapping implied by the classification table (Figure A.2).

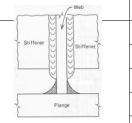
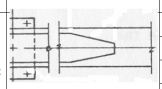
Step 6. Compute bridge composite rank. Compute the bridge-level composite rank (ranking-based fatigue risk index):

$$R_i = \sum_{k=1}^{K} r_{ik} \tag{A.41}$$

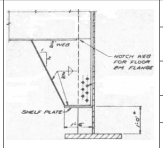
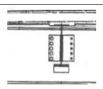
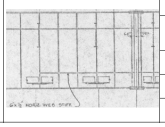
with $r_{ik} = 0$ for absent / not-applicable detail types.

Step 7. Database-ready output. Create an output record for each bridge i that includes:

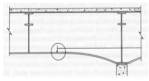
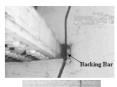
$$\texttt{BridgeID},\ R_i,\ r_{i1},\ r_{i2},\ \ldots,\ r_{iK} \tag{A.42}$$

Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Web Gap on Diaphragm Transverse Stiffener (excluding intermittent stiffener)	Gap usually from tight fit to 2" between vertical stiffener and bottom or top flange of girder where no welded or bolted connection exists		1989	top flange, *tw*>0.50", web gap<2", *L*<80'	2
			1989	top flange, *tw*>0.25", web gap<1", *L*<80'	2
			1989	top flange, *tw*>0.50", web gap<2", *L*<160', *skew* > 40º	2
			1989	top flange, *tw*>0.25", web gap<1", *L*<160', *skew* > 40º	2
			1989	bottom flange (positive moment region)	1
			1989	Otherwise	1
Insufficient Cope Radius	Re-entrant cope radius is nonexistent, too small, or not ground smooth.		1964 MN	Over-cut or square-cut	2
			1964 MN	Transition radius < 1"	1
			xxxx	Transition radius ≥ 1"	0
Partial Length Cover Plate	Plates welded to top or bottom girder flange to increase moment capacity locally.		1983r AASHTO	Flange thickness > 0.8"	3
			1986 MN	No end taper, wider than flange, no weld across end	3
			1986 MN	No end taper, other cases	2
			1986 MN	End taper	1

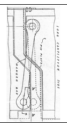
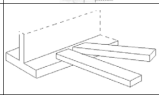
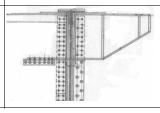
(a) Part 1

Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Shelf Plate Welded to Girder Web	Horizontal plate welded to inside face of girder web used for floor beam or lateral bracing connections. Intersecting plate is either coped around or welded to shelf plate.		2002 MN	Intersecting plate, cope and weld not compliant with *Mn/DOT Std*	4
			1989r AASHTO	Intersecting plate, cope and weld compliant with *Mn/DOT Std*, (*Lp*>12*tp* or 4") or *R*<6"	3
			xxxx	Intersecting plate, cope and weld compliant with *Mn/DOT Std*, (*Lp*≤12*tp* & 4") and *R*≥6"	2
			1989r AASHTO	No intersecting plate, (*Lp*>12*tp* or 4") or *R*<6"	2
			xxxx	No intersecting plate, (*Lp*≤12*tp* and 4") and *R*≥6"	1
Stringer or Truss Floor-beam Bracket	Angle bracket connecting stringers to floor beams or floor beam to truss.		xxxx	Floor beam to truss	3
			xxxx	Stringer to floor beam	1
Welded Horizontal Stiffener	Horizontal plates welded continuously to girder webs.		1960r Fisher	Continuation through tension zone	4
			1960r Fisher	Multiple plates welded end-to-end or vertical stiffener interrupts horizontal stiffener	3
			1989r AASHTO	*R*=0, *tp*>1"	3
			1989r AASHTO	*R*<2"	2
			xxxx	*R*≥2"	1

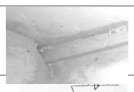
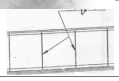
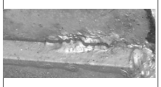
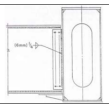
(b) Part 2

Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Haunch Insert	Bottom girder flange and part of web removed and section added to increase girder depth locally. Welded.		1989 AASHTO	Transverse *PJP* groove welds	3
			xxxx	No transverse *PJP* groove welds	2
Web Penetration	A hole cut through the web of member to allow another plate or member to pass through.		xxxx	Hole cut in box girder except at pier cap	3
			xxxx	Intersecting girders at pier cap	2
			xxxx	Hole cut in open girder	1
Plug-Welded Misplaced Hole	Misplaced holes filled by plug welding, regardless of location.		xxxx		4
Field-Welded Splice	Splice between girders, beams or girder to beam which was done by local bridge owners.		xxxx		4

(c) Part 3

Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Pin and Eyebar Truss and Pin and Hanger Assembly	Eyebars or pin plates used as tension members.		1989 P		4
Lateral Bracing to Girder Bottom Flange	Lateral bracing such as cross-braces or diaphragms welded to the bottom flange of girders and not the web.		1983r 1989 AASHTO		3
Cantilever Floor-beam Bracket	Plate lain horizontally over beam top flange cantilevered to side. Bolted or welded to top flange of each member.		1986 W		3

(d) Part 4

Detail Name	Description	Picture	Date of code prohibition	Geometric Partitions	Rank #
Backing Bar	Bars or plates left on bridges used for supporting back of welds, not ground smooth.		1978 MN		3
Intermittent Weld	Welding along a section such as vertical stiffener that is not continuous.		xxxx		3
Tack Weld	Bolts, plates or any other members that were tack-welded into place without removing welds afterwards.		1970 P		2
Tied Arch Floor-beam	Floor-beam shear connection to tied arch girder web		xxxx		1
A514 Steel (T1 steel)	Low fracture toughness steel		1989r AASHTO		3

(e) Part 5

Figure A.2: Overall Classification Table
Source: [4, Table 7.1]

optionally accompanied by the underlying geometric inputs $\mathbf{x}_{ik}$ (to support auditing, future refinement, and identification of the worst-case instance of each type).

Step 8. Network ranking and screening list. Rank bridges in descending order of R_i:

$$\text{Priority order: } i \succ j \quad \Leftrightarrow \quad R_i > R_j \tag{A.43}$$

Define a screening cutoff (e.g., top $X\%$ or R_i exceeding a threshold) to generate a prioritized list for further desk-based review, special inspection planning, or retrofit scoping.

A.4 Fire-Related Risk

A.4.1 Introduction

Bridge fire risk refers to the likelihood and consequences of a bridge being exposed to damaging thermal actions, whether from (i) wildfires and wildland: urban interface (WUI) fires that impinge on the superstructure, substructure, and approaches, or (ii) localized "ordinary" fires such as vehicle/tanker crashes, arson, encampment fires, or ignition of combustibles stored beneath elevated structures. In both pathways, rapid temperature rise and sustained heating can degrade steel strength and stiffness, trigger concrete spalling and loss of section, damage bearings and joints, and in extreme cases precipitate partial or global collapse. Beyond structural damage, bridge fires also generate severe network-wide impacts through emergency closures, detours, and long recovery times.

Recent incidents document bridge damage from wildfire-driven and accidental or operational fires Table A.12.

Recognizing these risks, U.S. federal and state agencies have issued guidance spanning prevention, inspection, and resilience investment. Federally, the National Transportation Safety Board (NTSB) has issued a safety alert on "Protecting Bridges From Fire Damage and Collapse" [50]. FHWA further issued a memorandum directing that discovery of flammable/explosive/hazardous materials stored under highway bridges be treated as a critical finding requiring immediate follow-up and reporting (per 23 CFR 650.313(q)) [51]. Several states have translated such federal direction into inspection practice. For instance, Michigan DOT released a bridge advisory updating its inspection manual to explicitly flag flammable materials stored beneath bridges as a critical-finding hazard [52]. For wildfire-driven exposure, state transportation agencies have also advanced corridor-level mitigation planning (e.g., prioritizing fuels reduction/defensible space along vulnerable highway corridors), as reflected in Caltrans' wildfire adaptation program describing a Wildfire Vulnerability Analysis used to guide fuels-reduction priorities [53].

Representative Case Studies in Texas

- Regency Bridge Wooden Deck Fire - Mills County, TX (2003)
  - Local teenagers accidentally set fire to the wooden deck of the Regency Bridge over the Colorado River in Mills County. The fire burned holes in the decking, causing structural damage estimated at about 20,000 USD. The bridge, an historic one-lane suspension bridge, was closed for repairs and reopened in early 2005 [54].
- San Saba Railroad Bridge Collapse Due to Fire (2013)

Table A.12: Selected U.S. bridge/overpass fire incidents (2021-2025)

Date	Bridge / Structure	Location	Fire origin (reported)	Impact (reported)	Ref.
2021-03-07	Flyover connecting E. Ben White Blvd to I-35 (TxDOT flyover)	Austin, TX	Homeless encampment fire beneath flyover	Bridge reported damaged; traffic impacts while assessed/repairs planned	[43]
2021-11 (month)	Cheshire Bridge Rd overpass (near Buford Connector)	Atlanta, GA	Fire under overpass attributed to homeless-encampment activity	Reported as a notable underbridge fire incident in the area	[44]
2022-01-13	I-70 at 18th St bridge area (downtown)	Kansas City, MO	Fire in a homeless encampment under the bridge	"Significant" heat damage reported to bridge components; westbound I-70 closures/repairs	[45]
2023-11-11	I-10 freeway overpass section near downtown (storage-yard beneath)	Los Angeles, CA	Large fire under I-10; authorities said arson; flammable materials stored beneath	Mile-long closure; extensive damage described (numerous columns affected)	[46]
2024-05-02	Fairfield Ave overpass over I-95	Norwalk, CT	Gasoline tanker crash; vehicle burst into flames	Overpass heavily damaged and subsequently demolished; major I-95 closure and rapid rebuild effort	[25]
2024-11-01	Daniel Carter Beard Bridge (I-471, "Big Mac Bridge")	Cincinnati, OH / N. Kentucky	Fire near/under bridge by playground materials (mulch/plastic/wood)	Fire warped steel beams; bridge partially closed for weeks; repairs required	[47]
2025-07-07	Blackwell Street Bridge (near Route 62)	Barre, VT	Fire started in an encampment under bridge girders	Bridge closed overnight for inspection; heat effects reported (e.g., paint damage)	[48]
2025-11-02	Loyola Ave on-ramp to US-90 Business (Pontchartrain Expwy / Crescent City Connection approach)	New Orleans, LA	Homeless encampment fire beneath ramp	Ramp closed indefinitely pending structural evaluation; reported girder/deck damage concerns	[49]

  - On May 19, 2013, a long wooden railroad trestle bridge caught fire. The blaze was so intense that firefighters determined it was too dangerous to fight directly and allowed it to burn. The bridge collapsed as a direct result of the fire, with video recording showing the structure collapse in flames [55].

- Big Piney River Bridge (Baptist Camp Access) (2022)
  - It was discovered that wood debris beneath the bridge had been on fire for hours, severely damaging structural elements (concrete and steel components). Although this did not collapse the bridge, the structural damage was significant enough to close the bridge indefinitely and prompt detailed repair planning [56].

- Historic Bryant Station Bridge Fire - Milam County, TX (2025)
  - A historic timber and metal Bryant Station Bridge over the Little River in Milam County caught fire late one night in 2025. Fire crews from multiple agencies responded, but due to the bridge's wooden, heavily creosote-treated structure and deteriorated condition, some areas continued to smolder [57].

A.4.2 Methods Reported in the Literature

A.4.2.1 FDOT Wildfire Assessment [1]

In the FDOT report, wildfire risk for each bridge is modeled as the product of a likelihood term derived from historical wildfire occurrence and a consequence/vulnerability term that reflects how susceptible the bridge is to wildfire damage and disruption.

(1) Wildfire likelihood is estimated from a detailed historical wildfire dataset (1980-2010) obtained from the Florida Department of Forestry, where wildfire events are spatially associated with bridges using GIS and a 1-mile buffer around each bridge location; an alternative county-level approach using NOAA wildfire records is also discussed. Under a Poisson-process assumption for event occurrence, the historical count within the observation window is converted to an annual rate λ, and then to an annual probability of at least one nearby wildfire via $p = 1 - e^{-\lambda}$.

(2) Consequences are framed in terms of both agency impacts (repair/recovery) and user impacts (traffic disruption), with the report emphasizing that wildfire effects are often dominated by road closures and user costs rather than total structural destruction.

(3) Vulnerability is represented through bridge attributes: (i) element-level susceptibility, where timber-related and certain non-structural/movable elements are identified as highly vulnerable, and (ii) bridge-type/material susceptibility, which shows timber superstructures as most vulnerable, followed by steel, then concrete, with a note that continuous superstructures may transmit wildfire effects more rapidly.

Step-by-step procedure **Step 1**. Obtain the bridge inventory (FDOT/Pontis bridge GIS layer) as the baseline asset set for risk computation.

Step 2. Obtain historical wildfire data from the Florida Department of Forestry for 1980–2010, including both GIS shapefiles and tabular database records.

Step 3. Perform a spatial overlay (spatial join) in ArcGIS between the wildfire GIS layer and the FDOT bridge GIS layer.

Step 4. For each bridge, create a 1-mile buffer zone around its location (the report notes that the 1-mile distance is chosen arbitrarily). Identify wildfire events within the 1-mile buffer of each bridge and assign those events to the corresponding bridge.

Step 5. For each bridge, count the number of wildfire occurrences within the buffer during the observation period, yielding the historical event frequency near that bridge.

Step 6. Convert the event count to an annual occurrence rate λ (the Poisson intensity) by dividing by the observation duration (30 years for 1980–2010).[1]

Step 7. Assume wildfires follow a Poisson process. Convert λ to the annual probability of at least one occurrence:

$$p_{\text{wildfire}} = 1 - e^{-\lambda} \tag{A.44}$$

Step 8. Compute the wildfire risk index by multiplying the annual probability by three vulnerability multipliers, and rank bridges in descending order of $\text{Risk}_{\text{wildfire}}$:

$$\text{Risk}_{\text{wildfire}} = p_{\text{wildfire}} \cdot V^{\text{elem}}_{\text{wildfire}} \cdot V^{\text{age}}_{\text{wildfire}} \cdot V^{\text{des}}_{\text{wildfire}} \tag{A.45}$$

The calculations of $V^{\text{elem}}_{\text{wildfire}}, V^{\text{age}}_{\text{wildfire}}, V^{\text{des}}_{\text{wildfire}}$ are shown below:

$V^{\text{elem}}_{\text{wildfire}}$ is the wildfire element vulnerability. For each bridge, let $\mathcal{E}$ denote the set of bridge elements recorded for that bridge. For each element e, obtain its wildfire element score $s_{\text{wildfire}}(e)$ from Table A.13, where $s_{\text{wildfire}}(e) \in \{0, 1, 2, 3, 4, 5\}$. The bridge-level raw element score is the simple average:

$$\bar{s}_{\text{wildfire}} = \frac{1}{|\mathcal{E}|} \sum_{e \in \mathcal{E}} s_{\text{wildfire}}(e) \tag{A.46}$$

The reported element vulnerability multiplier is obtained by rescaling the average score:

$$V^{\text{elem}}_{\text{wildfire}} = \frac{\bar{s}_{\text{wildfire}} + 1}{3} \tag{A.47}$$

$V^{\text{age}}_{\text{wildfire}}$ is the wildfire age vulnerability. Let Y be the year the bridge was built. The wildfire age vulnerability multiplier is modeled as a linear function of bridge age:

$$V^{\text{age}}_{\text{wildfire}} = 1 + \frac{1990 - Y}{35} \tag{A.48}$$

$V^{\text{des}}_{\text{wildfire}}$ is the wildfire design vulnerability. For a given bridge, identify its superstructure design type and material (as coded in the bridge inventory). The report assigns a wildfire design score $s^{\text{des}}_{\text{wildfire}}$ using Table A.14, which provides a discrete rating (0–5) for each (design type, material) combination. Then

$$V^{\text{des}}_{\text{wildfire}} = \frac{s^{\text{des}}_{\text{wildfire}} + 1}{3} \tag{A.49}$$

[1] The report also presents an alternative approach to estimate λ using county-level wildfire records from NOAA (1996–2010): compute a county-level λ and then use the same probability formula.

Table A.13: Levels of vulnerability of bridge elements to damage from wildfires

Table 4.2. Levels of vulnerability of bridge elements to damage from wildfires

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	5	476	Timber Walls	5
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	4	477	Other Walls	4
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	2
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	2	480	Mast Arm Found	2
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	2	481	Paint Mast Arm Vert	2
31	Timber Deck	5	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	2
32	Timber Deck/AC Ovly	5	234	R/Conc Cap	1	483	Other Mast Arm Vert	2
38	Bare Concrete Slab	1	235	Timber Cap	5	484	Paint Mast Arm Horzn	2
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	2	485	Galvan Mast Arm Horz	2
54	Timber Slab	5	241	Concrete Culvert	1	486	Other Mast Arm Horzn	2
55	Timber Slab/AC Ovly	5	242	Timber Culvert	5	487	Sign Member Horiz	2
98	Conc Deck on PC Pane	1	243	Misc Culvert	4	488	Sign Member Vertical	2
99	PS Conc Slab	1	290	Channel	2	489	Sign Foundation	2
101	Unpnt Stl Box Girder	2	298	Pile Jacket Bare	2	495	Uncoat High Mast L.	2
102	Paint Stl Box Girder	2	299	Pile Jacket/Cath Pro	2	496	Painted High Mast L.	2
104	P/S Conc Box Girder	1	300	Strip Seal Exp Joint	4	497	Galvan. High Mast L.	2
105	R/Conc Box Girder	1	301	Pourable Joint Seal	4	498	Other High Mast L.P.	2
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	4	499	H. M. L. P. Found.	2
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	4	540	Open Gearing	3
109	P/S Conc Open Girder	1	304	Open Expansion Joint	4	541	Speed Reducers	3
110	R/Conc Open Girder	1	310	Elastomeric Bearing	4	542	Shafts	3
111	Timber Open Girder	5	311	Moveable Bearing	4	543	Shaft Brgs and Coupl	3
112	Unpnt Stl Stringer	2	312	Enclosed Bearing	4	544	Brakes	3
113	Paint Stl Stringer	2	313	Fixed Bearing	4	545	Emergency Drive	3
115	P/S Conc Stringer	1	314	Pot Bearing	4	546	Span Drive Motors	3
116	R/Conc Stringer	1	315	Disk Bearing	4	547	Hydraulic Power Unit	3
117	Timber Stringer	5	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	3
120	U/Stl Thru Truss/Bot	2	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	3
121	P/Stl Thru Truss/Bot	2	330	Metal Rail Uncoated	3	550	Hopkins Frame	3
125	U/Stl Thru Truss/Top	2	331	Conc Bridge Railing	2	560	Locks	3
126	P/Stl Thru Truss/Top	2	332	Timb Bridge Railing	5	561	Live Load Shoes	3
130	Unpnt Stl Deck Truss	2	333	Other Bridge Railing	4	562	Counterweight Suppor	3
131	Paint Stl Deck Truss	2	334	Metal Rail Coated	3	563	Acc Ladd & Plat	3
135	Timber Truss/Arch	5	356	Steel Fatigue SmFlag		564	Counterweight	3
140	Unpnt Stl Arch	2	357	Pack Rust Smart Flag		565	Trun/Str and Cur Trk	3
141	Paint Stl Arch	2	358	Deck Cracking SmFlag		570	Transformers	3
143	P/S Conc Arch	1	359	Soffit Smart Flag		571	Submarine Cable	3
144	R/Conc Arch	1	360	Settlement SmFlag		572	Conduit & Junc. Box	3
145	Other Arch	4	361	Scour Smart Flag		573	PLCs	3
146	Misc Cable Uncoated	2	362	Traf Impact SmFlag		574	Control Console	3
147	Misc Cable Coated	2	363	Section Loss SmFlag		580	Navigational Lights	3
151	Unpnt Stl Floor Beam	2	369	Sub.Sect Loss SmFlag		581	Operator Facilities	3
152	Paint Stl Floor Beam	2	370	Alert Smart Flag		582	Lift Bridge Spec. Eq	3
154	P/S Conc Floor Beam	1	386	Fender/Dolphin Uncoa	2	583	Swing Bridge Spec. E	3
155	R/Conc Floor Beam	1	387	P/S Fender/Dolphin	2	590	Resistance Barriers	3
156	Timber Floor Beam	5	388	R/Conc Fender/Dolphi	1	591	Warning Gates	3
160	Unpnt Stl Pin/Hanger	2	389	Timber Fender/Dolphi	5	592	Traffic Signals	3
161	Paint Stl Pin/Hanger	2	390	Other Fender/Dolphin	4			
201	Unpnt Stl Column	2	393	Blkhd Sewl Metal Unc	1			
202	Paint Stl Column	2	394	R/Conc Abut Slope Pr	1			
204	P/S Conc Column	1	395	Timber Abut Slope Pr	5			
205	R/Conc Column	1	396	Other Abut Slope Pro	4			
206	Timber Column	5	397	Drain. Syst Metal	3			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	3			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	4			
211	Other Mtl Pier Wall	4	474	Walls Uncoated	4			
215	R/Conc Abutment	1	475	R/Conc Walls	1			

Source: [1, Table 4.2]

Table A.14: Levels of vulnerability of bridge types to damage from wildfires [1]

Table 4.3. Levels of vulnerability of bridge types to damage from wildfires

Superstructure design type	Concrete	Concrete Continuous	Steel	Steel Continuous	Prestressed concrete	Prestressed concrete continuous	Wood or Timber	Masonry	Aluminum, Wrought Iron, or Cast Iron
Slab	3	3			3	3	5		
Stringer/MultiBeam/Girder	3	3	4	4	3	3	5		
Girder & Floorbeam	3	3	4	4	3	3	5		
Tee Beam	3	3	4	4	3	3			
Box Beam or Girders - Multiple	3	3			3	3			
Box Beam/Girders - Single or Spread	3	3			3	3			
Frame (except frame culverts)									
Orthotropic									
Truss - Deck			4	4					
Truss - Thru			4	4					
Arch - Deck								3	
Arch - Thru								3	
Suspension			4	4					
Stayed Girder			4	4					
Movable - Lift	3	3	4	4					
Movable - Bascule	3	3	4	4					
Movable - Swing	3	3	4	4					
Tunnel									
Culvert									
Mixed Types									
Segmental Box Girder	3	3			3	3			
Channel Beam			4	4					

Source: [1, Table 4.3]

Agency cost. First, compute the replacement cost for bridge i:

$$R_i = A_i c \tag{A.50}$$

where A_i is deck area and c is the unit replacement cost per area. Define the agency weight as the normalized share over the bridge inventory set $\mathcal{E}$:

$$w_i = \frac{R_i \cdot \text{Risk}_{\text{wildfire}}(i)}{\sum_{j \in \mathcal{E}} R_j \cdot \text{Risk}_{\text{wildfire}}(j)} \tag{A.51}$$

Given the statewide average annual agency loss from wildfire L_{wildfire} (annual historical agency cost in Table A.15), allocate it using w_i:

$$\text{AgencyCost}_{\text{wildfire}}(i) = L_{\text{wildfire}} w_i \tag{A.52}$$

The values in this source table explain the referenced FDOT risk-allocation method. The Texas statewide application uses the TxDOT-specific calculations presented in Section 2.

Table A.15: Risk allocation results [1]

Table 10.1. Risk allocation results

Hazard	Annual historical agency cost ($M)	Annual bridge days of disruption	Annual historical user cost ($k)	Total annual hazard cost ($M)	Agency cost factor $1/Dc^A$	User cost factor $1/Dc^U$
Hurricane	1,185	100	11,325	12,510	30,699.6	107.0
Tornado	200	5	566	766	1,981.7	18.2
Wildfire	500	20	2,265	2,765	20,659.0	120.3
Flood	427	112	12,684	13,111	685.2	0.8
Vessel collision	100	1	113	213	324.4	2.8
Over-height collision	500	10	1,132	1,632	76.6	0.5
Overload	300	3	340	640	2,690.3	25.4
Truck crashes	300	20	2,265	2,565	1,156.4	3.4
Advanced deterioration	283,000	3,000	339,741	622,741	3.1	0.1
Fatigue	997	52	5,891	6,888	*	*

* The risk allocation process was not used for fatigue

Source: [1, Table 10.1]

User cost. For bridge i, define the daily detour cost:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}} \frac{\ell_i}{v_i} \right) \tag{A.53}$$

where ADT_i is average daily traffic, ℓ_i is detour length, v_i is detour speed, c_{op} is unit operating cost per mile, and c_{time} is value of time per hour.

Define the user weight as the normalized share over the bridge inventory:

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{wildfire}}(i)}{\sum_{j \in \mathcal{E}} U_j \cdot \text{Risk}_{\text{wildfire}}(j)} \tag{A.54}$$

Given the statewide annual control total for wildfire user losses L^U_{wildfire} (annual historical user cost in Ta-

ble A.15), allocate it using u_i:

$$\text{UserCost}_{\text{wildfire}}(i) = L^{U}_{\text{wildfire}} u_i \tag{A.55}$$

Total cost. After computing $\text{AgencyCost}_{\text{wildfire}}(i)$ and $\text{UserCost}_{\text{wildfire}}(i)$ for each bridge i, form the total annual expected impact (USD/year) by summing the two components:

$$\text{TotalCost}_{\text{wildfire}}(i) = \text{AgencyCost}_{\text{wildfire}}(i) + \text{UserCost}_{\text{wildfire}}(i) \tag{A.56}$$

The report generates various bridge-level metrics, such as a wildfire risk index, agency cost, and user cost. Consequently, bridge rankings can be determined based on specific objectives: prioritize by risk to highlight relative hazard and vulnerability, or rank by agency cost, or total cost to emphasize economic impact.

A.4.2.2 20-07/Task 378 Assessing Risk for Bridge Management [2]

This procedure estimates, for a defined **wildfire hazard scenario** over a specified geographic **area considered**, the:

- **Annual likelihood** that a given bridge experiences wildfire-driven service disruption (closure, restriction, or comparable operational disruption); and
- **Expected recovery cost** consistent with historical observations, scaled to current conditions (inflation and inventory growth) and allocated to individual bridges based on relative susceptibility.

Step-by-Step Procedure Step 1: Define the wildfire damage scenario and analysis boundary

1. **Damage-based incident definition.** Define a wildfire incident as bridge damage attributable to wildfire (i.e., repairable physical damage to any bridge component). Exclude events with no bridge damage (e.g., precautionary closures or nearby fires without asset impact).
2. **Analysis boundary.** State the fixed study area and the bridge inventory included.
3. **Historical window.** Choose the historical observation period length T (years) used to count wildfire-damage incidents and associated recovery expenditures.

Step 2: Compile historical wildfire damage and recovery data

1. Estimate N_{hist}: the total number of wildfire damage incidents to bridges in the study area over T years, using available sources (e.g., maintenance/emergency records, inspection reports, incident logs; supplement with expert polling if needed).
2. Estimate E_{hist}: total recovery expenditures attributable to wildfire bridge damage over the same window (in $ million).

Step 3: Collect scaling inputs (inflation and inventory change)

1. Collect a construction cost index at the start and end of the historical window, I_0 and I_T. Historical expenditures are reported in then-year dollars. I_0 and I_T provide the conversion needed to express wildfire

recovery costs on a consistent (current) cost basis; otherwise costs from earlier years would be systematically understated in today's dollars.

2. Collect the number of bridges in the study inventory at the start and end of the window, B_0 and B_T. The historical total number of wildfire-damaged bridges, N_{hist}, scales with how many bridges exist in the inventory being observed. If the inventory grew (or shrank) over T years, dividing by T alone conflates exposure change with risk change. B_0 and B_T allow conversion of historical totals to a current annual rate consistent with today's inventory size.
3. Collect the average deck area at the start and end of the window, $\bar{A}_0$ and $\bar{A}_T$.
   Why: recovery cost is driven not only by unit prices (captured by I) but also by the typical amount of work per damaged bridge, which tends to increase with bridge size. If the average bridge became larger over time, scaling costs using the cost index alone would understate current expected recovery costs. $\bar{A}_0$ and $\bar{A}_T$ capture changes in typical asset size used when scaling recovery expenditures.

Step 4: Compute annual change rates (for incidents vs. costs) This step computes two yearly change rates that describe how the system evolved during the historical window. They are used later to convert historical totals over T years into a current-equivalent annual level.

(a) Annual change rate (generic) Given any quantity that changes from V_0 (start) to V_T (end) over T years, the average annual rate of change is:

$$g \;=\; \left(\frac{V_T}{V_0}\right)^{1/T} - 1 \tag{A.57}$$

(b) Incident exposure change rate For wildfire damage incidents, the main driver of exposure is how many bridges exist in the inventory. The annual growth rate of the bridge count:

$$g_{\text{inc}} \;=\; \left(\frac{B_T}{B_0}\right)^{1/T} - 1 \tag{A.58}$$

(c) Cost scale change rate For recovery costs, changes come from three sources: (i) unit prices (construction cost index), (ii) number of bridges, and (iii) typical bridge size. They are combined into one "cost scale" measure:

$$V_0^{\text{cost}} = I_0\, B_0\, \bar{A}_0, \qquad V_T^{\text{cost}} = I_T\, B_T\, \bar{A}_T \tag{A.59}$$

and compute its annual growth rate:

$$g_{\text{cost}} \;=\; \left(\frac{V_T^{\text{cost}}}{V_0^{\text{cost}}}\right)^{1/T} - 1 \;=\; \left(\frac{I_T B_T \bar{A}_T}{I_0 B_0 \bar{A}_0}\right)^{1/T} - 1 \tag{A.60}$$

This is the average annual increase in the overall cost level driven by inflation plus changes in inventory size and typical bridge size.

Step 5: Convert growth rates to scaling multipliers (annuity factors) To convert historical totals accumulated over T years into a current-equivalent uniform annual quantity, compute the multiplier:

$$M(g,T) \;=\; \sum_{t=0}^{T-1}(1+g)^t \;=\; \begin{cases} \dfrac{(1+g)^T-1}{g}, & g \neq 0, \\ T, & g = 0. \end{cases} \tag{A.61}$$

Then compute:

$$M_{\text{inc}} = M(g_{\text{inc}}, T), \qquad M_{\text{cost}} = M(g_{\text{cost}}, T) \tag{A.62}$$

M_{inc} adjusts the incident total for changes in the number of bridges during the T-year observation period. M_{cost} adjusts the cost total for changes in prices and the bridge inventory during the same period.

Step 6: Estimate scaled annual disruption frequency and unit recovery cost

(a) Annual number of incidents (inventory-wide)

$$N_{\text{ann}} \;=\; \frac{N_{\text{hist}}}{M_{\text{inc}}} \quad \text{(expected wildfire disruption incidents per year over the area inventory).} \tag{A.63}$$

(b) Annual total recovery expenditure (inventory-wide) Convert \$ million to \$:

$$E_{\text{ann}} \;=\; \frac{E_{\text{hist}} \times 10^6}{M_{\text{cost}}} \quad (\$/\text{year}). \tag{A.64}$$

(c) Unit recovery cost intensity (area-normalized) Define the unit recovery cost per incident per unit deck area:

$$c_{\text{area}} \;=\; \frac{E_{\text{ann}}}{N_{\text{ann}}\,\bar{A}} \quad (\$/\text{ft}^2 \text{ per incident}), \tag{A.65}$$

where $\bar{A}$ is the current representative average deck area for the inventory (use $\bar{A}_T$ if available; otherwise use a current-year estimate).

Step 7: Assign bridge susceptibility factors Wildfire susceptibility is treated as a **relative** score intended to differentiate bridges. Choose categorical factors such as:

- Material factor (from bridge records such as NBI 43/44 codes):

$$F^{\text{mat}} = \begin{cases} 0.2, & \text{Concrete} \\ 1.0, & \text{Steel} \\ 2.0, & \text{Timber} \end{cases}$$

- Fuel availability (field assessment of adjacent fuels/vegetation):

$$F^{\text{fuel}} = \begin{cases} 0.2, & \text{Low} \\ 1.0, & \text{Medium} \\ 2.0, & \text{High} \end{cases}$$

- Mitigation effectiveness (fire breaks, defensible space, protective details, operations plan):

$$F^{\text{mit}} = \begin{cases} 0.5, & \text{High effectiveness} \\ 1.0, & \text{Medium effectiveness} \\ 1.5, & \text{Low effectiveness} \end{cases}$$

Bridge susceptibility score Compute a multiplicative susceptibility score:

$$S_i \;=\; F_i^{\text{mat}}\, F_i^{\text{fuel}}\, F_i^{\text{mit}} \tag{A.66}$$

Step 8: Allocate inventory-wide annual incidents to individual bridges (risk allocation) Let λ_i denote the **bridge-level annual incident rate** for bridge i, i.e., the expected number of wildfire damage incidents affecting bridge i per year (units: incidents/year).

The objective is to distribute the inventory-wide annual incident count N_{ann} across bridges such that:

$$\sum_i \lambda_i = N_{\text{ann}}$$

Allocation weights Choose a weight that reflects relative wildfire exposure and/or consequence scaling. A common choice is susceptibility times deck area:

$$w_i \;=\; S_i\, A_i \tag{A.67}$$

Bridge-level annual incident rates

$$\lambda_i \;=\; N_{\text{ann}}\, \frac{w_i}{\sum_j w_j} \tag{A.68}$$

Step 9: Compute expected annual recovery cost per bridge Using the area-normalized unit recovery cost c_{area} (\$/ft^2 per incident), estimate the **expected annual recovery cost** for bridge i as:

$$\mathbb{E}[C_i] \;=\; \lambda_i\; c_{\text{area}}\; A_i \qquad (\$/\text{year}) \tag{A.69}$$

Here, λ_i (incidents/year) is the bridge-level annual wildfire damage incident rate from Step 8, A_i (ft^2) is the deck area of bridge i, and c_{area} (\$/ft^2 per incident) is the average recovery cost per incident per unit area from Step 6.

A.4.2.3 USDOT BTS [5]

The USDOT Bureau of Transportation Statistics (BTS) Geospatial portal hosts the National Transportation Atlas Database (NTAD) and related open geospatial products, including the BETA - Hazard Exposure: National Highway System series. These layers provide a nationally consistent, map-ready way (Figure A.3) to explore and download transportation assets (e.g., bridges) with attributes describing their geographic exposure to multiple natural hazards, intended to support planning and screening.

For the BETA - Hazard Exposure: National Highway System Bridges layer, BTS derives per-bridge hazard-exposure attributes via GIS overlay: bridges are first filtered to those on the National Highway System, then bridge points are buffered (200 m for higher-resolution hazards; 1 km for lower-resolution hazards) and intersected with each hazard dataset; the resulting hazard attributes are then transferred back to the original bridge geometry. For FEMA National Risk Index variables, BTS applies a weighted average of tract-level attributes to each bridge; uncovered areas are labeled `No Data` for categorical hazards, and some numeric hazards use `NULL`. Wildfire exposure fields, including minimum, mean, and maximum burn probability, are derived from USDA burn-probability data. The BETA product covers NHS bridge exposure through buffer-based spatial association across source datasets with differing resolutions and update dates. Bridge-specific fragility, damage states, consequences, and non-NHS inventory coverage require additional analysis or inventory extension.

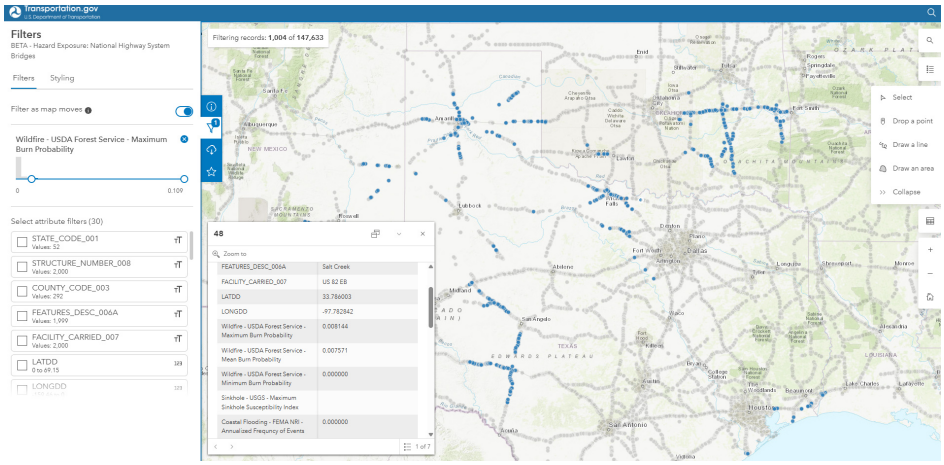


Figure A.3: Screenshot of the USDOT BTS BETA - Hazard Exposure: National Highway System Bridges web map [5]

A.4.2.4 Illinois Tollway Fire Risk Assessment [6]

The Illinois Tollway Appendix B provides a tiered fire risk assessment procedure for bridges. The procedure is designed as a screening workflow with increasing analytical effort, progressing from a rapid go/no-go check to a weighted scoring assessment, and finally to a bridge-specific detailed analysis. The three tiers are: (i) Preliminary Risk Analysis (PRA), (ii) Simplified Risk Analysis (SRA), and (iii) Detailed Risk Analysis (DRA). Bridges screened as low risk may skip SRA and/or DRA, while bridges flagged as high risk (or exceeding a length threshold) trigger DRA.

Step-by-step procedure **Step 1**. Obtain the bridge baseline information required by the Illinois Tollway fire-risk spreadsheet (geometry/length, vertical clearance, traffic context under and on the bridge, condition/health index, design live load, emergency response/access attributes, and surrounding assets/route role).

Step 2. Preliminary Risk Analysis (PRA): Answer three binary (Yes/No) screening questions:

1. Is the bridge length greater than 1000 ft?
2. Is the vertical clearance under the bridge less than 84 ft, or is there potential for flammable materials near critical structural elements (e.g., supporting columns)?
3. Is there potential for truck accessibility, railroad accessibility, and/or storage of flammable materials under or over/on the bridge?

Step 3. Use the PRA decision logic to determine whether to proceed:

- If PRA Question 1 is **Yes**, proceed to both **SRA** and **DRA** (SRA is used to identify critical risk contributors).
- If PRA Question 1 is **No** but both Questions 2 and 3 are **Yes**, proceed to **SRA**.
- Otherwise, terminate the assessment (no further analysis required).

Step 4. Simplified Risk Analysis (SRA): build the scoring inputs. For the candidate bridge, complete a structured SRA scoring sheet (Table A.16) by evaluating three subject categories:

- Occurrence: likelihood of a fire event.
- Vulnerability: susceptibility of the bridge to fire effects.
- Importance: magnitude of economic and social consequences.

Each category consists of several sub-criteria (questions). For every sub-criterion j, the assessor selects a qualitative rating

$$\text{Rating}_j \in \{\text{Low}, \text{Moderate}, \text{High}\}$$

based on available evidence (inventory data, traffic information, site context, emergency response considerations, etc.). The qualitative rating is then mapped to a numeric score

$$s_j = \begin{cases} 1, & \text{Rating}_j = \text{Low}, \\ 2, & \text{Rating}_j = \text{Moderate}, \\ 3, & \text{Rating}_j = \text{High}. \end{cases}$$

Let w_j denote the prescribed weight for sub-criterion j within its category (with $\sum_{j \in \mathcal{J}_c} w_j = 1$ for each category $c \in \{\text{occ}, \text{vuln}, \text{imp}\}$), where $\mathcal{J}_c$ is the set of sub-criteria in category c. The SRA sheet records, for each sub-criterion, its weight w_j, the chosen qualitative rating, and the resulting score s_j. These entries constitute the inputs for the category-level computations in subsequent steps (weighted sums within each category).

Step 5. Compute the Occurrence score as a weighted sum of four sub-criteria:

$$S_{\text{occ}} = 0.30\, s_{\text{mode}} + 0.30\, s_{\text{storage}} + 0.30\, s_{\text{truck}} + 0.10\, s_{\text{history}} \tag{A.70}$$

where the four sub-criteria correspond to (i) transportation mode under the bridge, (ii) potential for storage of hazardous/flammable materials under the bridge, (iii) volume of truck traffic under the bridge, and (iv) accident history under the bridge.

Step 6. Compute the Vulnerability score as a weighted sum of bridge and response-related sub-criteria

Table A.16: Simplified Risk Analysis (SRA) scoring sheet (Low/Moderate/High mapped to 1/2/3).

Category	Sub-criterion	Weight	Rating	Score	Notes / Evidence (optional)
Occurrence	Transportation mode under bridge	0.30	L/M/H	1/2/3	
	Storage of hazardous/flammable materials	0.30	L/M/H	1/2/3	
	Truck volume under bridge	0.30	L/M/H	1/2/3	
	Accident history under bridge	0.10	L/M/H	1/2/3	
Vulnerability	Superstructure type	0.15	L/M/H	1/2/3	
	Span configuration / redundancy	0.10	L/M/H	1/2/3	
	Vertical clearance height	0.40	L/M/H	1/2/3	
	Bridge maintenance / health index	0.15	L/M/H	1/2/3	
	Design live load	0.10	L/M/H	1/2/3	
	Emergency communication	0.02	L/M/H	1/2/3	
	Emergency response time / distance	0.02	L/M/H	1/2/3	
	Emergency vehicle access beneath bridge	0.02	L/M/H	1/2/3	
	Water access for fire suppression	0.02	L/M/H	1/2/3	
	Drainage system characteristics	0.02	L/M/H	1/2/3	
Importance	Average Daily Traffic (ADT) on bridge	0.40	L/M/H	1/2/3	
	Rehabilitation / replacement cost	0.20	L/M/H	1/2/3	
	Assets under bridge	0.20	L/M/H	1/2/3	
	Route purpose / criticality (e.g., emergency)	0.20	L/M/H	1/2/3	

Mapping: Low $= 1$, Moderate $= 2$, High $= 3$. **Category score:** $S_{\text{cat}} = \sum_j w_j \, s_j$ (weighted sum within each category).

(weights as specified in the spreadsheet), including:

- superstructure type (15%), span configuration/redundancy (10%), vertical clearance height (40%),
- bridge maintenance/health index (15%), design live load (10%),
- emergency communication (2%), emergency response time/distance (2%), emergency vehicle access beneath the bridge (2%),
- water access for fire suppression (2%), and drainage system characteristics (2%).

Denote the resulting weighted score by S_{vuln}.

Step 7. Compute the Importance score as a weighted sum of consequence-related sub-criteria:

$$S_{\text{imp}} = 0.40\, s_{\text{ADT}} + 0.20\, s_{\text{rehab}} + 0.20\, s_{\text{under}} + 0.20\, s_{\text{route}} \tag{A.71}$$

where the terms represent (i) traffic volume on the bridge, (ii) rehabilitation costs, (iii) assets under the bridge, and (iv) the purpose/criticality of the route (e.g., emergency routes).

Step 8. Convert each category score to a discrete risk level (Low/Moderate/High) using the prescribed thresholds:

$$\text{Low: } 1.0 \le S < 1.5, \qquad \text{Moderate: } 1.5 \le S < 2.5, \qquad \text{High: } S \ge 2.5 \tag{A.72}$$

Apply this rule to S_{occ}, S_{vuln}, and S_{imp} to obtain levels $L_{\text{occ}}, L_{\text{vuln}}, L_{\text{imp}} \in \{\text{Low}, \text{Moderate}, \text{High}\}$.

Step 9. Criticality: Determine the Criticality level by cross-referencing the Occurrence and Vulnerability levels using the Criticality matrix (Table A.17), yielding

$$L_{\text{crit}} \in \{\text{CL}, \text{CM}, \text{CH}\} \quad \text{(low/moderate/high criticality)}.$$

Table A.17: Matrix defining criticality levels

		Vulnerability		
		Low	Moderate	High
Occurrence	Low	CL	CL	CM
	Moderate	CL	CM	CH
	High	CM	CH	CH

Source: [6, Table 1]

Step 10. Overall Fire Risk: Determine the overall fire risk level by cross-referencing Criticality and Importance using the Risk matrix (Table A.18), yielding

$$L_{\text{risk}} \in \{\text{RL}, \text{RM}, \text{RH}\} \quad \text{(low/moderate/high overall fire risk)}.$$

Table A.18: Matrix defining SRA risk levels

		Importance		
		Low	**Moderate**	**High**
Criticality	**Low**	RL	RL	RM
	Moderate	RL	RM	RH
	High	RM	RH	RH

Source: [6, Table 2]

Step 11. If L_{risk} = RH (HIGH), or if the bridge length exceeds 1000 ft, perform a Detailed Risk Analysis (DRA). The DRA is bridge-specific and may include computer simulations, structural analysis, environmental analysis, and the development of mitigation/countermeasures (e.g., passive fire protection). Use the SRA category contributions to target the DRA toward the dominant drivers (Occurrence, Vulnerability, and/or Importance).

Step 12. Because fires are rare events and input data can be uncertain, conduct a peer review of the SRA inputs, calculations, assumptions, and the plausibility of the resulting risk level. Use the final risk level to guide recommendations: RL implies negligible risk with no further action; RM implies acceptable risk with routine maintenance and periodic reassessment; RH requires DRA and mitigation planning.

A.5 Flood and Scour Risk

A.5.1 Introduction

Definition. Flood risk for bridges can be defined as the joint effect of (i) the likelihood and intensity of flood-related hydraulic actions at a crossing (e.g., riverine flooding, flash floods, hurricane-driven rainfall) and (ii) the resulting consequences for structural safety, serviceability, and network functionality. In practice, this risk is governed by multiple, often coupled, damage mechanisms: overtopping and inundation of the deck/approaches, hydrodynamic and hydrostatic loading, debris impact and accumulation, and foundation scour and embankment erosion that can reduce geotechnical capacity and destabilize piers and abutments.

Recent incidents. Recent events illustrate how extreme floods can rapidly translate into bridge damage and loss of connectivity Table A.19.

Government guidance.

- Federal-level (USDOT/FHWA and other federal agencies)
  - 23 CFR Part 650, Subpart A (Flood Plains). Federal regulation prescribing FHWA policies/procedures for the location and hydraulic design of highway encroachments on flood plains, forming project-level hydraulic compliance baseline [71].
  - FHWA HEC-18: Evaluating Scour at Bridges (5th ed.). Core federal guidance for bridge-scour evaluation, including methods to estimate scour and integrate scour risk into design and management decisions [72].
  - FHWA HEC-23: Bridge Scour and Stream Instability Countermeasures (2nd ed.). Practical countermeasure selection/design guidance for scour and stream instability, widely used by State DOTs for mitigation planning and implementation [73].
  - FHWA HEC-20: Stream Stability at Highway Structures (4th ed.). Provides step-by-step assessment procedures and geomorphic/hydraulic tools to diagnose stream instability at crossings and reduce flood-driven channel change impacts [74].
  - FHWA HDS-6: River Engineering for Highway Encroachments. A foundational river-engineering reference for highway and bridge interactions with movable-bed rivers, supporting hydraulic design for flooding and channel migration [75].

Table A.19: Selected U.S. flood-damaged bridge events (2021-2025)

Date	Bridge / Route	Location	Waterbody / Flood driver	Damage summary (flood-related)
2021-08-21	Multiple state bridges (e.g., SR 230 over Hurricane Creek & overflow; SR 230 over Piney River; US 70 bridges over Trace Creek)	Humphreys & Hickman Counties, Tennessee	Flash flooding (Middle TN)	TDOT reported several bridges damaged beyond repair requiring replacement / long closures after the Aug 21, 2021 flood. [58, 59]
2022-06-13	Bridge(s) in/near Yellowstone NP access corridors (e.g., near Gardiner)	Yellowstone area, Montana/Wyoming	Yellowstone River system flooding	Major flooding swept away at least one bridge and washed out roads, prompting park closures and evacuations. [60]
2022-08-01	Bridge across Grapevine Creek (local access bridge reported collapsed)	Near Grapevine, Kentucky	Historic Eastern KY flooding	Post-flood reporting documents a bridge across Grapevine Creek collapsed following the late-July 2022 flooding. [61]
2023-07-10	VT Route 106 bridge (Route 106 bridge in Reading noted as buckling)	Reading (Felchville), Vermont	North Branch of the Black River flood	During the July 2023 Vermont floods, local reporting describes the Route 106 bridge buckling as floodwaters overtook it. [62]
2023-07-16	Local bridge in Wolcott (reported partial collapse)	Wolcott, Connecticut	Intense rainfall; pond overtopping	Local TV reporting states a bridge partially collapsed when a pond overtopped its banks during severe flooding. [63]
2024-01-24	"Bonnie and Clyde" Bridge (historic truss; along FM 2854)	Conroe / Montgomery County, Texas	San Jacinto River flooding (heavy rain)	Floodwaters washed away / collapsed the historic truss bridge along FM 2854 over the San Jacinto River. [64]
2024-09-27	Interstate 26 bridges over the Nolichucky River (near Erwin)	Unicoi County, Tennessee	Hurricane Helene flooding	TDOT documented major damage and long-term closures after Helene; reporting describes I-26 bridges over the Nolichucky River as destroyed by flooding. [65, 66]
2025-07-05	RM/FM 1431 bridge over Cow Creek (washout)	Near Marble Falls / Lago Vista area, Texas	Central Texas flooding (July 4 weekend storms)	Floods washed out the Cow Creek bridge on 1431, severing a key connector; later rebuilt on an expedited timeline. [67, 68]
2025-12-11	Multiple county bridges (washouts/compromised)	Lincoln County (Libby area), Montana	Atmospheric river and snowmelt flooding	Local officials and reporting describe multiple bridges compromised or completely washed away during historic flooding. [69, 70]

  - FHWA Technical Advisory T 5140.23: Evaluating Scour at Bridges (policy memo page). Establishes program expectations that every bridge over a waterway be evaluated for scour vulnerability and that scour-critical bridges have a Plan of Action (POA) [76].
  - FHWA: Companion Guide to Developing a Risk-Based Scour Program. A program-level guide describing how agencies can structure scour appraisal, prioritization, POA development, and resource planning in a risk-based way [77].
  - FHWA TechBrief (FHWA-HIF-19-060): Scour Considerations within AASHTO LRFD Design Specifications. Explains how scour is treated as a change in foundation conditions in LRFD limit states and how to align foundation design with scour depth assumptions [78].
  - FHWA SNBI/NBIS implementation materials. Provides national program context for inspection/management requirements and implementation references that interface with scour appraisal and POA-related practices [79].
  - FHWA-NHI-135085: Plan of Action (POA) for Scour Critical Bridges. Official FHWA training/guide material summarizing POA intent and recommended management/inspection strategies for scour-critical bridges [80].
- State-level (State DOT manuals/guidelines/procedures)
  - FDOT: FDOT Scour Manual. Florida DOT's formal manual documenting scour computation procedures (including local scour at piers with complex geometries) to support design and evaluation [81].
  - FDOT Design Manual (Topic 625-000-002), Section 250: Hydraulic Data and Agency Permits. State design guidance that explicitly defines scour design process requirements (e.g., design/check flood scour elevations) and integrates them into project development [82].
  - WisDOT Bridge Manual, Chapter 8: Hydraulics. A state bridge-manual chapter consolidating hydraulic design policy, including floodplain encroachment considerations and bridge-waterway design practice references [83].
  - NYSDOT: NYSDOT Bridge Manual (2025). Statewide manual for bridge project development and design; serves as the formal policy/procedure reference for NYSDOT bridge engineering practice [84].
  - NYSDOT: Plan of Action for Scour Critical Bridges (fillable form / program document). Official POA documentation package used to standardize monitoring actions, responsibilities, and triggers for scour-critical bridges [85].
  - VDOT: Bridge Inspection Documentation Guidelines (updated 2025). State inspection documentation requirements that explicitly call for scour-critical bridges to be monitored according to their POA, linking inspection practice to flood response actions [86].
  - ALDOT: Plan of Action for City-Owned Scour Critical Bridges (rev. 2020). A state DOT-maintained POA template/procedure outlining what to check during high-flow events and how monitoring is to be carried out for scour-critical structures [87].
  - WSDOT: Bridge Scour Appraisal Level 1-Screening and Assessment (Form 722-011). Standardized state screening/assessment document that includes the decision logic for whether a scour POA is required and how the appraisal is documented [88].
  - TxDOT: Online manual section "Scour". A formal, maintained DOT manual page describing how scour is treated in LRFD foundation evaluation and how design/check flood condition changes are

considered [89].

- Caltrans: Scour Critical Bridges in California. State guidance note describing the scour-critical designation context and pointing to the federal technical advisory and HEC-series documents for analysis and response [90].

Representative Case Studies in Texas

- I-20 Bridge over Salt Draw - Reeves County, TX (2004)
  - Flash flooding along Salt Draw destroyed the Interstate 20 bridge near Toyah, TX when flood flows washed it away. Intense flooding and erosion of the bridge's supporting soils (scour) led to failure; the bridge was washed out entirely [91].
- RM 165 Bridge over the Blanco River - Blanco/Mason County, TX (2015)
  - During the 2015 Memorial Day floods (extreme rainfall and river surge), the RM 165 two-lane bridge over the Blanco River was completely washed out, and another nearby bridge suffered significant damage due to flood forces and scour of foundations.Severe floodwaters exerted large hydrodynamic forces on substructure elements and washed out supporting material around piers/abutments [92].
- FM 2900 Bridge over the Llano River - Kingsland, TX (2018)
  - A historic rainfall and flooding event caused the FM 2900 bridge to collapse when floodwaters degraded the riverbed and undermined the structure. Severe flood and associated scour at pier/abutment locations leading to collapse [92].
- Cow Creek Bridge (FM 1431) - Travis County (2025)
  - The bridge was destroyed by Central Texas flooding and later rebuilt; locally linked to flood scour and high flows [93].
- Historic Joppa Iron Truss Bridge - Burnet County (2025)
  - The 118-year-old "Old Rusty Bridge" was washed away by floodwaters in Central Texas floods [94].

A.5.2 Methods Reported in the Literature

A.5.2.1 FDOT Assessment Procedure [1]

The report treats flood risk as a network-level screening problem in which each bridge is assigned (i) a site-specific hazard likelihood and (ii) a relative consequence (vulnerability) index based on readily available inventory data.

(1) **Flood hazard likelihood** is derived from GIS overlay of bridge locations with FEMA/FGDL floodplain maps, classifying bridges into 100-year flood zones (annual rate $\lambda = 0.01$), 500-year flood zones ($\lambda = 0.002$), or outside the floodplain ($\lambda \approx 0$). Assuming flood occurrences follow a Poisson process, the annual probability is computed as $p_{\text{flood}} = 1 - e^{-\lambda}$.

(2) **Vulnerability**: Because bridge-specific flood damage records are limited, consequences are represented using expert-judgment vulnerability scores informed by observed hurricane/flood damage mechanisms (e.g., deck inundation, debris impact, abutment/approach erosion, and scour). Vulnerability is modeled as a multiplicative index assembled from element susceptibility (with channel-related components treated as most

vulnerable), plus modifiers tied to key explanatory variables such as superstructure design type/material, average span length, channel condition (NBI Item 61), scour criticality (NBI Item 113), whether the bridge is over water, and bridge age.

The resulting flood risk index is used to rank bridges and to allocate statewide annual agency and user-loss "control totals" across the inventory in proportion to each bridge's risk.

Step-by-step procedure Step 1. Obtain the bridge inventory (FDOT/Pontis bridge database with bridge location and NBI/Pontis attributes) as the baseline asset set for risk computation.

Step 2. Identify whether bridge i is over a waterway using the bridge inventory attribute NBI Item 71 (Waterway Adequacy) (Table A.20). If Item 71 is coded as `N` (Bridge not over waterway), set the indicator to 0; otherwise set it to 1:

$$I_{\text{over water}}(i) = \begin{cases} 0, & \text{if NBI Item 71 indicates } \texttt{N} \text{ (bridge not over waterway)}, \\ 1, & \text{otherwise.} \end{cases} \tag{A.73}$$

Table A.20: NBI Item 71 Waterway adequacy and implied annual probability of bridge overtopping

Principal Arterials	Principal & Minor Arterials	Minor Collectors	Description Code	Chance of Overtopping	Annual Prob.
N	N	N	Bridge not over a waterway.	N/A	N/A
9	9	9	Bridge deck and roadway approaches above flood water elevations (highwater). Chance of overtopping is remote.	Remote	0.01
8	8	8	Bridge deck above roadway approaches. Slight chance of overtopping roadway.	Slight	0.02
6	6	7	Slight chance of overtopping bridge deck and roadway approaches.	Slight	0.02
4	5	6	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with insignificant traffic delays.	Occasional	0.10
3	4	5	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with significant traffic delays.	Occasional	0.10
2	3	4	Occasional overtopping of bridge deck and roadway approaches with significant traffic.	Occasional	0.10
2	2	3	Frequent overtopping of bridge deck and roadway approaches with significant traffic.	Frequent	0.50
2	2	2	Occasional or frequent overtopping of bridge deck and roadway approaches with significant traffic.	Frequent	0.50
0	0	0	Bridge closed.	–	–

Step 3. The annual overtopping probability, p_{flood}, is obtained by table lookup (Table A.21) using NBI Item 71 and NBI Item 26.

Step 4. Compute the flood/scour vulnerability multipliers from bridge attributes, each expressed on a di-

Table A.21: Annual probability of flood overtopping at bridge locations [1]

Table 6.6. Annual probability of flood overtopping at bridge locations

				Waterway Adequacy (NBI Item 71)								
				0	2	3	4	5	6	7	8	9	N
Functional Class (NBI Item 26)	Rural		Bridge Closed										
	Principal Arterial – Interstate	1		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Principal Arterial – Other	2		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Minor Arterial	6		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Major Collector	7		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Minor Collector	8		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N
	Local	9		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N
	Urban											
	Principal Arterial – Interstate	11		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Principal Arterial - Other Freeways or Expressways	12		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Other Principal Arterial	14		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Minor Arterial	16		0.50	0.10	0.10	0.10	0.02	0.02	0.02	0.01	N
	Collector	17		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N
	Local	19		0.50	0.50	0.10	0.10	0.10	0.02	0.02	0.01	N

Source: [1, Table 6.6]

Note: The annual overtopping probabilities reported in the table are not obtained from statistical fitting; they are constructed by converting the qualitative/interval-based overtopping frequency implied by NBI Item 71 into a single representative return period and then taking its reciprocal. The four frequency classes are mapped to representative return periods of 100, 50, 10, and 2 years for Remote (> 100 yr), Slight (11–100 yr), Occasional (3–10 yr), and Frequent (< 3 yr), respectively. The implied annual probability is then approximated by $p_{\text{flood}} \approx \frac{1}{T_{\text{rep}}}$, which yields the discrete probability levels $0.01, 0.02, 0.10$, and 0.50 used in the table.

mensionless scale (typically via a 0–5 raw score rescaled by the common transformation $(\cdot + 1)/3$).

Step 5. Compute the element vulnerability multiplier $V_{\text{flood}}^{\text{elem}}(i)$. Let $\mathcal{E}_i$ be the set of bridge elements recorded for bridge i. For each element $e \in \mathcal{E}_i$, obtain the raw flood element score $s_{\text{flood}}(e) \in \{0, 1, 2, 3, 4, 5\}$ from the flood element-parameter table (Table A.22). Define the bridge-level average raw element score:

$$\bar{s}_{\text{flood}}(i) = \frac{1}{|\mathcal{E}_i|} \sum_{e \in \mathcal{E}_i} s_{\text{flood}}(e) \tag{A.74}$$

and rescale it into a multiplier:

$$V_{\text{flood}}^{\text{elem}}(i) = \frac{\bar{s}_{\text{flood}}(i) + 1}{3} \tag{A.75}$$

Step 6. Compute the design vulnerability multiplier $V_{\text{flood}}^{\text{des}}(i)$.

$$\text{AvgSpan}(i) = \frac{\text{BridgeLength}(i)}{\text{NumSpans}(i)} \tag{A.76}$$

Using the bridge superstructure design type and the appropriate span-length category, obtain the raw design score $s_{\text{flood}}^{\text{des}}(i) \in \{0, 1, 2, 3, 4, 5\}$ from the flood design-parameter table (Table A.23), and rescale:

$$s_{\text{flood}}^{\text{des}}(i) = \begin{cases} \text{use the Short average span" column in Table A.23,} & \text{AvgSpan}(i) \leq 20 \text{ m,} \\ \text{use the Long average span" column in Table A.23,} & \text{AvgSpan}(i) > 20 \text{ m.} \end{cases} \tag{A.77}$$

Table A.22: Levels of vulnerability of bridge elements to damage from flooding

Table 5.6. Levels of vulnerability of bridge elements to damage from flooding

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	3	476	Timber Walls	4
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	3	477	Other Walls	4
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	4	478	MSE Walls	4
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	3	480	Mast Arm Found	2
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	3	481	Paint Mast Arm Vert	1
31	Timber Deck	2	233	P/S Conc Cap	3	482	Galvan Mast Arm Vert	1
32	Timber Deck/AC Ovly	2	234	R/Conc Cap	3	483	Other Mast Arm Vert	1
38	Bare Concrete Slab	1	235	Timber Cap	3	484	Paint Mast Arm Horzn	1
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	4	485	Galvan Mast Arm Horz	1
54	Timber Slab	2	241	Concrete Culvert	4	486	Other Mast Arm Horzn	1
55	Timber Slab/AC Ovly	2	242	Timber Culvert	4	487	Sign Member Horiz	1
98	Conc Deck on PC Pane	1	243	Misc Culvert	4	488	Sign Member Vertical	1
99	PS Conc Slab	1	290	Channel	5	489	Sign Foundation	2
101	Unpnt Stl Box Girder	2	298	Pile Jacket Bare	4	495	Uncoat High Mast L.	1
102	Paint Stl Box Girder	2	299	Pile Jacket/Cath Pro	4	496	Painted High Mast L.	1
104	P/S Conc Box Girder	1	300	Strip Seal Exp Joint	3	497	Galvan. High Mast L.	1
105	R/Conc Box Girder	1	301	Pourable Joint Seal	3	498	Other High Mast L.P.	1
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	3	499	H. M. L. P. Found.	2
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	3	540	Open Gearing	4
109	P/S Conc Open Girder	1	304	Open Expansion Joint	3	541	Speed Reducers	4
110	R/Conc Open Girder	1	310	Elastomeric Bearing	3	542	Shafts	4
111	Timber Open Girder	2	311	Moveable Bearing	3	543	Shaft Brgs and Coupl	4
112	Unpnt Stl Stringer	2	312	Enclosed Bearing	3	544	Brakes	4
113	Paint Stl Stringer	2	313	Fixed Bearing	3	545	Emergency Drive	4
115	P/S Conc Stringer	1	314	Pot Bearing	3	546	Span Drive Motors	4
116	R/Conc Stringer	1	315	Disk Bearing	3	547	Hydraulic Power Unit	4
117	Timber Stringer	2	320	P/S Conc Appr Slab	4	548	Hydraulic Piping Sys	4
120	U/Stl Thru Truss/Bot	2	321	R/Conc Approach Slab	4	549	Hydraulic Cylinders	4
121	P/Stl Thru Truss/Bot	2	330	Metal Rail Uncoated	1	550	Hopkins Frame	4
125	U/Stl Thru Truss/Top	2	331	Conc Bridge Railing	1	560	Locks	4
126	P/Stl Thru Truss/Top	2	332	Timb Bridge Railing	1	561	Live Load Shoes	4
130	Unpnt Stl Deck Truss	2	333	Other Bridge Railing	1	562	Counterweight Suppor	4
131	Paint Stl Deck Truss	2	334	Metal Rail Coated	1	563	Acc Ladd & Plat	4
135	Timber Truss/Arch	2	356	Steel Fatigue SmFlag		564	Counterweight	4
140	Unpnt Stl Arch	2	357	Pack Rust Smart Flag		565	Trun/Str and Cur Trk	4
141	Paint Stl Arch	2	358	Deck Cracking SmFlag		570	Transformers	4
143	P/S Conc Arch	1	359	Soffit Smart Flag		571	Submarine Cable	4
144	R/Conc Arch	1	360	Settlement SmFlag		572	Conduit & Junc. Box	4
145	Other Arch	2	361	Scour Smart Flag		573	PLCs	4
146	Misc Cable Uncoated	3	362	Traf Impact SmFlag		574	Control Console	4
147	Misc Cable Coated	3	363	Section Loss SmFlag		580	Navigational Lights	4
151	Unpnt Stl Floor Beam	2	369	Sub.Sect Loss SmFlag		581	Operator Facilities	4
152	Paint Stl Floor Beam	2	370	Alert Smart Flag		582	Lift Bridge Spec. Eq	4
154	P/S Conc Floor Beam	1	386	Fender/Dolphin Uncoa	3	583	Swing Bridge Spec. E	4
155	R/Conc Floor Beam	1	387	P/S Fender/Dolphin	3	590	Resistance Barriers	4
156	Timber Floor Beam	2	388	R/Conc Fender/Dolphi	3	591	Warning Gates	4
160	Unpnt Stl Pin/Hanger	2	389	Timber Fender/Dolphi	3	592	Traffic Signals	4
161	Paint Stl Pin/Hanger	2	390	Other Fender/Dolphin	3			
201	Unpnt Stl Column	2	393	Blkhd Sewl Metal Unc	3			
202	Paint Stl Column	2	394	R/Conc Abut Slope Pr	4			
204	P/S Conc Column	1	395	Timber Abut Slope Pr	4			
205	R/Conc Column	1	396	Other Abut Slope Pro	4			
206	Timber Column	2	397	Drain. Syst Metal	3			
207	P/S Conc Holl Pile	3	398	Drain. Syst Other	3			
210	R/Conc Pier Wall	3	399	Other Xpansion Joint	3			
211	Other Mtl Pier Wall	3	474	Walls Uncoated	4			
215	R/Conc Abutment	3	475	R/Conc Walls	4			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Source: [1, Table 5.6]

$$V_{\text{flood}}^{\text{des}}(i) = \frac{s_{\text{flood}}^{\text{des}}(i) + 1}{3} \tag{A.78}$$

Table A.23: Levels of vulnerability of bridge types to damage from flooding

Table 5.7. Levels of vulnerability of bridge types to damage from flooding

Superstructure design - main unit	Short average span	Long average span
Slab	4	2
Stringer/MultiBeam/Girder	4	2
Girder & Floorbeam	4	2
Tee Beam	4	2
Box Beam or Girders - Multiple	4	2
Box Beam/Girders - Single or Spread	4	2
Frame (except frame culverts)	4	2
Orthotropic	4	2
Truss - Deck	4	2
Truss - Thru	4	2
Arch - Deck	4	2
Arch - Thru	1	1
Suspension	4	2
Stayed Girder	4	2
Movable - Lift	4	2
Movable - Bascule	4	2
Movable - Swing	4	2
Tunnel	4	2
Culvert	4	2
Mixed Types	4	2
Segmental Box Girder	4	2
Channel Beam	4	2

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Source: [1, Table 5.7]

Step 7. Compute the channel vulnerability multiplier $V_{\text{flood}}^{\text{chan}}(i)$ from channel condition (NBI Item 61).

$$V_{\text{flood}}^{\text{chan}}(i) = \begin{cases} 0.400, & NBI_{61}(i) \in \{8, 9\} \\ 0.995, & NBI_{61}(i) = 7 \\ 1.139, & NBI_{61}(i) = 6 \\ 1.600, & NBI_{61}(i) \in \{4, 5\} \\ 2.995, & NBI_{61}(i) \in \{0, 1, 2, 3\} \\ 1.000, & NBI_{61}(i) = \text{N (not applicable / missing)} \end{cases} \tag{A.79}$$

Note: The FDOT report uses channel condition (NBI Item 61) as an explanatory variable in the flood/scour consequence component. The case-study values define the inferred mapping in Equation A.79: ratings 8–9 map to 0.400, ratings 6–7 map to 0.995 and 1.139, ratings 4–5 map to 1.600, and ratings 0–3 map to 2.995. This binning assigns higher flood/scour vulnerability factors to degraded channels, unstable banks, and inadequate protection.

Step 8. Compute the scour-criticality vulnerability multiplier $V_{\text{flood}}^{\text{scour}}(i)$ from scour criticality (NBI Item 113). Obtain a raw scour score $s_{\text{flood}}^{\text{scour}}(i) \in \{0, 1, 2, 3, 4, 5\}$ from the flood/scour parameter table keyed to Item 113, and rescale:

$$V_{\text{flood}}^{\text{scour}}(i) = \begin{cases} 2.000, & \text{if } c_{113}(i) \in \{0, 1, 2, 3, U\} \\ 1.200, & \text{if } c_{113}(i) \in \{4, 5, 6, 7, 8, 9\} \\ 0.333, & \text{if } c_{113}(i) = N \end{cases} \tag{A.80}$$

Step 9. Compute the age vulnerability multiplier $V_{\text{flood}}^{\text{age}}(i)$. Let Y_i be the year bridge i was built (NBI Item

27). The report uses a linear age factor of the same form as the other hazards:

$$V^{\text{age}}_{\text{flood}}(i) = 1 + \frac{1990 - Y_i}{35} \tag{A.81}$$

Step 10. Compute the floods-and-scour risk index and rank bridges in descending order:

$$I_{\text{over water}}(i) = \begin{cases} 1, & \text{if bridge } i \text{ is over a waterway,} \\ 0, & \text{otherwise.} \end{cases} \tag{A.82}$$

$$\text{Risk}_{\text{flood}}(i) = I_{\text{over water}}(i)\; p_{\text{flood}}(i)\; V^{\text{scour}}_{\text{flood}}(i)\; V^{\text{elem}}_{\text{flood}}(i)\; V^{\text{age}}_{\text{flood}}(i)\; V^{\text{des}}_{\text{flood}}(i)\; V^{\text{chan}}_{\text{flood}}(i) \tag{A.83}$$

Agency cost. First, compute the replacement cost for bridge i:

$$R_i = A_i c \tag{A.84}$$

where A_i is deck area and c is the unit replacement cost per area. Define the agency weight as the normalized share over the bridge inventory set $\mathcal{E}$:

$$w_i = \frac{R_i \cdot \text{Risk}_{\text{flood}}(i)}{\sum_{j \in \mathcal{E}} R_j \cdot \text{Risk}_{\text{flood}}(j)} \tag{A.85}$$

Given the statewide average annual agency loss from flooding L_{flood} (annual historical agency cost in Table A.15), allocate it using w_i:

$$\text{AgencyCost}_{\text{flood}}(i) = L_{\text{flood}}\, w_i \tag{A.86}$$

User cost. For bridge i, define the daily detour cost:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}} \frac{\ell_i}{v_i} \right) \tag{A.87}$$

where ADT_i is average daily traffic, ℓ_i is detour length, v_i is detour speed, c_{op} is unit operating cost per mile, and c_{time} is value of time per hour. Define the user weight as the normalized share over the bridge inventory:

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{flood}}(i)}{\sum_{j \in \mathcal{E}} U_j \cdot \text{Risk}_{\text{flood}}(j)} \tag{A.88}$$

Given the statewide annual control total for flooding user losses L^U_{flood} (annual historical user cost in Table A.15), allocate it using u_i:

$$\text{UserCost}_{\text{flood}}(i) = L^U_{\text{flood}}\, u_i \tag{A.89}$$

Total cost. After computing $\text{AgencyCost}_{\text{flood}}(i)$ and $\text{UserCost}_{\text{flood}}(i)$ for each bridge i, form the total annual expected impact (USD/year) by summing the two components:

$$\text{TotalCost}_{\text{flood}}(i) = \text{AgencyCost}_{\text{flood}}(i) + \text{UserCost}_{\text{flood}}(i) \tag{A.90}$$

A.5.2.2 Bridge Scour Screening Guidelines [7]

This guide operationalizes flood vulnerability at the bridge-network level primarily through foundation scour susceptibility screening. The approach is designed as a portfolio screening method to prioritize limited resources, using (i) a qualitative scour susceptibility code rating (NBI Item 113) as the primary indicator of likelihood, and (ii) a qualitative Consequence Analysis (CA) to represent failure consequences. A Plan of Action (POA) is then developed for scour-critical bridges to mitigate risk.

Step-by-step procedure **Step 1**. Define the baseline asset set as all existing Type I bridges (public-road highway bridges) in the owner inventory.

Step 2. For each bridge, check whether a comprehensive quantitative scour analysis has already been performed and the calculated scour is fully documented. If yes, a new scour screening is not required.

Step 3. Assemble the bridge information package for an office screening:

- as-built plans/drawings,
- routine/underwater inspection reports,
- maintenance and modification history,
- flood history and any documented scour observations.

Step 4. Conduct the qualitative office screening linked with field data to evaluate scour susceptibility drivers, including (at minimum):

- channel bed material and planform history (lateral migration / stability),
- bridge features susceptible to scour,
- prior inspection documentation regarding potential scour.

Step 5. Trigger follow-up field visits for a subset of bridges, especially when documentation is inadequate or scour susceptibility is deemed greater. Field work may include visual assessment of channel and foundation conditions and use of repeat cross-section surveys (if available or collected).

Step 6. Assign the bridge scour susceptibility code rating (NBI Item 113) using the guide's coding descriptions and engineering judgement supported by field assessment. Record the code, narrative rationale, and supporting evidence.

- Scour critical: code rating ≤ 3.
- Non-scour critical: code rating ≥ 4, except code 6.
- Scour inconclusive: code rating $= 6$ (temporary; collect sufficient information and re-assess within a defined short timeframe).

Step 7. Interpret risk using the guide's risk framing:

$$\text{Risk} \;=\; \text{Likelihood} \times \text{Consequence} \tag{A.91}$$

where Likelihood is incorporated into the scour code rating, and Consequence is evaluated through the Consequence Analysis (CA).

Step 8. For every scour critical bridge (code ≤ 3), perform a Consequence Analysis (CA) assuming bridge failure has occurred. The CA is a qualitative, locally-informed assessment that classifies the bridge into low or high consequence (significance) categories based on environmental, transportation, and public-safety impacts. At minimum, document factors such as:

- access to significant infrastructure (e.g., high hazard dam, power plant, pumping plant),
- sole access to private property/residences,
- recreation access,
- trucking route relevance,
- average daily traffic threshold (as used by the guide),
- economic costs to the public if the bridge failed,
- threatened/endangered species and environmental costs,
- historical significance.

Note: A low-consequence outcome does not change a scour-critical code to non-scour-critical.

Step 9. Develop a Plan of Action (POA) for each scour critical bridge to mitigate risk. The POA is site-specific, commensurate with the CA outcome, and documents what should be done regarding monitoring, repairing, or replacing the bridge (without including design). It serves two primary purposes:

1. define a systematic process for monitoring and (if needed) closing/re-opening bridges during and after significant floods to protect bridge users,
2. support programming and prioritization of scour countermeasures for scour critical bridges.

Step 10. Select the POA type based on CA-informed risk:

- Simplified POA (low consequence / lower risk): typically focuses on establishing a monitoring program (and may include cost:benefit consideration for countermeasures). Include: site identification, hydrologic/hydraulic characteristics, decision/communication responsibilities, closure triggers, detour routes, etc. Monitoring alone does not change the scour code.
- Comprehensive POA (high consequence / higher risk): typically recommends longer-term actions such as repair via structural or hydraulic countermeasures, or replacement. Countermeasures should be drawn from one or more categories: monitoring, hydraulic, structural. If countermeasures are installed, re-screen the bridge; code upgrade is permitted only when appropriate countermeasures are in place and functioning.

Step 11. For bridges requiring deeper evaluation (commonly as part of a comprehensive POA), perform a quantitative scour analysis (site-specific hydraulics and channel morphology assessment) following current state-of-practice guidance (e.g., HEC-18). This analysis must be performed by a hydraulic engineer.

Step 12. Implement routine field inspection scour checks per NBIS after initial screening/analysis:

- Observe channel conditions (planform, slope, bed material, vegetation, bank erosion indicators) alongside bridge conditions.
- Repeat cross-section surveys and collect photographs/field descriptions to track scour trends.
- If scour/countermeasures are submerged and not visible, perform underwater inspection (e.g., depth sounding, probing, snorkeling, diving).
- Each routine inspection report should explicitly state the current scour code rating and whether it remains valid.

A.5.2.3 CTDOT Inland Flooding Vulnerability Screening for Bridges and Culverts [8]

The CTDOT pilot implements a systems-level vulnerability screening for state-owned bridge and culvert structures (span length 6-20 ft) subject to inland flooding from extreme rainfall. The workflow combines (i) inventory-based screening and field review, (ii) hydrologic estimation of design/check discharges using

current precipitation data, (iii) hydraulic capacity checks and "adaptive capacity" assessment using HY-8 rating curves, and (iv) a qualitative multi-factor criticality ranking that integrates hydraulic, spatial, and social considerations.

Step-by-step procedure **Step 1 (Define scope and baseline asset set).** Define the study region and structure scope (state-road structures conveying watercourses with span length 6-20 ft). Extract the baseline asset set from the state bridge/structure inventory (SIS/NBI-linked database), including key attributes (e.g., route, structure number, year built, ADT, condition items, scour critical flag, geometry, etc.).

Step 2 (Initial network screening and candidate selection). Use the CTDOT Structure Information System (SIS) inventory to (i) define the eligible structure population within the project limits and (ii) select candidates for field review via explicit database queries and prioritization rules.

1. **Define the eligible population from SIS.** Run GIS/attribute queries to count and extract, within the study towns, the subset of State road structures over water with span length 6-20 ft. This yields the scoped population (176 structures within the project limits).
2. **Exclude structures with active/pending projects.** Further investigate the scoped set to identify structures that already have an active design project or are in the scoping phase. Exclude these from the screening pool (18 excluded), leaving 158 structures eligible for selection and evaluation.
3. **Round 1 candidate selection: worst condition first (for field review).** From the eligible pool, select an initial batch (30 structures) for field review by sorting on the Structural Evaluation condition rating (NBI Item 67 / SIS field B67) and taking those with the lowest ratings at the time of the original query. The report notes that the selected set was predominantly Item 67 = 5 ("Fair") with one Item 67 = 4 ("Poor"), and interprets ratings ≤ 5 as indicating a need for more intensive maintenance/preservation in the near term.
4. **Round 2 candidate selection: increase sample size using traffic/strategic importance and hydraulic flags.** After preliminary evaluations indicated that the budget/schedule could support evaluating roughly 50-60 structures, select an additional batch (34 structures) using targeted database queries, with one explicit exception noted below. With the exception of Structure No. 05418, the second-round selection is based on:
   - Highest ADT (Average Daily Traffic) but not recently built or reconstructed (20 structures);
   - On the Designated National Network (NBI Item 110; national network for trucks), but not previously selected and not recently built or reconstructed (4 structures);
   - Remaining structures with Waterway Adequacy (NBI Item 71) rating ≤ 5 (i.e., potential hydraulic inadequacy flags).

   Structure No. 05418 is included because it was concurrently being investigated for a potential scour issue.
5. **Check hydrologic-method applicability.** As part of selection, aim to include a varied range of drainage areas so that different hydrologic methodologies (e.g., Rational Method vs. SCS Unit Hydrograph vs. regression/StreamStats) can be applied across the sample.

Step 3 (Field review and refinement of candidate set). Conduct structured field reviews of the initially selected candidates, and use the findings to (i) document condition factors relevant to flood vulnerability and (ii) screen out sites that would require analysis beyond the pilot scope.

1. **Plan and execute site visits.** Perform field reviews for the first-round candidate set during Spring 2013.
2. **Observe and record flood-relevant condition indicators.** At each site, the field team visually assessed and documented flood-relevant condition information for the NBI items most directly tied to inland-

flood performance and vulnerability: Substructure (Item 60), Channel and Channel Protection (Item 61), Waterway Adequacy (Item 71), and Scour Critical Bridges (Item 113). For these items, the numeric ratings were taken from the most recent bridge inspection report (i.e., existing NBIS/NBI ratings in the inventory), while the field review focused on recording the observed condition and any discrepancies or concerns relevant to increased discharge. The overall Structure Evaluation rating (Item 67) was also noted and carried into the hydraulic evaluation narrative. Unlike the other items, the report provides an explicit condition-rating scale for Item 67 (Table A.24) to support consistent interpretation of the rating values.

Table A.24: CT DOT General Structure Condition Rating Scale

Rating	Condition	Description
9	Excellent	New.
8	Very Good	No problems noted.
7	Good	Some minor problems.
6	Satisfactory	Structural elements show some minor deterioration.
5	Fair	All primary structural elements are sound, but may have minor section loss, cracking, spalling or scour.
4	Poor	Advanced section loss, deterioration, spalling or scour.
3	Serious	Loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	Critical	Advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	Imminent Failure	Major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic, but corrective action may put back in light service.
0	Failed	Out of service-beyond corrective action.

Source: [8, Table 8]

3. **Cross-check against the most recent inspection report.** For each structure, compare the field-observed conditions with the condition ratings and descriptive notes in the most recent bridge inspection report. Use this comparison to judge whether increases in discharge (from increased precipitation) could plausibly worsen observed issues (e.g., exposed footings, bank erosion, outlet scour holes, debris blockage).
4. **Cross-check against the most recent inspection report.** For each structure, reconcile the field observations with the most recent inspection-report ratings and notes by (i) checking whether observed issues correspond to the reported deficiencies and (ii) documenting any new or worsening conditions. Based on this reconciliation, populate the Condition Assessment worksheet (Table A.25) fields Potential Impact of Increased Discharge and Probable/Recommended Action Required (e.g., risk of increased bank erosion, outlet scour, debris blockage, or exposure/undermining of substructure elements) and indicate whether prioritization or immediate action is warranted.
5. **Screen out structures not suitable for the pilot scope.** Following the field review of the initial 30 structures, eliminate structures deemed not suitable for the pilot for reasons including that their evaluation would require more extensive hydrologic/hydraulic modeling beyond the scope of this project.
6. **Replace candidates as needed and advance to hydraulic evaluation.** Replace candidates when appropriate (the field team replaced one structure with Structure No. 02205) and finalize the set advanced to hydraulic evaluation (27 structures advanced from the first-round set after eliminating four).

Step 4 (Drainage area determination and structure classification). Drainage areas for the structures were initially determined using the USGS StreamStats application and were revised as necessary following the

Table A.25: Condition Assessment worksheet

Condition Category	Recent Rating	Field Observed	Discharge Impact	Recommended Action	Immediate Action?
Substructure (Item 60)					
Channel (Item 61)					
Evaluation (Item 67)					
Waterway (Item 71)					
Scour (Item 113)					

field reviews when the hydrologic and hydraulic calculations were performed.

Drainage area definition. For structure i, let Ω_i denote the StreamStats-delineated tributary watershed to the selected control point (inlet/outlet). Define the drainage area as

$$A_i = \int_{\Omega_i} dA \tag{A.92}$$

where A_i is reported in mi^2. The study procedure uses StreamStats [2] for the initial delineation and revises Ω_i if field review indicates that the delineation/control point should be corrected.

Structure class assignment (CTDOT Drainage Manual). The Drainage Manual classifies both culverts and bridges based on whether the structure conveys an established watercourse and by the size of the drainage area, with classes titled Minor, Small, Intermediate, Large, and Monumental. In this project, structures fall within Small and Intermediate, defined in the report text. Let $W_i \in \{0, 1\}$ indicate whether an established watercourse exists [3]. Then:

$$\text{Class}(i) = \begin{cases} \text{Minor}, & A_i < 1,\ W_i = 0 \\ \text{Small}, & A_i < 1,\ W_i = 1 \\ \text{Intermediate}, & 1 < A_i < 10 \\ \text{Large}, & 10 < A_i < 1000 \end{cases} \tag{A.93}$$

Design/check flood frequencies. For each class, select the design and check flood frequencies from the report (Table A.26). For Small structures, the design frequency is 50-yr unless the stream has been studied in detail by a FEMA Flood Insurance Study (FIS), in which case a 100-yr frequency is used for design, and the 100-yr effects are investigated. For Intermediate structures, design is 100-yr and the 500-yr discharge is investigated.

Step 5 (Hydrologic estimation of design/check peak discharges). Compute the required design and check peak discharges for each structure using CTDOT-standard hydrologic methods. The procedure follows

[2] USGS StreamStats is a web-based GIS application that delineates an upstream watershed at a user-selected stream location and returns basin characteristics and (where available) streamflow statistics computed from regional regression equations.

[3] Set $W_i = 1$ if the crossing conveys a distinct, continuous natural stream channel with identifiable bed and banks (often mapped as a stream/"blue line"), and set $W_i = 0$ if drainage is primarily unchannelized runoff or roadside/ditched flow without an established upstream-downstream channel.

Table A.26: Summary of Hydraulic Design Criteria for Culverts

Structure Class	Drainage Area (mi^2)	Design Freq. (yr)	Check Freq. (yr)	Backwater (ft)	Min. Freeboard (ft)
Minor	< 1 (no established watercourse)	25	–	–	1
Small	< 1	50	100	–	1
Intermediate	$\geq 1 < 10$	100	500	≤ 1	1
Large	$\geq 10 < 1000$	100	500	≤ 1	1

Source: [8, Table 2]

the CTDOT Drainage Manual (Hydrology, Chapter 6), FHWA HDS-2 (Highway Hydrology), and USGS regional-regression documentation for the mathematical details.

1. **Select the applicable method(s) and check applicability limits.** Apply one or more of the following methods, subject to their intended ranges:
   - **Rational Method:** used only for small watersheds, with a drainage-area limit of $A \leq 200$ acres.
   - **USGS Regression Equations (via StreamStats):** used only when watershed explanatory variables fall within the equations' development ranges; the report notes the drainage-area range $1.7 \leq A \leq 715$ mi^2 for the referenced Connecticut regressions.
   - **SCS Unit Hydrograph:** used when a unit-hydrograph approach is needed or when other methods are not applicable, and also used for method inter-comparison.
2. **Choose precipitation inputs consistent with the selected method.** Use current precipitation inputs for the design/check events and, where performed in the study, compute comparative discharges using legacy precipitation references. The report distinguishes precipitation inputs by method:
   - Regression and SCS methods use 24-hr rainfall depth (in.).
   - The Rational Method uses rainfall intensity (in/hr).

   The precipitation datasets used/compared are documented in Appendix C, including Table C-1 (HYDRO-35 and TP-40 rainfall data), Table C-2 (NRCC Precip.net example), and supporting figures (e.g., Figures C-1 to C-4 illustrating maps and DDF/IDF curves).
3. **Set SCS design-storm conventions (if SCS UH is used).** When using the SCS Unit Hydrograph, adopt CTDOT current-practice storm assumptions used in the report, including a 24-hr Type III temporal distribution and AMC II antecedent moisture condition.
4. **Compute Q_{design} and Q_{check} for each structure.** For each structure i, compute peak discharges for the return periods required by the Step 4 class assignment (e.g., 50-yr/100-yr for Small; 100-yr/500-yr for Intermediate per Table 2), yielding at minimum $Q_{i,\text{design}}$ and $Q_{i,\text{check}}$. When multiple methods are applied at a site, retain the set of results for comparison and for subsequent hydraulic screening.
5. **Document uncertainty/validity for regression-based estimates.** When USGS regression/StreamStats estimates are used, document the applicable prediction uncertainty (e.g., standard error of prediction and confidence limits), as illustrated in Appendix D.

Step 6 (Effect of the precipitation parameter on discharge). For each structure, incrementally increase the precipitation input used in the selected hydrologic method and plot precipitation increase against discharge

increase. Use the study's simplified increment rules (e.g., fixed depth increments for 24-hr depth methods and percent increments for intensity-based methods) to compare the resulting discharge curves.

Step 7 (Hydraulic modeling and rating curve generation with HY-8). Use FHWA HY-8 to evaluate hydraulic performance across a range of flows and to generate rating (performance) curves:

$$\text{Headwater depth vs. peak discharge,} \tag{A.94}$$

$$\text{Outlet velocity vs. peak discharge} \tag{A.95}$$

The HY-8 analyses follow FHWA culvert hydraulic procedures (HDS-5) and produce structure-specific performance curves usable for screening at multiple flow levels.

Step 8 (Hydraulic adequacy check against design criteria). Determine whether each structure satisfies the current hydraulic design criteria at the current design discharge estimate. Criteria referenced in the study include:

- allowable headwater constraints (including limits tied to freeboard and headwater-to-opening ratio, e.g. HW/D),
- freeboard definition and minimum freeboard considerations,
- review/check discharge evaluation (storm of greater magnitude),
- maximum outlet velocity consistency / need for protection and stabilization,
- flood frequency requirements by structure class.

If the structure fails the water-surface-elevation criteria at (or below) the design discharge (e.g., roadway overtopping begins), classify it as hydraulically inadequate for current conditions.

Step 9 (Adaptive capacity / "cushion" assessment for adequate structures). For structures that are hydraulically adequate at the current design discharge, assess additional hydraulic capacity (adaptive capacity) using the HY-8 rating curves by locating key hydraulic performance points, such as:

- design discharge,
- check discharge,
- onset of inlet submergence,
- discharge at the HW/D threshold,
- discharge at the 1-ft freeboard threshold,
- discharge at onset of roadway overtopping.

Define a simple adaptive-capacity metric as the margin between the design discharge and the controlling failure/violation point discharge (e.g., overtopping or controlling headwater stage):

$$\Delta Q_i = Q_{i,\text{limit}} - Q_{i,\text{design}}, \qquad \text{AC}_i = \frac{\Delta Q_i}{Q_{i,\text{design}}} \tag{A.96}$$

Use both headwater response and outlet-velocity response to judge whether increased discharge would introduce unacceptable inundation risk, erosion potential, or scour concerns over the remaining service life.

Step 10 (Condition and scour/erosion considerations). Integrate field-observed conditions and recent inspection report ratings (including scour critical status) with the hydraulic results. Flag cases where high velocities and/or existing scour indicators suggest elevated vulnerability even if headwater criteria are satisfied.

Step 11 (Network-level criticality ranking). Combine hydraulic results (adequacy and adaptive capacity),

spatial context, and social/operational importance in a qualitative criticality matrix. The study groups factors as:

- Hydraulic: adaptive capacity, history of closure/performance, satisfaction of water surface elevation criteria, adjacency to scour critical structures;
- Spatial: FEMA flood zone context, imperviousness/development patterns;
- Social: traffic measures (e.g., ADT, V/C), safety/incidents, NHS/emergency-route relevance, proximity to emergency services.

Assign overall criticality categories (e.g., Low / Moderate / Critical) to prioritize assets for follow-on action.

A.6 Hurricane Risk

A.6.1 Introduction

Definition. Hurricane risk for bridges can be defined as the combination of (i) the probability and intensity of hurricane-driven hazards (e.g., extreme winds, storm surge, waves, and rainfall-driven flooding), (ii) the exposure of bridge assets and lifeline connectivity, and (iii) the structural and geotechnical vulnerability of bridge components and adjacent infrastructure. In practice, this risk is expressed through both safety (collapse or loss of load path) and functionality (closure, reduced capacity, or loss of access) outcomes. Key hurricane-driven bridge failure modes include superstructure and deck damage from wave and surge loading, approach and embankment erosion, and foundation instability due to scour and debris-laden flows, where the hydrodynamic forcing can be substantially amplified during coastal storms and surges.

Recent events. Recent events illustrate that hurricane impacts on bridges are recurring and system-disruptive. During Hurricane Ian (September 2022), several portions of Florida's Sanibel Causeway (a series of bridges connecting Sanibel Island to the mainland) were destroyed, which left the link impassable and requiring structural rebuilds [95]. More recently, Hurricane Helene (late September 2024) produced widespread transportation damage in western North Carolina; the N.C. Department of Transportation reported thousands of damage sites to roads, bridges, and culverts and projected that at least 140 bridges would require replacement [96].

Table A.27: U.S. bridge damage events attributed to hurricanes (approx. 2021–2025).

Year	Hurricane	Key impact date (US)	State/Territory	Bridge / reported damage (with `\cite`)
2021	Ida	2021-08-29	Louisiana (LA)	Barataria-area access bridge (only bridge access reported): storm impacts left the bridge unusable; National Guard/Army reporting describes installing a temporary floating bridge to restore access [97, 98].
2022	Ian	2022-09-28	Florida (FL)	Sanibel Causeway (3-bridge causeway, Fort Myers–Sanibel): causeway washed out / severely damaged; temporary repairs reopened within about three weeks [99, 100].
2022	Fiona	2022-09-18	Puerto Rico (PR)	Utuado highway bridge (over the Guaónica River reported): bridge washed away by river flooding during Fiona [101, 102].
2024	Debby	2024-08-05	Florida (FL)	Howard Frankland Bridge / approaches (I-275, Tampa Bay): waves/flooding washed away parts of the shoulder/roadway; lanes closed [103, 104].
2024	Debby	2024-08-06	Florida (FL)	Silk Oak Lodge mobile home park bridge (Clearwater/Pinellas County): floodwater washed out a large section (nearly half) of a newly reconstructed bridge [105].
2024	Beryl	2024-07-08	Texas (TX)	Moss Oaks Dr. bridge (Riverwood Estates, Houston area): local reporting described a washed-out/partially destroyed bridge after Beryl; NHC TCR documents Texas landfall [106].
2024	Helene	2024-09-26 (landfall); Sep 2024 remnants inland	North Carolina (NC)	Lake Lure Flowering Bridge (Broad River): one end collapsed and other sections buckled; demolition planned after engineering determinations [107, 108].

To manage hurricane risk for bridges, U.S. transportation agencies rely on a small set of recurring guidance documents that together cover (i) coastal storm surge and waves, (ii) flood/scour/stream-instability mechanisms that are often triggered or intensified by hurricanes, and (iii) agency-scale vulnerability/adaptation planning:

- **FHWA HEC-25 (3rd ed., 2020), Highways in the Coastal Environment.** This is FHWA's primary coastal-highway/bridge reference, describing how tides, storm surge, and waves affect coastal transportation assets and summarizing methods to estimate water levels, waves, and related processes; it explicitly flags coastal bridge issues such as wave loads on bridge decks and coastal scour as key applications. [109]
- **FHWA HEC-25 Volume 2 (2014), Assessing Extreme Events.** This companion volume (archival; superseded by the 2020 3rd edition) lays out practical "levels of effort" for storm surge and wave assessment, including storm surge and wave modeling workflows, and it discusses hurricane-relevant damage mechanisms such as bridge deck damage by waves on surge. [110]
- **FHWA HEC-18 (5th ed., 2012), Evaluating Scour at Bridges.** HEC-18 provides the standard FHWA procedures for bridge-scour evaluation and risk management (including risk-based approaches and planning/response concepts for scour-critical bridges), which are directly applicable to hurricane-driven floods and debris-laden flows. [72]
- **FHWA HEC-23 (3rd ed.), Bridge Scour and Stream Instability Countermeasures.** HEC-23 focuses on countermeasure selection and design (e.g., for scour and stream instability), consolidating experience and design guidance used by State DOTs to mitigate damage at highway-stream crossings during extreme events. [73]
- **FHWA HEC-20 (4th ed.), Stream Stability at Highway Structures.** HEC-20 provides procedures to diagnose and address stream instability at highway crossings through geomorphic and hydraulic evaluation, reconnaissance, and analysis workflows. These procedures support designs that address channel change and erosion during hurricane flooding [74].
- **FHWA (2017), Vulnerability Assessment and Adaptation Framework (3rd ed.).** This framework is used by State DOTs and partners to structure vulnerability assessments for extreme weather (including hurricanes) and to translate findings into adaptation options and implementation steps. [111]
- **State DOT design manuals (example: FDOT Structures Design Guidelines).** State bridge manuals operationalize hazard-relevant criteria within design practice (e.g., loads and other requirements adopted as additions/modifications to AASHTO LRFD), applying hurricane-prone-region design assumptions across projects. [112]

Representative Case Studies in Texas

- Hurricane Ike Damage to Coastal Bridges (2008)
  - Hurricane Ike produced a very large storm surge (22 ft) and powerful wind-driven waves, severely damaging transportation infrastructure along the upper Texas Gulf Coast. Many low-lying and timber or low-clearance bridges were destroyed or heavily damaged by storm surge and wave forces; scour and debris impact also contributed to structural failures. For example, The Rollover Pass bridge (Highway 87) on Bolivar Peninsula was reduced from three lanes to one lane, with spans shifted or washed out by surge and wave action. Other bridges such as the Pelican Island bridge and Humble

Camp Bridge experienced severe surge and hydrodynamic forces that compromised functionality and closed them for repair. In total, dozens of bridges in the Houston/Galveston region exhibited unseating of deck spans or other forms of damage from surf and surge [113].

- Hurricane Harvey: Highway 96 Bridge Collapse (Village Creek, Silsbee/Jasper Area, TX) (2017)
  - Hurricane Harvey dropped record rainfall (up to 60 inches) and caused catastrophic flooding across Southeast Texas. The U.S. Highway 96 bridge over Village Creek, located between Jasper and Beaumont, collapsed when floodwaters washed out the supporting soils and deck span [114].
- Tropical Storm Imelda: Barge Strike and Damage to I-10 Bridge (2019)
  - Heavy rainfall caused nine barges to break free from their moorings on the San Jacinto River. Multiple barges collided with pier columns supporting the Interstate 10 bridge at Channelview, severely damaging structural protection cells and forcing extended closures. More than 5.4 million USD in repairs were needed [115].

A.6.2 Methods Reported in the Literature

A.6.2.1 FDOT Assessment Procedure [1]

The report formulates hurricane risk for each bridge using a likelihood $\times$ vulnerability framework, which produces a risk index that combines (i) the probability of hurricane hazard at the bridge location and (ii) bridge-specific vulnerability multipliers that proxy the relative consequence of damage and service disruption.

(1) **Hazard likelihood** is derived from FEMA/HAZUS hurricane wind maps by assigning a return period $T_{i,s}$ to each Saffir–Simpson wind category s at bridge i. The return period is converted to an occurrence rate $\lambda_{i,s} = 1/T_{i,s}$, and hurricane arrivals are treated as Poisson. Over a time horizon t (years), the probability of at least one category-s event is

$$p_{i,s}(t) = 1 - \exp(-\lambda_{i,s} t), \qquad \lambda_{i,s} = \frac{1}{T_{i,s}} \tag{A.97}$$

(2) **Vulnerability:** Bridge vulnerability is represented by multiplicative factors obtained from statewide parameter tables: (a) an element vulnerability score (0-5) for each hurricane category, averaged over the set of recorded bridge elements and rescaled to a multiplier, (b) a design/material vulnerability based on NBI superstructure type and material (Items 43A/43B), and additional modifiers for (c) scour criticality (NBI Item 113) and (d) bridge age (from the year built, NBI Item 27). These discrete scores were set primarily by expert judgment informed by historical hurricane damage experience and then converted to multipliers through a linear rescaling.

(3) **Risk index:** For each category s, the report computes a category-specific contribution and combines categories to rank bridges. A representative form is

$$X_{i,s}^{\text{hurr}} = p_{i,s} \cdot V_{i,s}^{\text{elem}} \cdot V_i^{\text{des}} \cdot V_i^{\text{scour}} \cdot V_i^{\text{age}} \tag{A.98}$$

with missing hazard probabilities filled using district/state averages when required. The resulting indices support network-wide prioritization by relative hurricane risk.

Step-by-step procedure **Step 1.** Obtain the bridge inventory (FDOT/Pontis bridge GIS layer) as the baseline asset set for risk computation.

Step 2. Obtain hurricane wind hazard map data from FEMA HAZUS for Florida. The HAZUS GIS layers provide hurricane wind speeds associated with multiple return periods (e.g., 10-1000 years) by census tract.

Step 3. Perform a GIS spatial overlay (spatial join) between the HAZUS hurricane GIS layer and the FDOT bridge GIS layer, so that each bridge inherits the wind-speed V and return-period T of its census tract (if multiple tracts intersect a bridge location, average the wind speeds at each return period).

Step 4. Classify the wind-speed levels at each bridge location into hurricane categories $g \in \{1, 2, 3, 4, 5\}$ using the Saffir–Simpson wind scale (Table A.28). This yields a category-specific return period T_g for category g.

Table A.28: Saffir-Simpson Hurricane Wind Scale (SSHWS) Classifications

Category	V (mph)	V (km/h)	Damage Level	Structural Impact
1	$74 \leq V \leq 95$	$119 \leq V \leq 153$	Very Dangerous	Minimal damage to frame homes.
2	$96 \leq V \leq 110$	$154 \leq V \leq 177$	Extremely Dangerous	Major roof and siding damage.
3	$111 \leq V \leq 129$	$178 \leq V \leq 208$	Devastating	Significant structural damage.
4	$130 \leq V \leq 156$	$209 \leq V \leq 251$	Catastrophic	Roof structure failures likely.
5	$V \geq 157$	$V \geq 252$	Total Destruction	Total collapse of many buildings.

Step 5. Convert each category-specific return period into a Poisson/exponential occurrence model. Define the mean annual arrival rate

$$\lambda_g = \frac{1}{T_g} \tag{A.99}$$

Assuming hurricane-category-g winds at a fixed location follow a Poisson process, the annual probability of at least one occurrence is

$$p_g = 1 - e^{-\lambda_g} \tag{A.100}$$

Step 6. Compute the hurricane element vulnerability for each category g. Let $\mathcal{E}$ be the set of bridge elements recorded for a bridge. For each element $e \in \mathcal{E}_i$, obtain the raw hurricane consequence/vulnerability score (0-5) for category g, denoted s_g, from the hurricane element-parameter tables (the report distinguishes coastal Table A.30 vs. non-coastal tables Table A.29). Define the bridge-level average raw element score for category g as

$$\bar{s}_g = \frac{1}{|\mathcal{E}|} \sum_{e \in \mathcal{E}} s_g \tag{A.101}$$

The element vulnerability score for category g (the report notes scores are commonly shifted by $+1$ and divided by 3 so that a "typical" bridge is around 1.0):

$$V^{\text{elem}}_{\text{Hurricane}}(g) = \frac{\bar{s}_g + 1}{3} \tag{A.102}$$

The combined elementary vulnerability score is

$$V^{\text{elem}}_{\text{Hurricane}} = \sum_{g} V^{\text{elem}}_{\text{Hurricane}}(g) \cdot p_g \tag{A.103}$$

Table A.29: Levels of vulnerability of non-coastal bridge elements to hurricane damage

Table 2.22. Levels of vulnerability of non-coastal bridge elements to hurricane damage

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
12	Bare Concrete Deck	1	1	1	2	2
13	Unp Conc Deck/AC Ovl	1	1	1	2	2
28	Steel Deck/Open Grid	1	1	1	2	2
29	Steel Deck/Conc Grid	1	1	1	2	2
30	Comp/Orthotpc Deck	1	1	1	2	2
31	Timber Deck	1	1	1	2	2
32	Timber Deck/AC Ovly	1	1	1	2	2
38	Bare Concrete Slab	1	1	1	2	2
39	Unp Conc Slab/AC Ovl	1	1	1	2	2
54	Timber Slab	1	1	1	2	2
55	Timber Slab/AC Ovly	1	1	1	2	2
98	Conc Deck on PC Pane	1	1	1	2	2
99	P/S Conc Slab	1	1	1	2	2
101	Unprnt Stl Box Girder	0.5	0.5	0.5	2	2
102	Paint Stl Box Girder	0.5	0.5	0.5	2	2
104	P/S Conc Box Girder	0.5	0.5	0.5	2	2
105	R/Conc Box Girder	0.5	0.5	0.5	2	2
106	Unprnt Stl Opn Girder	0.5	0.5	0.5	2	2
107	Paint Stl Opn Girder	0.5	0.5	0.5	2	2
109	P/S Conc Open Girder	0.5	0.5	0.5	2	2
110	R/Conc Open Girder	0.5	0.5	0.5	2	2
111	Timber Open Girder	0.5	0.5	0.5	2	2
112	Unprnt Stl Stringer	0.5	0.5	0.5	2	2
113	Paint Stl Stringer	0.5	0.5	0.5	2	2
115	P/S Conc Stringer	0.5	0.5	0.5	2	2
116	R/Conc Stringer	0.5	0.5	0.5	2	2
117	Timber Stringer	0.5	0.5	0.5	2	2
120	U/Stl Thru Truss/Bot	0.5	0.5	0.5	2	2
121	P/Stl Thru Truss/Bot	0.5	0.5	0.5	2	2
125	U/Stl Thru Truss/Top	0.5	0.5	0.5	2	2
126	P/Stl Thru Truss/Top	0.5	0.5	0.5	2	2
130	Unprnt Stl Deck Truss	0.5	0.5	0.5	2	2
131	Paint Stl Deck Truss	0.5	0.5	0.5	2	2
135	Timber Truss/Arch	0.5	0.5	0.5	2	2
140	Unprnt Stl Arch	0.5	0.5	0.5	2	2
141	Paint Stl Arch	0.5	0.5	0.5	2	2
143	P/S Conc Arch	0.5	0.5	0.5	2	2
144	R/Conc Arch	0.5	0.5	0.5	2	2
145	Other Arch	0.5	0.5	0.5	2	2
146	Misc Cable Uncoated	0.5	0.5	0.5	2	2
147	Misc Cable Coated	0.5	0.5	0.5	2	2
151	Unprnt Stl Floor Beam	0.5	0.5	0.5	2	2
152	Paint Stl Floor Beam	0.5	0.5	0.5	2	2
154	P/S Conc Floor Beam	0.5	0.5	0.5	2	2
155	R/Conc Floor Beam	0.5	0.5	0.5	2	2
156	Timber Floor Beam	0.5	0.5	0.5	2	2
160	Unprnt Stl Pin/Hanger	0.5	0.5	0.5	2	2
161	Paint Stl Pin/Hanger	0.5	0.5	0.5	2	2
201	Unprnt Stl Column	1	1	1	2	2
202	Paint Stl Column	1	1	1	2	2
204	P/S Conc Column	1	1	1	2	2
205	R/Conc Column	1	1	1	2	2
206	Timber Column	1	1	1	2	2
207	P/S Conc Holl Pile	1	1	1	2	2
210	R/Conc Pier Wall	2	2	2	3	3

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

(a) Part 1

Table 2.22. Levels of vulnerability of non-coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
211	Other Mlt Pier Wall	2	2	2	3	3
215	R/Conc Abutment	2	2	2	3	3
216	Timber Abutment	2	2	2	3	3
217	Other Mlt Abutment	2	2	2	3	3
220	R/C Sub Pile Cap/Ftg	1	1	1	2	2
230	Unprnt Stl Cap	1	1	1	2	2
231	Paint Stl Cap	1	1	1	2	2
233	P/S Conc Cap	1	1	1	2	2
234	R/Conc Cap	1	1	1	2	2
235	Timber Cap	1	1	1	2	2
240	Metal Culvert	3	3	3	4	4
241	Concrete Culvert	3	3	3	4	4
242	Timber Culvert	3	3	3	4	4
243	Misc Culvert	3	3	3	4	4
290	Channel	3	3	3	4	4
298	Pile Jacket Bare	2	2	2	3	3
299	Pile Jacket/Cath Pro	2	2	2	3	3
300	Strip Seal Exp Joint	1	1	1	2	2
301	Pourable Joint Seal	1	1	1	2	2
302	Compressn Joint Seal	1	1	1	2	2
303	Assembly Joint/Seal	1	1	1	2	2
304	Open Expansion Joint	1	1	1	2	2
310	Elastomeric Bearing	1	1	1	2	2
311	Moveable Bearing	1	1	1	2	2
312	Enclosed Bearing	1	1	1	2	2
313	Pot Bearing	1	1	1	2	2
314	Pot Bearing	1	1	1	2	2
315	Disk Bearing	1	1	1	2	2
320	R/Conc Appr Slab	2	2	2	3	3
321	R/Conc Approach Slab	2	2	2	3	3
330	Metal Rail Uncoated	2	2	2	3	3
331	Conc Bridge Railing	2	2	2	3	3
332	Timb Bridge Railing	2	2	2	3	3
333	Other Bridge Railing	2	2	2	3	3
334	Metal Rail Coated	2	2	2	3	3
356	Steel Fatigue Sm/Flag	0	0	0	0	0
357	Pack Rust Sm/t Flag	0	0	0	0	0
358	Deck Cracking Sm/Flag	0	0	0	0	0
359	Soffit Sm/t Flag	0	0	0	0	0
360	Settlement Sm/Flag	0	0	0	0	0
361	Scour Sm/t Flag	0	0	0	0	0
362	Traf Impact Sm/Flag	0	0	0	0	0
363	Section Loss Sm/Flag	0	0	0	0	0
369	Sub.Sect Loss Sm/Flag	0	0	0	0	0
370	Alert Sm/t Flag	0	0	0	0	0
386	Fender/Dolphin Uncoa	2	2	2	3	3
387	P/S Fender/Dolphin	2	2	2	3	3
388	R/Conc Fender/Dolphi	2	2	2	3	3
389	Timber Fender/Dolphi	2	2	2	3	3
390	Other Fender/Dolphin	2	2	2	3	3
393	Bikhd Swwl Metal Unc	2	2	2	3	3
394	R/Conc Abut Slope Pr	2	2	2	3	3
395	Timber Abut Slope Pr	2	2	2	3	3
396	Other Abut Slope Pro	2	2	2	3	3
397	Drain. Syst.Metal	1	1	1	2	2
398	Drain. Syst.Other	1	1	1	2	2
399	Other Xpansion Joint	1	1	1	2	2

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

(b) Part 2

Table 2.22. Levels of vulnerability of non-coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
474	Walls Uncoated	3	3	3	4	4
475	R/Conc Walls	3	3	3	4	4
476	Timber Walls	3	3	3	4	4
477	Other Walls	3	3	3	4	4
478	MSE Walls	3	3	3	4	4
480	Mast Arm Found	4	4	4	5	5
481	Paint Mast Arm Vert	4	4	4	5	5
482	Galvan Mast Arm Vert	4	4	4	5	5
483	Other Mast Arm Vert	4	4	4	5	5
484	Paint Mast Arm Horzn	4	4	4	5	5
485	Galvan Mast Arm Horz	4	4	4	5	5
486	Other Mast Arm Horzn	4	4	4	5	5
487	Sign Member Horiz	4	4	4	5	5
488	Sign Member Vertical	4	4	4	5	5
489	Sign Foundation	5	5	5	5	5
495	Uncoat High Mast L.	4	4	4	5	5
496	Painted High Mast L.	4	4	4	5	5
497	Galvan. High Mast L.	4	4	4	5	5
498	Other High Mast L.P.	4	4	4	5	5
499	H. M. L. P. Found.	4	4	4	5	5
540	Open Gearing	2	2	2	3	3
541	Speed Reducers	2	2	2	3	3
542	Shafts	2	2	2	3	3
543	Shaft Brgs and Coupl	2	2	2	3	3
544	Brakes	2	2	2	3	3
545	Emergency Drive	2	2	2	3	3
546	Span Drive Motors	2	2	2	3	3
547	Hydraulic Power Unit	2	2	2	3	3
548	Hydraulic Piping Sys	2	2	2	3	3
549	Hydraulic Cylinders	2	2	2	3	3
550	Hopkins Frame	2	2	2	3	3
560	Locks	2	2	2	3	3
561	Live Load Shoes	2	2	2	3	3
562	Counterweight/Suppor	2	2	2	3	3
563	Acc Ladd & Pat	2	2	2	3	3
564	Counterweight	2	2	2	3	3
565	Trun/Stir and Cur Trk	2	2	2	3	3
570	Transformers	2	2	2	3	3
571	Submarine Cable	2	2	2	3	3
572	Conduit & Junc. Box	2	2	2	3	3
573	PLCs	2	2	2	3	3
574	Control Console	2	2	2	3	3
580	Navigational Lights	2	2	2	3	3
581	Operator Facilities	5	5	5	5	5
582	Lift Bridge Spec. Eq	2	2	2	3	3
583	Swing Bridge Spec. E	2	2	2	3	3
590	Resistance Barriers	2	2	2	3	3
591	Warning Gates	5	5	5	5	5
592	Traffic Signals	5	5	5	5	5

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

(c) Part 3

Source: [1, Table 2.22]

Step 7. Compute the hurricane design/material vulnerability multiplier. Using the bridge superstructure design type and material codes (NBI item 43), obtain a raw design score $s^{\text{des}}_{\text{Hurricane}} \in \{0, 1, 2, 3, 4, 5\}$ from the hurricane design-parameter table (Table A.31), and rescale:

$$V^{\text{des}}_{\text{Hurricane}} = \frac{s^{\text{des}}_{\text{Hurricane}} + 1}{3} \tag{A.104}$$

Step 8. Compute the hurricane scour vulnerability multiplier. Using scour criticality (NBI item 113), let $\text{NBI113}(i) \in \{0, 1, 2, 3, 4, 5, 6, 7, 8, 9, N, U, T\}$ be the scour code of the bridge. Define the hurricane scour vulnerability multiplier by a two-bin classification [4]:

$$V^{\text{scour}}_{\text{hurricane}} = \begin{cases} 1.2, & \text{NBI113} \in \{0, 1, 2, 3, 4, 6, U\}, \\ 0.4, & \text{NBI113} \in \{5, 7, 8, 9, N, T\}. \end{cases} \tag{A.105}$$

Step 9. Compute the hurricane age vulnerability multiplier. Let Y be the year the bridge was built (NBI item 27). The report models age vulnerability as a linear function of age:

$$V^{\text{age}}_{\text{Hurricane}} = 1 + \frac{1990 - Y}{35} \tag{A.106}$$

Step 10. Compute the hurricane risk index and rank bridges. The report defines hurricane likelihood by category ($p_{H,s}(i)$) and combines it with (i) category-specific element vulnerability and (ii) bridge-level mul-

[4] The two-level values of 0.4 and 1.2 and the binning are inferred from the parameter-table examples in the FDOT report.

Table A.30: Levels of vulnerability of coastal bridge elements to hurricane damage

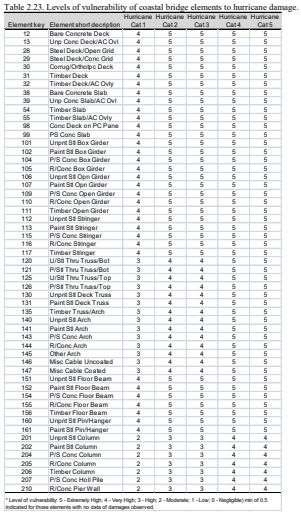


Table 2.23. Levels of vulnerability of coastal bridge elements to hurricane damage.

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
12	Bare Concrete Deck	4	5	5	5	5
13	Unp Conc Deck/AC Ovl	4	5	5	5	5
28	Steel Deck/Open Grid	4	5	5	5	5
29	Steel Deck/Conc Grid	4	5	5	5	5
30	Comp/Orthotpc Deck	4	5	5	5	5
31	Timber Deck	4	5	5	5	5
32	Timber Deck/AC Ovly	4	5	5	5	5
38	Bare Concrete Slab	4	5	5	5	5
39	Unp Conc Slab/AC Ovl	4	5	5	5	5
54	Timber Slab	4	5	5	5	5
55	Timber Slab/AC Ovly	4	5	5	5	5
98	Conc Deck on PC Pane	4	5	5	5	5
99	P/S Conc Slab	4	5	5	5	5
101	Unprt Stl Box Girder	4	5	5	5	5
102	Paint Stl Box Girder	4	5	5	5	5
104	P/S Conc Box Girder	4	5	5	5	5
105	R/Conc Box Girder	4	5	5	5	5
106	Unprt Stl Opn Girder	4	5	5	5	5
107	Paint Stl Opn Girder	4	5	5	5	5
109	P/S Conc Open Girder	4	5	5	5	5
110	R/Conc Open Girder	4	5	5	5	5
111	Timber Open Girder	4	5	5	5	5
112	Unprt Stl Stringer	4	5	5	5	5
113	Paint Stl Stringer	4	5	5	5	5
115	P/S Conc Stringer	4	5	5	5	5
116	R/Conc Stringer	4	5	5	5	5
117	Timber Stringer	4	5	5	5	5
120	U/Stl Thru Truss/Bot	3	4	4	5	5
121	P/Stl Thru Truss/Bot	3	4	4	5	5
125	U/Stl Thru Truss/Top	3	4	4	5	5
126	P/Stl Thru Truss/Top	3	4	4	5	5
130	Unprt Stl Deck Truss	3	4	4	5	5
131	Paint Stl Deck Truss	3	4	4	5	5
135	Timber Truss/Arch	3	4	4	5	5
140	Unprt Stl Arch	3	4	4	5	5
141	Paint Stl Arch	3	4	4	5	5
143	P/S Conc Arch	3	4	4	5	5
144	R/Conc Arch	3	4	4	5	5
145	Other Arch	3	4	4	5	5
146	Misc Cable Uncoated	3	4	4	5	5
147	Misc Cable Coated	3	4	4	5	5
151	Unprt Stl Floor Beam	4	5	5	5	5
152	Paint Stl Floor Beam	4	5	5	5	5
154	P/S Conc Floor Beam	4	5	5	5	5
155	R/Conc Floor Beam	4	5	5	5	5
156	Timber Floor Beam	4	5	5	5	5
160	Unprt Stl Pin/Hanger	4	5	5	5	5
161	Paint Stl Pin/Hanger	4	5	5	5	5
201	Unprt Stl Column	2	3	3	4	4
202	Paint Stl Column	2	3	3	4	4
204	P/S Conc Column	2	3	3	4	4
205	R/Conc Column	2	3	3	4	4
206	Timber Column	2	3	3	4	4
207	P/S Conc Holl Pile	2	3	3	4	4
210	R/Conc Pier Wall	2	3	3	4	4

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

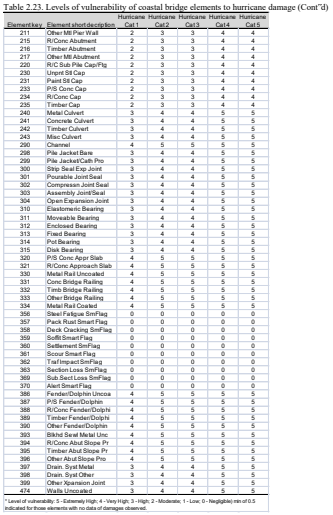


Table 2.23. Levels of vulnerability of coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
211	Other Mtl Pier Wall	2	3	3	4	4
215	R/Conc Abutment	2	3	3	4	4
216	Timber Abutment	2	3	3	4	4
217	Other Mtl Abutment	2	3	3	4	4
220	R/C Sub Pile Cap/Ftg	2	3	3	4	4
230	Unprt Stl Cap	2	3	3	4	4
231	Paint Stl Cap	2	3	3	4	4
233	P/S Conc Cap	2	3	3	4	4
234	R/Conc Cap	2	3	3	4	4
235	Timber Cap	2	3	3	4	4
240	Metal Culvert	3	4	4	5	5
241	Concrete Culvert	3	4	4	5	5
242	Timber Culvert	3	4	4	5	5
243	Misc Culvert	3	4	4	5	5
290	Channel	4	5	5	5	5
298	Pile Jacket Bare	3	4	4	5	5
299	Pile Jacket/Cath Pro	3	4	4	5	5
300	Strip Seal Exp Joint	3	4	4	5	5
301	Pourable Joint Seal	3	4	4	5	5
302	Compresion Joint Seal	3	4	4	5	5
303	Assembly Joint/Seal	3	4	4	5	5
304	Open Expansion Joint	3	4	4	5	5
310	Elastomeric Bearing	3	4	4	5	5
311	Moveable Bearing	3	4	4	5	5
312	Enclosed Bearing	3	4	4	5	5
313	Fixed Bearing	3	4	4	5	5
314	Pot Bearing	3	4	4	5	5
315	Disk Bearing	3	4	4	5	5
320	P/S Conc Appr Slab	4	5	5	5	5
321	R/Conc Approach Slab	4	5	5	5	5
330	Metal Rail Uncoated	4	5	5	5	5
331	Conc Bridge Railing	4	5	5	5	5
332	Timb Bridge Railing	4	5	5	5	5
333	Other Bridge Railing	4	5	5	5	5
334	Metal Rail Coated	4	5	5	5	5
356	Steel Fatigue SmFlag	0	0	0	0	0
357	Pack Rust Smart Flag	0	0	0	0	0
358	Deck Cracking SmFlag	0	0	0	0	0
359	Soffit Smart Flag	0	0	0	0	0
360	Settlement SmFlag	0	0	0	0	0
361	Scour Smart Flag	0	0	0	0	0
362	Traf Impact SmFlag	0	0	0	0	0
363	Section Loss SmFlag	0	0	0	0	0
369	Sub.Sect Loss SmFlag	0	0	0	0	0
370	Alert Smart Flag	0	0	0	0	0
386	Fender/Dolphin Uncoa	4	5	5	5	5
387	P/S Fender/Dolphin	4	5	5	5	5
388	R/Conc Fender/Dolphi	4	5	5	5	5
389	Timber Fender/Dolphi	4	5	5	5	5
390	Other Fender/Dolphin	4	5	5	5	5
393	Bkhd Seal Metal Unc	4	5	5	5	5
394	R/Conc Abut Slope Pr	4	5	5	5	5
395	Timber Abut Slope Pr	4	5	5	5	5
396	Other Abut Slope Pro	4	5	5	5	5
397	Drain. Syst Metal	3	4	4	5	5
398	Drain. Syst Other	3	4	4	5	5
399	Other Xpansion Joint	3	4	4	5	5
474	Walls Uncoated	3	4	4	5	5

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

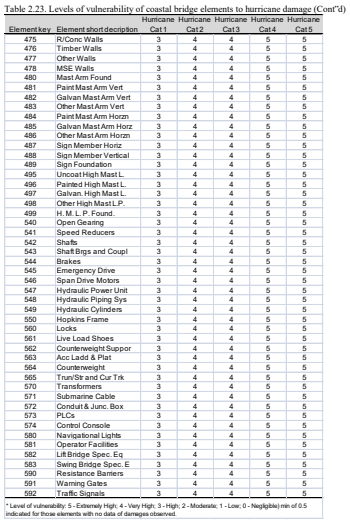


Table 2.23. Levels of vulnerability of coastal bridge elements to hurricane damage (Cont'd)

Element key	Element short decription	Hurricane Cat 1	Hurricane Cat 2	Hurricane Cat 3	Hurricane Cat 4	Hurricane Cat 5
475	R/Conc Walls	3	4	4	5	5
476	Timber Walls	3	4	4	5	5
477	Other Walls	3	4	4	5	5
478	MSE Walls	3	4	4	5	5
480	Mast Arm Found	3	4	4	5	5
481	Paint Mast Arm Vert	3	4	4	5	5
482	Galvan Mast Arm Vert	3	4	4	5	5
483	Other Mast Arm Vert	3	4	4	5	5
484	Paint Mast Arm Horzn	3	4	4	5	5
485	Galvan Mast Arm Horz	3	4	4	5	5
486	Other Mast Arm Horzn	3	4	4	5	5
487	Sign Member Horiz	3	4	4	5	5
488	Sign Member Vertical	3	4	4	5	5
489	Sign Foundation	3	4	4	5	5
495	Uncoat High Mast L.	3	4	4	5	5
496	Painted High Mast L.	3	4	4	5	5
497	Galvan. High Mast L.	3	4	4	5	5
498	Other High Mast L.P.	3	4	4	5	5
499	H. M. L. P. Found.	3	4	4	5	5
540	Open Gearing	3	4	4	5	5
541	Speed Reducers	3	4	4	5	5
542	Shafts	3	4	4	5	5
543	Shaft Bngs and Coupl	3	4	4	5	5
544	Brakes	3	4	4	5	5
545	Emergency Drive	3	4	4	5	5
546	Span Drive Motors	3	4	4	5	5
547	Hydraulic Power Unit	3	4	4	5	5
548	Hydraulic Piping Sys	3	4	4	5	5
549	Hydraulic Cylinders	3	4	4	5	5
550	Hopkins Frame	3	4	4	5	5
560	Locks	3	4	4	5	5
561	Live Load Shoes	3	4	4	5	5
562	Counterweight Suppor	3	4	4	5	5
563	Acc Ladd & Plat	3	4	4	5	5
564	Counterweight	3	4	4	5	5
565	Trun/Str and Cur Trk	3	4	4	5	5
570	Transformers	3	4	4	5	5
571	Submarine Cable	3	4	4	5	5
572	Conduit & Junc. Box	3	4	4	5	5
573	PLCs	3	4	4	5	5
574	Control Console	3	4	4	5	5
580	Navigational Lights	3	4	4	5	5
581	Operator Facilities	3	4	4	5	5
582	Lift Bridge Spec. Eq	3	4	4	5	5
583	Swing Bridge Spec. E	3	4	4	5	5
590	Resistance Barriers	3	4	4	5	5
591	Warning Gates	3	4	4	5	5
592	Traffic Signals	3	4	4	5	5

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible) min of 0.5 indicated for those elements with no data of damages observed.

(a) Part 1 **(b)** Part 2 **(c)** Part 3

Source: [1, Table 2.23]

Table A.31: General vulnerability to hurricane damage for coastal bridges by design and material types

Table 2.24. General vulnerability to hurricane damage for coastal bridges by design and material types

	Superstructure material type								
Superstructure design type	Concrete	Concrete Continuous	Steel	Steel Continuous	Prestressed concrete	Prestressed concrete continuous	Wood or Timber	Masonry	Aluminum, Wrought Iron, or Cast Iron
Slab	5	3			5	3	5		
Stringer/MultiBeam/Girder	5	3	3	2	5	3	5		
Girder & Floorbeam	5	3	3	2	5	3	5		
Tee Beam	5	3	3	2	5	3	5		
Box Beam or Girders - Multiple	5	3	3	2	5	3	5		
Box Beam/Girders - Single or Spread	5	3	3	2	5	3	5		
Frame (except frame culverts)									
Orthotropic									
Truss - Deck									
Truss - Thru									
Arch - Deck								2	
Arch - Thru								2	
Suspension				2					
Stayed Girder				2					
Movable - Lift			4	3					
Movable - Bascule			4	3					
Movable - Swing			4	3					
Tunnel									
Culvert	4		4		4		4	4	4
Mixed Types									
Segmental Box Girder	4								
Channel Beam	4								
	* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible)								

Source: [1, Table 2.24]

tipliers for design, scour, and age. The hurricane risk index is

$$\text{Risk}_{\text{hurricane}} = V^{\text{elem}}_{\text{Hurricane}}\, V^{\text{des}}_{\text{Hurricane}}\, V^{\text{scour}}_{\text{Hurricane}}\, V^{\text{age}}_{\text{Hurricane}} \tag{A.107}$$

Rank bridges in descending order of $\text{Risk}_{\text{hurricane}}(i)$ (larger value means higher relative hurricane risk).

Agency cost Compute replacement cost R_i and allocate the statewide annual control total for hurricane agency losses $L_{\text{hurricane}}$ (Table A.15) using the hurricane risk index as the allocation driver:

$$R_i = A_i c \tag{A.108}$$

$$w_i = \frac{R_i \cdot \text{Risk}_{\text{hurricane}}(i)}{\sum_{j \in \mathcal{E}} R_j \cdot \text{Risk}_{\text{hurricane}}(j)} \tag{A.109}$$

$$\text{AgencyCost}_{\text{hurricane}}(i) = L_{\text{hurricane}}\, w_i \tag{A.110}$$

User cost Define a daily detour cost proxy U_i (e.g., from ADT, detour length ℓ_i, detour speed v_i, operating cost c_{op}, value of time c_{time}), then allocate the statewide annual control total for hurricane user losses $L^U_{\text{hurricane}}$:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}}\, \frac{\ell_i}{v_i} \right) \tag{A.111}$$

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{hurricane}}(i)}{\sum_{j \in \mathcal{B}} U_j \cdot \text{Risk}_{\text{hurricane}}(j)} \tag{A.112}$$

$$\text{UserCost}_{\text{hurricane}}(i) = L^U_{\text{hurricane}}\, u_i \tag{A.113}$$

Total cost:

$$\text{TotalCost}_{\text{hurricane}}(i) = \text{AgencyCost}_{\text{hurricane}}(i) + \text{UserCost}_{\text{hurricane}}(i) \tag{A.114}$$

A.6.2.2 HEC-25 Vol. 2 [9]

HEC-25 Vol. 2 defines a three-level ("levels of effort") framework for assessing exposure of coastal transportation infrastructure to extreme events and climate change. Level of Effort 1 is intended as a rapid, planning/screening-level approach that relies on existing inundation products (primarily FEMA flood hazard maps / FIRMs) to estimate exposure to: (i) flood depth, (ii) depth-limited wave height, (iii) wave crest elevation, and (iv) flood flow velocity.

Step-by-step procedure (Level 1) **Step 1. Obtain FEMA flood map elevations (baseline).** From FEMA FIRMs / Flood Insurance Study products, extract the baseline still water elevation field for the design event (typically the 1% annual chance / 100-year coastal flood still water elevation), referenced to a fixed vertical datum (e.g., NAVD88). Denote the baseline still water elevation as

$$\eta_{\text{SWL},0}(\mathbf{x}) \quad [\text{L}] \tag{A.115}$$

where:

- $\eta_{\mathrm{SWL},0}(\mathbf{x})$ is the baseline still water elevation at horizontal location $\mathbf{x}$ for the selected FEMA coastal flood event. "Still water elevation" here refers to the water surface elevation due to storm surge (and tide, if included in the FEMA product) excluding wave setup, wave runup, and the oscillatory wave component. Units are length (e.g., ft or m).
- The subscript "SWL" denotes still water level. The subscript "0" denotes the baseline (present-day / FEMA-mapped) condition before adding any sea level change scenario.
- $\mathbf{x}$ is the plan-view location (horizontal coordinate) at which the elevation is evaluated, typically a 2D coordinate in a projected system, e.g.,

  $$\mathbf{x} = (x, y) \tag{A.116}$$

  where x and y are the easting and northing coordinates (units of length) in the chosen map projection.
- $[\mathrm{L}]$ indicates that the quantity has the dimension of length. In implementation, the same units must be used consistently across all elevation datasets (e.g., all in ft NAVD88 or all in m NAVD88).

Step 2. **Choose sea level change scenarios.** Select a set of relative sea level change increments for planning horizons of interest (e.g., 2030/2050/2100). For scenario s, denote the sea level change increment as

$$\Delta\eta_{\mathrm{SLR},s} \quad [\mathrm{L}] \tag{A.117}$$

Step 3. **Account for nonlinear SLR–surge coupling (screening option).** HEC-25 Vol. 2 explains that the storm-surge response to sea level rise (SLR) can be nonlinear; therefore, at Level 1 the user may account for this nonlinearity by multiplying each selected relative sea level change increment by a constant factor ranging from 1 to 5 (FHWA HEC-25 Vol. 2, Sec. 4.2.1.2).

Define the adjusted (effective) sea level change increment for scenario s as

$$\Delta\eta^{*}_{\mathrm{SLR},s} = k_{\mathrm{nl}}\,\Delta\eta_{\mathrm{SLR},s}, \qquad k_{\mathrm{nl}} \in [1, 5] \tag{A.118}$$

where:

- $\Delta\eta_{\mathrm{SLR},s}$ is the selected relative sea level change increment for scenario s (units of length, e.g., ft or m). It represents the change in mean sea level relative to the present-day baseline, and in this manual it is intended to include both global eustatic sea level rise and local vertical land motion (subsidence/uplift) through a relative sea level rise scenario.
- The subscript "SLR" denotes sea level rise (more generally, relative sea level change).
- The index s labels the scenario (e.g., a planning horizon such as 2050 or 2100, or a low/medium/high SLR pathway).
- k_{nl} is a dimensionless nonlinearity multiplier used to represent the possibility that the incremental change in storm surge elevation can be greater or less than the sea level change increment. In the HEC-25 Vol. 2 Level 1 guidance, k_{nl} is allowed to range from 1 to 5, where:
  - $k_{\mathrm{nl}} = 1$ represents a **linear (one-to-one) coupling** between SLR and surge increase;
  - $k_{\mathrm{nl}} = 5$ represents a **strongly nonlinear coupling** in which the incremental surge increase is five times the sea level change increment.

- $\Delta\eta^{*}_{\mathrm{SLR},s}$ is the adjusted (effective) sea level change increment used in subsequent Level 1 computations of future flood levels (units of length).

For rapid Level 1 screening, a reasonable bracket is

$$k_{\mathrm{nl}} \in [1, 2] \tag{A.119}$$

Use $k_{\mathrm{nl}} = 1$ for a static additive treatment. A value of $k_{\mathrm{nl}} \approx 1.5$ represents the nonlinear amplification in shallow coastal settings described in the cited literature.

Step 4. **Compute scenario still water elevations.** For each scenario s, compute the modified still water elevation surface by adding the adjusted sea level increment to the FEMA still water elevation:

$$\eta_{\mathrm{SWL},s}(\mathbf{x}) = \eta_{\mathrm{SWL},0}(\mathbf{x}) + \Delta\eta^{*}_{\mathrm{SLR},s} \tag{A.120}$$

Step 5. **Obtain ground surface elevation data.** Assemble a ground (terrain) elevation surface referenced to the same vertical datum as the FEMA elevations. Denote the ground elevation as

$$z_g(\mathbf{x}) \quad [\mathrm{L}] \tag{A.121}$$

from sources such as USGS topography and/or LiDAR-derived DEMs (preferred for higher resolution).

Step 6. **Compute flood depth.** Compute the flood depth for scenario s as the difference between still water elevation and ground elevation:

$$d_s(\mathbf{x}) = \max\left\{0,\, \eta_{\mathrm{SWL},s}(\mathbf{x}) - z_g(\mathbf{x})\right\} \tag{A.122}$$

Step 7. **Estimate maximum depth-limited wave height (Level 1).** At Level 1, HEC-25 Vol. 2 approximates the maximum depth-limited wave height as a fixed fraction of the local flood depth. Specifically, the manual uses the Level 1 coefficient 0.8 and sets:

$$H_{\mathrm{max},s}(\mathbf{x}) = 0.8\, d_s(\mathbf{x}) \tag{A.123}$$

where:

- $H_{\mathrm{max},s}(\mathbf{x})$ is the maximum depth-limited wave height at plan location $\mathbf{x}$ under scenario s (units of length, e.g., ft or m). In this Level 1 screening context, it represents an estimate of the largest wave height that can be sustained by the local water depth (i.e., waves cannot grow arbitrarily large in shallow water because they break when they become too steep relative to depth).
- The coefficient 0.8 is a dimensionless depth-limited wave-height factor specified for Level 1 screening in HEC-25 Vol. 2 (Gulf of Mexico / South Atlantic Coast). It encodes the shallow-water breaking limit assumption that the maximum wave height is proportional to the local water depth:

  $$\frac{H_{\mathrm{max}}}{d} \approx 0.8 \tag{A.124}$$

  At Level 1, the manual assumes that, once waves are depth-limited by breaking, their height is approximately 80% of the available water depth.

Step 8. **Estimate wave crest elevation (Level 1).** At Level 1, HEC-25 Vol. 2 approximates the wave crest

elevation as the still water elevation plus a fixed fraction of the (depth-limited) wave height:

$$\eta_{\text{crest},s}(\mathbf{x}) = \eta_{\text{SWL},s}(\mathbf{x}) + 0.75\, H_{\text{max},s}(\mathbf{x}) \tag{A.125}$$

where:

- $\eta_{\text{crest},s}(\mathbf{x})$ is the wave crest elevation at plan location $\mathbf{x}$ for scenario s. It is the estimated instantaneous elevation of the top of a wave crest above the chosen vertical datum (e.g., NAVD88) during the coastal flood event. In screening, it represents a conservative "peak" water surface that may interact with bridge low chords, deck soffits, approach fills, and other vulnerable components.
- The subscript "crest" denotes wave crest. The index s denotes the scenario (e.g., SLR planning horizon).
- $\eta_{\text{SWL},s}(\mathbf{x})$ is the still water elevation under scenario s at location $\mathbf{x}$ (storm surge + tide as represented in the FEMA product, excluding the oscillatory wave component).
- $H_{\text{max},s}(\mathbf{x})$ is the maximum depth-limited wave height under scenario s at location $\mathbf{x}$ (from Step 7, $H_{\text{max},s} = 0.8\, d_s$).
- The coefficient 0.75 is a dimensionless crest-elevation factor specified for Level 1 screening in HEC-25 Vol. 2 (Gulf of Mexico / South Atlantic Coast). It provides a simple mapping from wave height to an estimate of the crest elevation above still water:

  $$\eta_{\text{crest}} - \eta_{\text{SWL}} \approx 0.75 H_{\text{max}} \tag{A.126}$$

  Physically, this factor is an empirical screening assumption about wave shape / asymmetry in shallow water (i.e., the crest is higher above the mean water surface than a perfectly symmetric sinusoidal wave would imply). In a purely sinusoidal wave, the crest elevation above mean water level would be $H/2 = 0.5H$; the Level 1 choice of $0.75H$ is therefore more conservative for identifying potential wave-structure interaction (e.g., whether crests may reach the bridge deck).

Step 9. **Estimate flood flow velocity bounds.**

HEC-25 Vol. 2 provides a simple screening approach to bound possible flood flow velocities using only the local flood depth. Define the lower- and upper-bound velocities for scenario s at plan location $\mathbf{x}$ as

$$V_{\text{low},s}(\mathbf{x}) = \frac{d_s(\mathbf{x})}{1\ \text{s}}, \tag{A.127}$$

$$V_{\text{high},s}(\mathbf{x}) = \sqrt{g\, d_s(\mathbf{x})} \tag{A.128}$$

where:

- $V_{\text{low},s}(\mathbf{x})$ is the lower-bound estimate of flood flow velocity at location $\mathbf{x}$ under scenario s (units of length per time, e.g., m/s or ft/s). This bound is a Level 1 screening construct that converts depth to a velocity scale by dividing by a 1-second time scale.
- $V_{\text{high},s}(\mathbf{x})$ is the upper-bound estimate of flood flow velocity at location $\mathbf{x}$ under scenario s (units of length per time). This bound is based on a gravity-wave (shallow-water) celerity scale.
- 1 s is a fixed reference time interval of one second used only to set a minimal velocity scale for screening. Dividing by 1 s preserves the correct units for velocity.

- g is the gravitational acceleration (units of length per time squared), e.g.,

$$g = 9.81 \text{ m/s}^2 \quad \text{or} \quad g = 32.2 \text{ ft/s}^2 \tag{A.129}$$

  and must be consistent with the unit system used for d_s.

In this Level 1 screening interpretation, $V_{\text{low},s}$ and $V_{\text{high},s}$ provide a depth-based velocity range for exposure mapping and prioritization. Detailed current modeling supplies site-specific hydrodynamic velocities at higher levels of analysis.

Step 10. **Consider shoreline retreat and erosion (context adjustment).** Adjust the coastal exposure interpretation using historic trends in shoreline retreat and erosion to supplement the Level 1 inundation products with shoreline-change information.

Step 11. **Map the damaging coastal processes.** For each scenario s, produce GIS layers (rasters/contours) for the key exposure variables:

$$\left\{ d_s(\mathbf{x}),\ H_{\text{max},s}(\mathbf{x}),\ \eta_{\text{crest},s}(\mathbf{x}),\ V_{\text{low},s}(\mathbf{x}),\ V_{\text{high},s}(\mathbf{x}) \right\} \tag{A.130}$$

Step 12. **Evaluate exposure of bridge inventory overlay.** Overlay the mapped hazard layers with the bridge asset inventory. For each bridge asset i with location $\mathbf{x}_i$, extract exposure metrics such as

$$d_{s,i} = d_s(\mathbf{x}_i), \quad H_{\text{max},s,i} = H_{\text{max},s}(\mathbf{x}_i), \quad \eta_{\text{crest},s,i} = \eta_{\text{crest},s}(\mathbf{x}_i) \tag{A.131}$$

(and similarly for velocity bounds), to support rapid screening and prioritization under each sea level scenario.

Level 1 exposure metrics are intended to function as a screening tool to identify exposed areas or infrastructure assets and to support follow-on evaluation with refined Level 1 analyses or higher Levels of Effort (HEC-25 Vol. 2, Sec. 4.1.1). Accordingly, after Step 12 extracts bridge-level exposure attributes under scenario s (e.g., inundation depth $d_{s,i}$, depth-limited wave height $H_{\text{max},s,i}$, wave crest elevation $\eta_{\text{crest},s,i}$, and velocity bounds $V_{\text{low},s,i}$ and $V_{\text{high},s,i}$), these metrics can be used to screen the bridge inventory by ranking bridges according to hazard intensity. A practical statewide screening implementation is to flag bridges in the upper tail of the exposure distributions (e.g., the top 5-10%) as high-exposure candidates for follow-on assessment.

A.7 Landslide Risk

A.7.1 Introduction

Landslide risk refers to the probability and consequences of slope instability events that threaten bridge foundations, substructures, or approaches, potentially leading to loss of serviceability or structural failure.

Table A.32: Selected U.S. bridge damage/destruction events linked to landslides, mudslides, or debris flows (2021-2025).

Date	Bridge	Location	Landslide trigger / type	Impact
Summer 2021	Hanging Lake Trail bridges (7 total)	Glenwood Canyon, Colorado	Heavy rain on the Grizzly Creek Fire burn scar mobilized sediment and burned vegetation, producing mud/debris flows. [116]	Multiple trail bridges damaged; later replacement/reconstruction.
Jul 2022	Bridge/culvert across Manuelitas Creek	Near the Hermit's Peak–Calf Canyon burn area, New Mexico	Post-wildfire storms produced flooding with high debris loads (mud/woody debris), washing out crossings. [117]	Bridge/culvert washed out (reported as one of many damaged crossings).
2023-05-14	Spirit Lake Outlet Bridge (SR 504, MP 49)	Near Johnston Ridge Observatory (Mount St. Helens area), Washington	Large landslide/debris slide (snowmelt/unstable slope conditions) impacted the corridor. [118, 119]	Bridge washed out and highway segment destroyed; long-term closure and major repair project.
2024-09-25–28	Multiple bridges (79 reported damaged)	Western North Carolina	Hurricane Helene rainfall triggered widespread landslides (many debris flows) and associated flooding/scour effects near bridges. [120]	Region-wide bridge damage (reported: 79 bridges damaged).
Dec 2025	Bridge in the Lytle Creek community (name not reported in source)	San Bernardino County, California	Atmospheric-river winter storms caused mud/debris movement and destructive flow in steep terrain. [121]	Bridge destroyed during the storm impacts.

Representative Case Studies in Texas

- West Fifth Street and Shoal Creek Landslide Concerns – Austin Area (2018)
  - Heavy rainfall around Shoal Creek in downtown Austin caused slope collapses and bluff failures along creek banks. these landslides caused significant erosion adjacent to trails and infrastructure near bridges (including embankments that support creek crossings). The unstable geology (Buda limestone over Del Rio clay) led to progressive slope movement that threatened nearby structures, including bike path bridges and utilities [122].
- Slope/Riprap Failures on Multiple Bridge Approaches (2023)
  - The technical report published in 2023 documented a number of slope instability and riprap slope failures near bridge sites in a research compilation of case histories. These include cases where the slope or riprap protecting the slope beneath a bridge approach failed, potentially undermining the bridge support, including [123]:
    - US 59 over Morgan Creek (Lufkin District): Riprap slope covering the embankment under the bridge lost stability due to erosion or saturation, requiring slope rework and stabilization.
    - FM 982 over Tickey Creek (Dallas District): Riprap slope failure threatening stability of the embankment near the bridge.
    - IH 35 NBFR over Cobb Creek (Waco District): Riprap layer on a 2:1 slope failed, necessitating remediation.
    - FM 218 over Cowhouse Creek (Waco District) and FM 667 at Cobb Hollow Creek (Wichita Falls District): Additional slope problems around bridge approaches documented.

A.7.2 Methods Reported in the Literature

A.7.2.1 Italian Bridge Guidelines (MIT-CSLP, 2020) [10]

This guideline, issued by the Italian Ministry of Infrastructure and Transport (MIT) and prepared under the Consiglio Superiore dei Lavori Pubblici (CSLP, the national High Council of Public Works), provides a nationwide framework for the classification and management of risk, safety assessment, and monitoring of existing bridges. It defines a network-level screening procedure to rank bridges by Class of Attention for landslide risk. The method is class-based (logic operators and lookup matrices), and uses susceptibility rather than a fully time-dependent hazard probability.

Overview. For each bridge i, compute three ordinal factors:

- Landslide **susceptibility** class S_i (spatial propensity of slope instability affecting the bridge),
- **vulnerability** class V_i (structural resistance and degree of interaction with the landslide),
- **exposure** class E_i (traffic/importance and consequences).

Then determine the landslide **Class of Attention** $\mathrm{CdA}_i^{\mathrm{frane}}$ by combining (S_i, V_i, E_i) through the guideline's lookup matrix (Table 4.18).

Step-by-step procedure **Step 1. (Asset baseline)** Obtain the bridge inventory as the baseline asset set, including: location, basic geometry (e.g., mean span length), structural typology, foundation typology, and

traffic data.

Step 2 (Collect landslide screening inputs). Step 2 collects landslide-specific screening inputs for each bridge, beyond the standard bridge inventory.

- Compile territorial (area-wide) geohazard information for the bridge site:
  - Landslide inventory records and mapped landslide polygons in the vicinity of the bridge.
  - Landslide susceptibility zoning and any published slope-instability studies for the corridor.
  - Geomorphologic, geologic, and terrain information (e.g., slope morphology, materials, structural controls) relevant to instability.
  - Hydrogeologic and drainage context (surface drainage patterns, groundwater indicators, known erosion/scour at the slope toe).
- Perform a rapid visual check and update the desk-study interpretation:
  - Evidence of past or ongoing instability: scarps, tension cracks, bulging, displaced retaining elements, misalignments, settlements.
  - Water-related triggers or aggravating factors: seepage, wet zones, blocked/damaged drains, concentrated runoff, toe erosion.
  - Identify which bridge components could be affected and how: abutments, approach embankments, piers; and whether the potential impact is localized or could involve the whole bridge.
- Record the observations and source information in a standardized form so that the same parameters are captured consistently across the network:
  - Susceptibility descriptors (presence/potential vs recognized landslide; qualitative activity/criticality; expected magnitude and velocity classes).
  - Data quality and uncertainty (source type, completeness, and date).
  - Existing mitigation and monitoring measures (presence/absence and apparent condition).
  - Exposure-relevant context for later classification (e.g., traffic class, availability of alternatives, strategic importance).

The output of Step 2 is a consistent set of landslide screening parameters for each bridge, which enables subsequent classification of susceptibility, vulnerability, exposure, and the resulting landslide Class of Attention.

Step 3 (Screen-out check) determines whether the bridge has a landslide interaction that warrants scoring.

Screen out the bridge only when the available information supports the conclusion that no landslide process can reasonably affect the bridge or its approaches. This requires the following conditions to be simultaneously satisfied:

- No relevant slope setting: the bridge and its approaches are not adjacent to natural valley-side slopes, cut slopes, or high embankments that could produce or transmit slope movements toward the bridge.
- No mapped or documented landslide influence: the bridge is not located within, nor immediately affected by, any mapped landslide body or landslide-prone area in the available territorial datasets (inventories, zoning, prior studies).
- No observable evidence of instability: there are no visible indicators of past or ongoing ground movement near the bridge, such as scarps, tension cracks, bulging ground, displaced retaining elements, abnormal settlements, or structural misalignments.

- No field evidence of adverse water conditions that could trigger instability: no seepage zones, concentrated runoff, toe erosion, or dysfunctional drainage that would suggest a plausible instability mechanism affecting the bridge.

If any one of these conditions is not satisfied, the bridge is not screened out and the procedure proceeds to assign susceptibility, vulnerability, and exposure classes and then compute the bridge's landslide risk category for ranking and prioritization.

Step 4. (Primary susceptibility parameters) For bridges not screened out, determine three primary parameters:

- activity state for recognized landslides P_A (or criticality for potential landslides P_C),
- maximum potential velocity P_V,
- magnitude as mobilizable volume P_M.

Assign numerical scores to P_A/P_C, P_V, and P_M following the guideline's rating table (Table A.33).

Table A.33: Values for landslide susceptibility parameters (Source: [10, Table 4.16]

(A) Activity state for recognized landslides, or degree of criticality for potential landslides					
	Active / Highly critical		Inactive / Critical		Stabilized / Slightly critical
Recognized (P_A)	Signs of ongoing movement		No movement for cycles		Stabilized
Potential (P_C)	Highly critical		Critical		Slightly critical
Score (P_A, P_C)	**5**		**3**		**1**
(B) Maximum expected velocity (P_V)					
Velocity	$>$ 3 m/min	3 m/min – 1.8 m/h	1.8 m/h – 13 m/mo	13 m/mo – 1.6 m/y	$<$ 1.6 m/y
Description	Extr./Very rapid	Rapid	Moderate	Slow	Extr./Very slow
Score	**5**	**4**	**3**	**2**	**1**
(C) Expected magnitude based on volume (P_M)					
Volume (m^3)	$> 10^6$	2.5×10^5–10^6	10^4–2.5×10^5	10^2–10^4	$< 5 \times 10^2$
Description	Extr./Very large	Large	Medium	Small	Very small
Score	**15**	**12**	**9**	**6**	**3**

Step 5. (Slope-instability score) Compute the slope-instability score P_i as

$$P_i = \begin{cases} P_A + P_M + P_V, & \text{recognized landslide,} \\ P_C + P_M + P_V, & \text{potential landslide.} \end{cases} \tag{A.132}$$

Step 6 maps the score P_i to a base susceptibility class $S_i^{(0)}$ using the instability classification:

$$S_i^{(0)} = \begin{cases} \text{Very High}, & 20 \le P_i \le 25, \\ \text{High}, & 16 \le P_i \le 19, \\ \text{Medium}, & 12 \le P_i \le 15, \\ \text{Low}, & 8 \le P_i \le 11, \\ \text{Very Low}, & 5 \le P_i \le 7. \end{cases} \tag{A.133}$$

The resulting class $S_i^{(0)}$ is the bridge's base landslide susceptibility category. It indicates how prone the surrounding slope setting is to producing a landslide that could involve the bridge (from Very Low to Very High), and it is used to prioritize bridges for further evaluation: bridges in higher classes are treated as more urgent candidates for detailed assessment, monitoring, or mitigation.

Step 7. (Secondary susceptibility adjustments) Adjust $S_i^{(0)}$ to account for two secondary factors:

- model/knowledge uncertainty: greater uncertainty increases the susceptibility class,
- mitigation/monitoring measures: absence of stabilization/monitoring increases the susceptibility class (per the guideline's logical flow, Figure A.4).

Denote the adjusted susceptibility class by S_i.

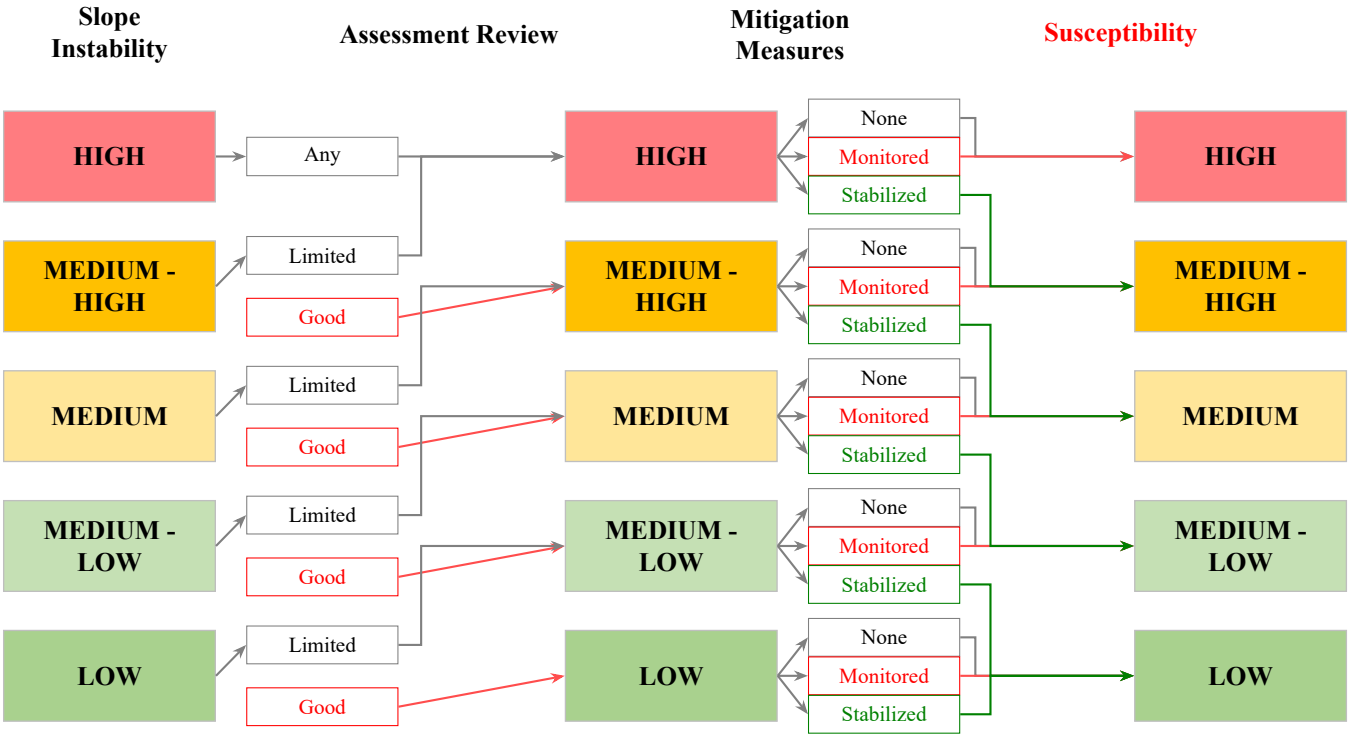


Figure A.4: Logical flow to determine the final landslide susceptibility class (Source: [10, Figura 4.7])

Step 8. (Base vulnerability)

Step 8 assigns a base vulnerability class $V_i^{(0)}$ for bridge i, representing the bridge's inherent susceptibility to damage or loss of functionality if a landslide affects the site. This classification uses readily available

inventory attributes that control the bridge response to ground deformation (lateral displacement, differential settlement, and rotations), namely: (i) structural typology, (ii) static scheme, (iii) span configuration (single- vs multi-span), (iv) representative span length, and (v) material. Foundation type is considered in subsequent adjustment steps (e.g., inadequate lateral resistance).

In relation to previous steps, Steps 3-7 characterize landslide susceptibility at the site (how likely the surrounding slopes are to generate a landslide that could involve the bridge). Step 8 complements this by characterizing the bridge-side response. The resulting $V_i^{(0)}$ is later combined with the susceptibility class and the exposure class to produce the final landslide risk category used for network-level ranking.

To assign $V_i^{(0)}$, classify the bridge according to:

- Material (R.C. = reinforced concrete; P.C. = prestressed concrete),
- Span type (single span vs multi-span),
- Static scheme (statically determinate, e.g., simply supported; statically indeterminate, e.g., continuous/integral),
- Span length class: Medium–Small spans ($L \leq 20$ m) vs High spans ($L > 20$ m).

Then read the corresponding base vulnerability class from Table A.34.

Table A.34: Classification based on static scheme, span length (L), and material

Material	**Span Type**	**Statically Determinate**		**Statically Indeterminate**	
		Med-Small L	**High** L	**Med-Small** L	**High** L
R.C.	Single span	Medium	Medium-High	Low	Medium-Low
	Multi-span	Medium-High	High	Medium-Low	Medium
P.C.	Single span	Medium	Medium-High	-	-
	Multi-span	Medium-High	High	Medium-Low	Medium
Masonry	Single span	-	-	Low	Medium-Low
	Multi-span	-	-	Medium-Low	Medium
Steel	Single span	Medium-Low	Medium-Low	Low	Low
	Multi-span	Medium	Medium	Medium-Low	Medium-Low

Source: [10, Tabella 4.13]

Step 9 (Foundation correction for lateral actions) increases the base vulnerability if the foundations are likely to perform poorly under lateral ground movement (e.g., shallow foundations or inadequate lateral resistance).

Let the ordered vulnerability classes be

$$\mathcal{V} = \{\text{Very Low}, \text{Low}, \text{Medium}, \text{High}, \text{Very High}\}$$

Define the one-step increase operator $\uparrow(\cdot)$ as moving to the next higher class in $\mathcal{V}$ and saturating at the maximum class. Then

$$V_i^{(1)} = \begin{cases} \uparrow\left(V_i^{(0)}\right), & \text{if shallow foundations or inadequate lateral resistance are indicated,} \\ V_i^{(0)}, & \text{otherwise.} \end{cases} \tag{A.134}$$

Step 10 (Interference-extent correction). Step 10 converts the bridge-side vulnerability obtained in Steps 8-9 into a site-specific final vulnerability by accounting for where the landslide action is expected to act.

Steps 8-9 first assign a vulnerability level driven by the bridge itself: Step 8 provides a base class from structural typology, and Step 9 increases that class when foundations are shallow or otherwise expected to have inadequate resistance to lateral ground movements. Step 10 then adjusts this bridge-based vulnerability according to the extent of landslide-bridge interaction at the site:

- (i) total, when the landslide can involve the main structural system (abutments, piers, or foundations);
- (ii) partial, when only a portion of the structural system can be involved; or
- (iii) approach zone, when the effect is limited to the approach embankments/roadway without direct involvement of the main load-carrying structure.

Applying the rule-based mapping in Figure A.5 yields the final vulnerability class V_i used in the subsequent risk-ranking steps.

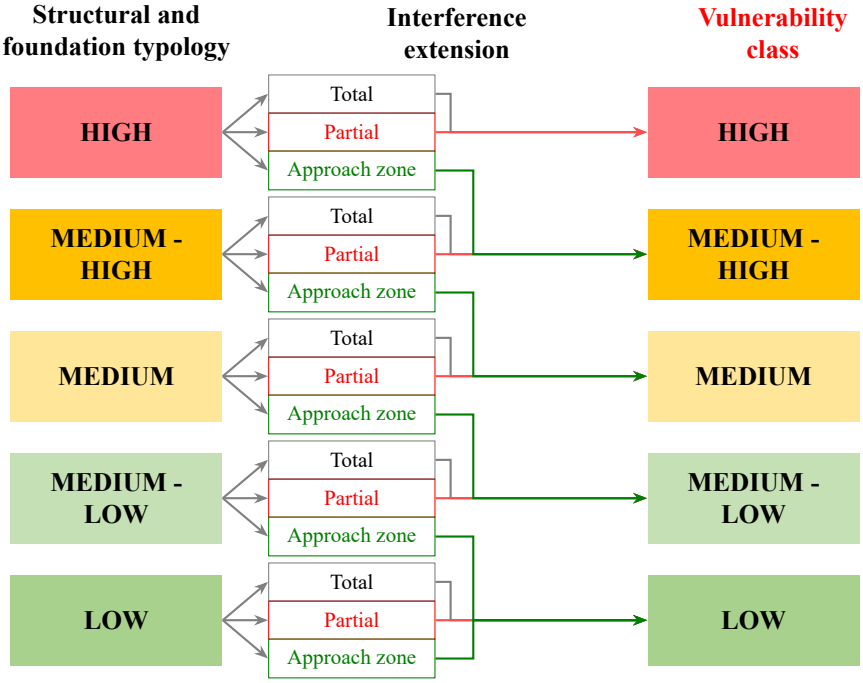


Figure A.5: Logical flow for the determination of the vulnerability class
Source: [10, Figura 4.8]

Step 11 (Exposure class). Step 11 assigns the exposure class E_i, which represents the expected consequences if the bridge becomes unusable due to a landslide. This step measures the bridge's importance to the transportation system and the magnitude of disruption. The preceding susceptibility and vulnerability steps characterize landslide likelihood and potential bridge damage.

In this guideline, the landslide exposure class is obtained using the same rule set adopted for seismic exposure, except that the dangerous-goods parameter is not considered. The procedure is rule-based and combines tables and decision logic as follows:

- **Traffic class.** Classify the bridge traffic level from the average daily traffic (TGM) using the traffic thresholds in Table A.35.

- **Span-length adjustment.** Combine the traffic class with the mean span length to obtain a preliminary exposure class using Table A.36.
- **Alternative-route adjustment.** Adjust the preliminary exposure class based on the availability of suitable alternative routes and detour severity, following the decision logic in Figure A.6 (if information is insufficient, the conservative assumption is to treat alternatives as not available).
- **Crossed-entity adjustment.** Adjust the exposure class based on the type of crossed entity (grouped into consequence classes in Table 4.9), again following Figure A.6.
- **Strategic-importance adjustment.** Apply the final adjustment based on the bridge's strategic role for emergency response and network connectivity using the decision logic in Figure A.7.

The resulting E_i is then combined with the susceptibility and vulnerability classes in the final step to obtain the landslide risk category for network-level ranking.

Table A.35: Average Daily Traffic Level (vehicles/day for the entire roadway)

High	Medium	Low
$\geq 25,000$ vehicles/day	$10,000 <$ vehicles/day $< 25,000$	$\leq 10,000$ vehicles/day

Source: [10, Tabella 4.7.]

Table A.36: Average Daily Traffic (ADT) level and average bridge span length

Average span length	ADT Level		
	High	**Medium**	**Low**
Large span	High	Medium-High	Medium
Medium span	Medium-High	Medium	Medium-Low
Small span	Medium	Medium-Low	Low

Source: [10, Tabella 4.8.]

Step 12 (Final landslide risk category). Step 12 combines the three classification results obtained for bridge i: the landslide susceptibility class S_i, the bridge vulnerability class V_i, and the exposure class E_i. These three inputs are merged using the guideline's predefined lookup matrix (Table A.37) to assign a single final landslide risk category, $\mathcal{M}$, for ranking and prioritization.

A higher category indicates a more critical combination of site susceptibility, bridge vulnerability, and consequence of loss of service, and therefore a higher priority for detailed evaluation, monitoring, or mitigation.

Output and ranking. The procedure outputs, for each bridge i, a final landslide risk category $\mathcal{M}$ (five ordered classes from Very High to Very Low). The network-level prioritization is obtained by sorting bridges primarily by $\mathcal{M}$ (highest category first). If multiple bridges fall in the same category, a secondary ordering is applied to distinguish priorities within that group, for example by using higher susceptibility S_i and/or larger consequence indicators (e.g., higher traffic demand and poorer availability of alternative routes).

A.7.2.2 NCHRP 20-07 Task 378 [2]

This procedure follows the report's two-step structure for landslides: (i) estimate the site-specific extreme-event probability of a landslide occurring near a bridge, and (ii) estimate the probability of service disruption conditional on that landslide event, using polling.

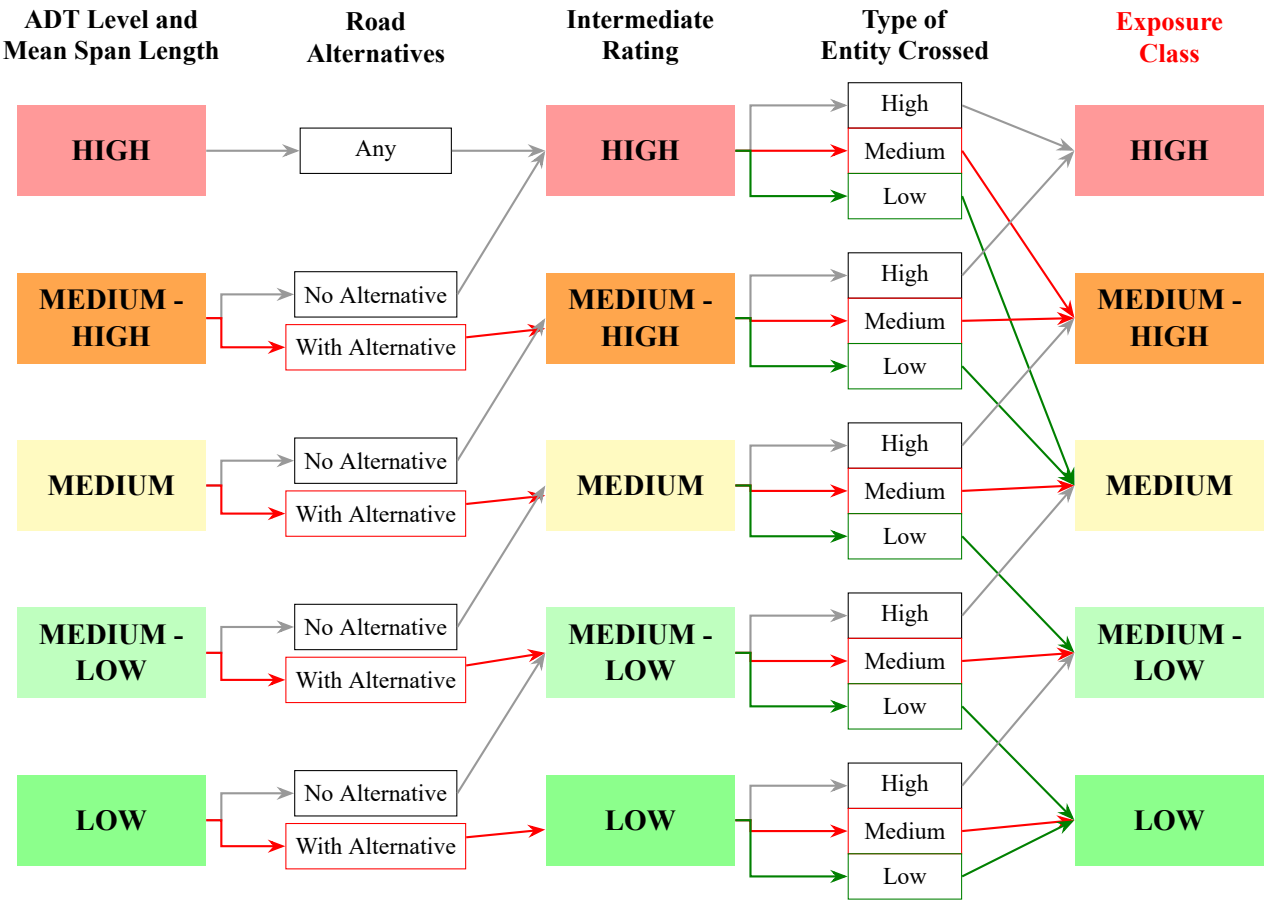


Figure A.6: Logic flow for the determination of the structural and foundation exposure class
Source: [10, Figura 4.3.]

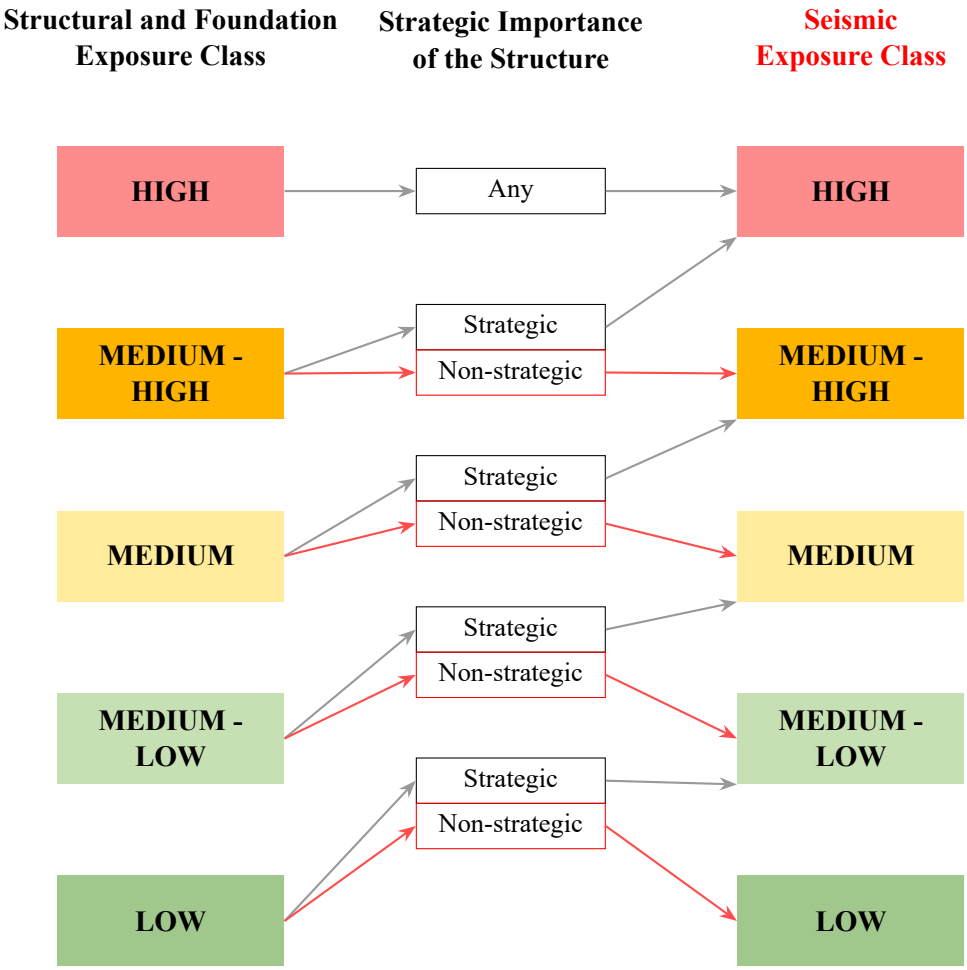


Figure A.7: Logical flow for the determination of the seismic exposure class
Source: [10, Figura 4.6.]

Table A.37: Determination of the landslide attention class as a function of susceptibility, vulnerability, and exposure classes

Susceptibility	Vulnerability	Exposure				
		High	**Med-High**	**Medium**	**Med-Low**	**Low**
High	High		H		MH	
	Med-High		H		MH	M
	Medium	H		MH		M
	Med-Low			MH		M
	Low		MH		M	
Med-High	High	H		MH		M
	Med-High		MH		M	
	Medium		MH		M	
	Med-Low	MH		M		
	Low		MH		M	
Medium	High		MH		M	
	Med-High		MH		M	
	Medium	MH		M		
	Med-Low			M		ML
	Low		M		ML	
Med-Low	High	MH		M		
	Med-High			M		ML
	Medium		M		ML	
	Med-Low		M		ML	
	Low		M		ML	
Low	High			M		ML
	Med-High			M		ML
	Medium	M		ML		
	Med-Low			ML		L
	Low		ML		L	

Key: H = High; MH = Medium-High; M = Medium; ML = Medium-Low; L = Low.
Source: [10, Tabella 4.18]

Step-by-step procedure

Step 1 Define the hazard scenario and the vulnerable bridge population.

Let h denote the hazard scenario (e.g., Landslide-100). Use GIS (or engineering judgment where needed) to define landslide-vulnerable areas (typically roads adjacent to unstable or potentially unstable slopes).

For the analysis area (e.g., a district), define:

$$\mathcal{B} = \{\text{all bridges in the analysis area}\}, \tag{A.135}$$

$$\mathcal{B}_{\text{vuln}} = \{\text{bridges located in landslide-vulnerable areas}\}, \tag{A.136}$$

$$N_{\text{vuln}} = |\mathcal{B}_{\text{vuln}}| \tag{A.137}$$

Step 2 Estimate the extreme-event probability for landslides near bridges.

The report describes two practical GIS-based ways to obtain the annual extreme-event probability for landslides in vulnerable areas.

Approach A: slope-inventory length scaling (bridge-specific). Using a slope inventory, compute:

$$L_{\text{inv}} = \text{total centerline length of roadway affected by unstable slopes (inventory length)}, \tag{A.138}$$

$$L_{\text{inc}} = \text{total centerline length of roadway involved in landslide incidents over the poll window}, \tag{A.139}$$

$$T = \text{number of years covered by the poll window} \tag{A.140}$$

Then the estimated annual landslide probability per unit roadway length is:

$$p_{\text{ft}} = \frac{L_{\text{inc}}}{L_{\text{inv}}\, T} \tag{A.141}$$

For a given bridge $b \in \mathcal{B}_{\text{vuln}}$, let L_b be the total roadway length exposed to the landslide threat (on and under the bridge, as defined by the analyst). The bridge-specific annual extreme-event probability is:

$$p^{\text{ext}}_{b,h} = p_{\text{ft}}\, L_b \tag{A.142}$$

Approach B: hazard-map overlay (population-based). If a jurisdiction provides landslide hazard mapping (e.g., polygons representing zones for a specified return period), overlay the hazard map with bridge locations and compute a scenario-consistent annual probability $p^{\text{ext}}_{b,h}$ for bridges in $\mathcal{B}_{\text{vuln}}$. In the worksheet example, this is entered as line 15 ("Landslide probability (GIS analysis)") and is typically treated as applicable to the vulnerable bridge population for the scenario:

$$p^{\text{ext}}_{b,h} = p^{\text{ext}}_{h} \quad \text{for } b \in \mathcal{B}_{\text{vuln}} \tag{A.143}$$

where

- $p^{\text{ext}}_{b,h}$: annual probability that an extreme landslide event (consistent with scenario h) occurs near bridge b (as represented by the hazard map overlay result).
- p^{ext}_{h}: annual probability of an extreme landslide event for the scenario h, applied uniformly to all bridges in $\mathcal{B}_{\text{vuln}}$ when bridge-specific differentiation is not available.

Step 3 Collect incident recollections through polling to estimate how often vulnerable bridges are affected. Using the "Sheet LD – Landslide" polling worksheet, compile a list of recalled landslide-related bridge incidents involving damage, closure, or delay on or under a bridge during the respondent's defined recollection period. Define:

$$T = \text{years included in the recollection period,} \tag{A.144}$$

$$N_{\text{listed}} = \text{number of listed incidents/bridges affected in the table,} \tag{A.145}$$

$$N_{\text{add}} = \text{additional incidents believed to have occurred,} \tag{A.146}$$

$$N_{\text{aff}} = N_{\text{listed}} + N_{\text{add}} \tag{A.147}$$

Step 4 Convert polling totals to an annual affected-bridge rate within landslide-vulnerable areas.

Assuming each incident corresponds to one bridge being affected (consistent with the worksheet's "Total number of bridges affected" interpretation), the annual rate at which bridges in vulnerable areas are affected is:

$$r_{\text{aff}} = \frac{N_{\text{aff}}}{N_{\text{vuln}}\, T} \tag{A.148}$$

Step 5 Estimate the conditional probability of service disruption given a landslide occurs near a vulnerable bridge. Let p_h^{dis} denote the probability that transportation service is disrupted, conditional on a landslide extreme event occurring near a bridge in the vulnerable population. The worksheet computes this by dividing the observed annual affected-bridge rate by the extreme-event probability:

$$p_h^{\text{dis}} = \frac{r_{\text{aff}}}{p_h^{\text{ext}}} = \frac{N_{\text{aff}}}{N_{\text{vuln}}\, T\, p_h^{\text{ext}}} \tag{A.149}$$

Step 6 Compute the annual likelihood of service disruption for each bridge under the landslide hazard scenario.

For a bridge $b \in \mathcal{B}_{\text{vuln}}$, the annual likelihood of service disruption for scenario h is the product:

$$p_{b,h}^{\text{SD}} = p_{b,h}^{\text{ext}} \cdot p_h^{\text{dis}} \tag{A.150}$$

For bridges outside landslide-vulnerable areas, the report's simplifying assumption is:

$$p_{b,h}^{\text{SD}} = 0 \quad \text{for } b \notin \mathcal{B}_{\text{vuln}} \tag{A.151}$$

Step 7 Special case: use of a uniform extreme-event probability.

When the analysis uses a uniform extreme-event probability in place of bridge-specific GIS, hazard-map, or slope-inventory values, set

$$p_{b,h}^{\text{ext}} = 1.0 \tag{A.152}$$

which has the effect of folding the extreme-event likelihood into the disruption likelihood estimate (i.e., $p_{b,h}^{\text{SD}} = p_h^{\text{dis}}$).

Step 8 Combine likelihood with consequences to compute risk (optional, but consistent with the report's risk definition). For any performance criterion c (e.g., mobility cost, safety cost, agency recovery cost), define the consequence per event at bridge b under scenario h as $C_{b,h,c}$. The report's general risk relationship is:

$$R_{b,h,c} = p_{b,h}^{\text{SD}} \cdot C_{b,h,c} \tag{A.153}$$

A commonly used example consequence model in the report is the mobility (detour) social cost per event:

$$C_{b,h,\text{mob}} = ADT_b \times \frac{D_{b,h}}{24} \times \left(VO \cdot DL_b \cdot VOC + TT \cdot \frac{DL_b}{DS_b} \right) \tag{A.154}$$

where ADT_b is forecast vehicles per day affected, $D_{b,h}$ is disruption duration in hours, DL_b is detour length (miles), DS_b is detour speed (mph), VOC is vehicle operating cost ($/mile), TT is travel time cost ($/hour), and VO is vehicle occupancy (persons/vehicle). Total (multi-criterion) risk can be formed by summing across selected criteria c and/or hazard scenarios h.

A.8 Other Truck Collisions Risk

A.8.1 Introduction

Truck collision risk refers to the probability that a truck or other large vehicle will impact a bridge component (pier, abutment, girder, or deck), multiplied by the potential consequences of such an impact on structural integrity, serviceability, and safety.

Table A.38: U.S. bridge/overpass damage events from 2020 to 2025 caused by vehicle collisions

Date	Bridge / facility	Location	Striking vehicle	Damage / outcome	Ref.
2021-06-23	I-40 WB overpass at Mt. Juliet Rd	Mt. Juliet, TN	Tractor-trailer	Truck struck a support column; column required temporary shoring and multi-week repairs; lane restrictions during repair	[124]
2023-06-11	I-95 overpass (Cottman Ave off-ramp area)	Philadelphia, PA	Gasoline tank-trailer (combination vehicle)	Rollover crash and post-crash fire caused collapse of the northbound lanes; southbound lanes significantly damaged	[125, 126]
2024-03-26	Francis Scott Key Bridge	Baltimore, MD	Containership *Dali*	Vessel struck a support pier after loss of propulsion/steering; bridge collapsed	[127]
2024-05-02	Fairfield Ave overpass over I-95	Norwalk, CT	Gasoline tanker truck (multi-vehicle crash)	Intense fire severely damaged the overpass; bridge demolished and roadway repaired for reopening	[25]
2025-06-14	Old Vaucluse Rd I-20 overpass (S-105)	Aiken County, SC	Fuel tanker truck (collision and fire)	Overpass destroyed in tanker collision/fire; emergency replacement expedited by state order	[128]

Representative Case Studies in Texas

- Truck Collision Causing Bridge Collapse on IH-45 near Corsicana (2002)
  - A truck-tractor-trailer loaded with paper crashed into a bridge pier of the State Highway 14 bridge over IH-45. The impact caused failure of the 30-inch diameter pier, and the bridge subsequently collapsed as a direct result of the collision [129].
- Truck Pier Impact at IH-35 and US-77, Red Oak, Texas (2005)
  - A truck-tractor-trailer struck a central bridge pier on the US-77 overpass carrying US-77 over IH-35. The collision damaged the 30-inch-diameter pier and compromised the structural column. The bridge did not collapse, but the damage required investigation [130].
- Oversized Load Strikes Farm-to-Market 2484 Overpass (Salado, TX) (2015)
  - A commercial vehicle transporting an oversize load on I-35 struck the understructure of the FM 2484 bridge over the interstate. The impact caused two concrete beams of the overpass to collapse onto the highway below, injuring three people and killing one. The National Transportation Safety Board (NTSB) identified inadequate route planning and the absence of required permits as contributing factors [131].
- I-20 Cotton Flat Rd Overpass Struck by Oversized Loads (Midland, TX) (2020)
  - Two separate oversized trucks carrying heavy equipment struck the same Cotton Flat Rd overpass on I-20 within about 30 minutes. The first impact caused enough concern to force closure of westbound lanes while inspections and response were handled, and the driver was cited for multiple violations including overweight and failure to secure the load. A second oversized vehicle also struck the structure with less apparent damage [132].
- Crane Transport Truck Hits Overpass on I-27 (Swisher County) (2025)
  - A truck-tractor-trailer transporting a crane struck an overpass on I-27, causing significant damage to two outside beams, the deck, and concrete on the main lanes. The overpass and main lanes were closed pending replacement of damaged elements and debris removal [133].
- SH 29 Bridge Over I-35 Damaged by 18-Wheeler (Georgetown, TX) (2025)
  - An 18-wheeler struck the SH 29 bridge while traveling under it, damaging structural beams and prompting closure of eastbound lanes. Two beams required replacement, and repair work was planned over several weeks [134].

A.8.2 Methods Reported in the Literature

A.8.2.1 FDOT Truck Collision Assessment Procedure [1]

In the report, truck-collision risk is treated as a man-made, unintentional hazard and assessed as the product of collision likelihood and relative consequences. The likelihood component is computed as an expected annual truck-crash frequency at each bridge location using a regression-style crash model driven by roadway and bridge attributes, including functional class, lanes, structure or roadway length, roadway width, and ADT. Truck fraction and fuel-tanker fraction can refine the traffic inputs. Vulnerability multipliers represent

consequences: element-level vulnerability aggregates statewide 0–5 consequence scores, design vulnerability uses structure type and material from NBI Items 43A and 43B to represent fire and impact resistance, and bridge age represents changes in protective details and risk-based design practice.

Step-by-step procedure **Step 1.** Obtain the bridge inventory (bridge GIS layer / bridge database) as the baseline asset set for risk computation. Key fields should include bridge ID, location, year built (NBI 27), deck area, superstructure type/material (NBI 43A/43B), functional class, lanes, length, roadway width, ADT, and truck percentage.

Step 2. Obtain historical truck-collision or vehicle-impact incident data for a multi-year period (e.g., 10-30 years), preferably including: (i) crash point GIS (or route-milepost), (ii) tabular crash database (vehicle type, severity, damage notes), and (iii) bridge-hit maintenance logs (if maintained separately). Filter to truck-involved impacts relevant to bridge damage, such as: (i) overheight vehicle hits to superstructure, (ii) vehicle impacts to piers/columns/abutments, (iii) truck collisions damaging railings, parapets, or bearings.

Step 3. Link incidents to bridges. If the crash/incident record contains a bridge/structure ID, join directly by ID. Otherwise, perform a spatial overlay (spatial join) between the incident point layer and the bridge GIS layer.

Step 4. Define an association rule to assign incidents to each bridge (choose one consistent rule and document it), for example: (i) a buffer zone around each bridge footprint/centerline (e.g., 0.25-1 mile), or (ii) a narrow corridor around the roadway approaches (e.g., 200-500 ft), or (iii) nearest-bridge assignment with a maximum distance threshold. Assign all qualified truck-collision events to their corresponding bridge.

Step 5. For each bridge, count the number of truck-collision occurrences during the observation period, yielding the historical event frequency near/at that bridge. Denote the count by N_i.

Step 6. Convert the event count to an annual occurrence rate λ_i (Poisson intensity) by dividing by the observation duration T (years):

$$\lambda_i = \frac{N_i}{T} \tag{A.155}$$

Step 7. Assume truck-collision events follow a Poisson process. Convert λ_i to the annual probability of at least one truck-collision occurrence:

$$p_{\text{truck}}(i) = 1 - e^{-\lambda_i} \tag{A.156}$$

Step 8. Compute the truck-collision risk index by multiplying the annual probability by three vulnerability multipliers, and rank bridges in descending order of $\text{Risk}_{\text{truck}}$:

$$\text{Risk}_{\text{truck}}(i) = p_{\text{truck}}(i) \cdot V_{\text{truck}}^{\text{elem}}(i) \cdot V_{\text{truck}}^{\text{age}}(i) \cdot V_{\text{truck}}^{\text{des}}(i) \tag{A.157}$$

The calculation of $V_{\text{truck}}^{\text{elem}}, V_{\text{truck}}^{\text{age}}, V_{\text{truck}}^{\text{des}}$ is shown below.

Element vulnerability $V_{\text{truck}}^{\text{elem}}$. For each bridge, let $\mathcal{E}_i$ denote the set of bridge elements recorded for that bridge that are potentially affected by truck impacts (e.g., girders, bearings, diaphragms, parapets/railings, piers/columns, pier caps, abutments). For each element $e \in \mathcal{E}_i$, assign a truck-collision element score $s_{\text{truck}}(e) \in \{0, 1, 2, 3, 4, 5\}$ using an agency-defined lookup table (Table A.39), where larger values indi-

cate higher susceptibility and/or higher consequence if impacted. A simple bridge-level raw element score is the average:

$$\bar{s}_{\text{truck}}(i) = \frac{1}{|\mathcal{E}_i|} \sum_{e \in \mathcal{E}_i} s_{\text{truck}}(e) \tag{A.158}$$

Rescale the average score to obtain the element vulnerability multiplier:

$$V_{\text{truck}}^{\text{elem}}(i) = \frac{\bar{s}_{\text{truck}}(i) + 1}{3} \tag{A.159}$$

Table A.39: Levels of vulnerability of bridge elements to damage from truck collisions

Table 7.24. Levels of vulnerability of bridge elements to damage from truck collisions

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	3	216	Timber Abutment	2	476	Timber Walls	2
13	Unp Conc Deck/AC Ovl	3	217	Other Mtl Abutment	2	477	Other Walls	2
28	Steel Deck/Open Grid	3	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	2
29	Steel Deck/Conc Grid	3	230	Unpnt Stl Cap	1	480	Mast Arm Found	1
30	Corrug/Orthotpc Deck	3	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	1
31	Timber Deck	3	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	1
32	Timber Deck/AC Ovly	3	234	R/Conc Cap	1	483	Other Mast Arm Vert	1
38	Bare Concrete Slab	3	235	Timber Cap	1	484	Paint Mast Arm Horzn	1
39	Unp Conc Slab/AC Ovl	3	240	Metal Culvert	1	485	Galvan Mast Arm Horz	1
54	Timber Slab	3	241	Concrete Culvert	0	486	Other Mast Arm Horzn	1
55	Timber Slab/AC Ovly	3	242	Timber Culvert	0	487	Sign Member Horiz	1
98	Conc Deck on PC Pane	3	243	Misc Culvert	0	488	Sign Member Vertical	1
99	PS Conc Slab	3	290	Channel	1	489	Sign Foundation	1
101	Unpnt Stl Box Girder	5	298	Pile Jacket Bare	1	495	Uncoat High Mast L.	1
102	Paint Stl Box Girder	5	299	Pile Jacket/Cath Pro	1	496	Painted High Mast L.	1
104	P/S Conc Box Girder	5	300	Strip Seal Exp Joint	3	497	Galvan. High Mast L.	1
105	R/Conc Box Girder	5	301	Pourable Joint Seal	3	498	Other High Mast L.P.	1
106	Unpnt Stl Opn Girder	5	302	Compressn Joint Seal	3	499	H. M. L. P. Found.	1
107	Paint Stl Opn Girder	5	303	Assembly Joint/Seal	3	540	Open Gearing	1
109	P/S Conc Open Girder	5	304	Open Expansion Joint	3	541	Speed Reducers	1
110	R/Conc Open Girder	5	310	Elastomeric Bearing	3	542	Shafts	1
111	Timber Open Girder	5	311	Moveable Bearing	3	543	Shaft Brgs and Coupl	1
112	Unpnt Stl Stringer	5	312	Enclosed Bearing	3	544	Brakes	1
113	Paint Stl Stringer	5	313	Fixed Bearing	3	545	Emergency Drive	1
115	P/S Conc Stringer	5	314	Pot Bearing	3	546	Span Drive Motors	1
116	R/Conc Stringer	5	315	Disk Bearing	3	547	Hydraulic Power Unit	1
117	Timber Stringer	5	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	1
120	U/Stl Thru Truss/Bot	3	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	1
121	P/Stl Thru Truss/Bot	3	330	Metal Rail Uncoated	3	550	Hopkins Frame	1
125	U/Stl Thru Truss/Top	3	331	Conc Bridge Railing	3	560	Locks	1
126	P/Stl Thru Truss/Top	3	332	Timb Bridge Railing	3	561	Live Load Shoes	1
130	Unpnt Stl Deck Truss	3	333	Other Bridge Railing	3	562	Counterweight Suppor	1
131	Paint Stl Deck Truss	3	334	Metal Rail Coated	3	563	Acc Ladd & Plat	1
135	Timber Truss/Arch	3	356	Steel Fatigue SmFlag	0	564	Counterweight	1
140	Unpnt Stl Arch	3	357	Pack Rust Smart Flag	0	565	Trun/Str and Cur Trk	1
141	Paint Stl Arch	3	358	Deck Cracking SmFlag	0	570	Transformers	1
143	P/S Conc Arch	3	359	Soffit Smart Flag	0	571	Submarine Cable	1
144	R/Conc Arch	3	360	Settlement SmFlag	0	572	Conduit & Junc. Box	1
145	Other Arch	3	361	Scour Smart Flag	0	573	PLCs	1
146	Misc Cable Uncoated	3	362	Traf Impact SmFlag	0	574	Control Console	1
147	Misc Cable Coated	3	363	Section Loss SmFlag	0	580	Navigational Lights	1
151	Unpnt Stl Floor Beam	3	369	Sub.Sect Loss SmFlag	0	581	Operator Facilities	1
152	Paint Stl Floor Beam	3	370	Alert Smart Flag	0	582	Lift Bridge Spec. Eq	1
154	P/S Conc Floor Beam	3	386	Fender/Dolphin Uncoa	0	583	Swing Bridge Spec. E	1
155	R/Conc Floor Beam	5	387	P/S Fender/Dolphin	0	590	Resistance Barriers	1
156	Timber Floor Beam	5	388	R/Conc Fender/Dolphi	0	591	Warning Gates	1
160	Unpnt Stl Pin/Hanger	3	389	Timber Fender/Dolphi	0	592	Traffic Signals	1
161	Paint Stl Pin/Hanger	3	390	Other Fender/Dolphin	0			
201	Unpnt Stl Column	3	393	Blkhd Sewl Metal Unc	0			
202	Paint Stl Column	3	394	R/Conc Abut Slope Pr	2			
204	P/S Conc Column	3	395	Timber Abut Slope Pr	2			
205	R/Conc Column	3	396	Other Abut Slope Pro	2			
206	Timber Column	3	397	Drain. Syst Metal	1			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	1			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	1			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	1			
215	R/Conc Abutment	2	475	R/Conc Walls	2			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Source: [1, Table 7.24]

Age vulnerability $V^{\text{age}}_{\text{truck}}$. Let Y_i be the year the bridge was built, or the year of the last major reconstruction when selected by the analysis. Older bridges may have lower redundancy, older design details, or earlier roadside safety features. Model the age vulnerability multiplier as a linear function of bridge age using the template form and calibrated coefficients:

$$V^{\text{age}}_{\text{truck}}(i) = 1 + \frac{Y_0 - Y_i}{K} \tag{A.160}$$

where Y_0 is a reference year (e.g., 1990) and K is a scaling parameter (e.g., 35) controlling how strongly age increases vulnerability.

Design vulnerability $V^{\text{des}}_{\text{truck}}$. Identify each bridge's key impact-relevant design attributes, for example: (i) superstructure type/material (steel girder, prestressed concrete girder, slab, truss, etc.), (ii) pier type/configuration (single column, multi-column bent, wall pier), (iii) vertical clearance and overheight exposure, (iv) presence/absence of protective systems (crash walls, pier protection, guide rail). Assign a discrete design score $s^{\text{des}}_{\text{truck}}(i) \in \{0, 1, 2, 3, 4, 5\}$ using a lookup table. [5]

Once $s^{\text{des}}_{\text{truck}}(i)$ is obtained, then rescale:

$$V^{\text{des}}_{\text{truck}}(i) = \frac{s^{\text{des}}_{\text{truck}}(i) + 1}{3} \tag{A.161}$$

Agency cost. First, compute the replacement (or repair proxy) cost for bridge i:

$$R_i = A_i c \tag{A.162}$$

where A_i is deck area and c is the unit replacement cost per area. Define the agency weight as the normalized share over the bridge inventory set $\mathcal{B}$:

$$w_i = \frac{R_i \cdot \text{Risk}_{\text{truck}}(i)}{\sum_{j \in \mathcal{B}} R_j \cdot \text{Risk}_{\text{truck}}(j)} \tag{A.163}$$

Given the statewide (or network-level) average annual agency loss from truck collisions L_{truck} (control total), allocate it using w_i:

$$\text{AgencyCost}_{\text{truck}}(i) = L_{\text{truck}} w_i \tag{A.164}$$

User cost. For bridge i, define the daily detour cost:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}} \frac{\ell_i}{v_i} \right) \tag{A.165}$$

where ADT_i is average daily traffic, ℓ_i is detour length, v_i is detour speed, c_{op} is unit operating cost per mile, and c_{time} is value of time per hour. (If truck impacts are the focus, an optional refinement is to split ADT_i into passenger and truck components with different operating costs and values of time.)

[5] The discussion derives this variable from the Fire Risk values in Table A.14. NBI Items 43A and 43B determine the corresponding value.

Define the user weight as the normalized share over the bridge inventory:

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{truck}}(i)}{\sum_{j \in \mathcal{B}} U_j \cdot \text{Risk}_{\text{truck}}(j)} \tag{A.166}$$

Given the statewide annual control total for truck-collision user losses L^U_{truck}, allocate it using u_i:

$$\text{UserCost}_{\text{truck}}(i) = L^U_{\text{truck}} u_i \tag{A.167}$$

Total cost. After computing $\text{AgencyCost}_{\text{truck}}(i)$ and $\text{UserCost}_{\text{truck}}(i)$ for each bridge i, form the total annual expected impact (USD/year) by summing the two components:

$$\text{TotalCost}_{\text{truck}}(i) = \text{AgencyCost}_{\text{truck}}(i) + \text{UserCost}_{\text{truck}}(i) \tag{A.168}$$

A.8.2.2 NCHRP 892 Guidelines for Shielding Bridge Piers [11]

The source report defines two network-scale procedures: (i) a heavy-vehicle pier-collision collapse-risk procedure (LRFD pier protection) and (ii) a passenger-vehicle occupant-risk procedure (RDG occupant protection). Both procedures are applied at the pier-system level. A bridge-level screening score can be calculated as the sum or maximum of the pier-system results.

Notation Index $i = 1, \ldots, m$ denotes a direction of travel that can generate an encroachment toward the pier system. Let n be the number of columns in the pier system.

Heavy vehicles (bridge-collapse risk):

- HVE_i: base annual frequency of heavy-vehicle encroachments in direction i (veh/yr), from Table A.40.
- N_i: site-specific encroachment adjustment factor in direction i, computed from Table A.41.
- O_i: offset from the relevant lane edge line to the face of the critical pier component (ft).
- D_i: critical pier component size (ft), defined as column diameter for circular columns or the largest dimension for rectangular columns.
- $P(C \mid HVE_i)$: probability that a heavy-vehicle encroachment results in a collision with the (unshielded) critical pier component, from Table A.42.
- R_{CPC}: nominal lateral resistance of the critical pier component (kips).
- Q_{CT}: worst-case collision force (as defined in the report).
- $P(Q_{\text{CT}} > R_{\text{CPC}} \mid C)$: probability that the worst-case collision force exceeds capacity given a collision, from Table A.43 (selected by roadway/traffic class, posted speed, and R_{CPC};).
- AF_{BC}: annual frequency of bridge collapse induced by heavy-vehicle collisions (1/yr).

Passenger vehicles (occupant protection risk):

- PVE_i: base annual frequency of passenger-vehicle encroachments in direction i (veh/yr), from Table A.44.

Table A.40: Base annual heavy-vehicle encroachments in direction

Two-Way AADT	Undivided Highways							
	PT							
veh/day	5	10	15	20	25	30	35	≥40
0	0	0	0	0	0	0	0	0
1,000	0.0009	0.0017	0.0019	0.0020	0.0021	0.0022	0.0022	0.0023
2,000	0.0014	0.0028	0.0031	0.0033	0.0034	0.0035	0.0036	0.0037
3,000	0.0017	0.0034	0.0038	0.0040	0.0042	0.0043	0.0044	0.0045
4,000	0.0019	0.0037	0.0041	0.0043	0.0045	0.0046	0.0048	0.0049
5,000–41,000	0.0019	0.0038	0.0042	0.0044	0.0046	0.0047	0.0048	0.0049
42,000	0.0020	0.0039	0.0043	0.0045	0.0047	0.0049	0.0050	0.0051
43,000	0.0020	0.0040	0.0044	0.0047	0.0048	0.0050	0.0051	0.0052
44,000	0.0020	0.0041	0.0045	0.0048	0.0049	0.0051	0.0052	0.0054
45,000	0.0021	0.0042	0.0046	0.0049	0.0051	0.0052	0.0054	0.0055
≥46,000	0.0021	0.0043	0.0047	0.0050	0.0052	0.0053	0.0055	0.0056
Two-Way AADT	**Divided Highways**							
	PT							
veh/day	5	10	15	20	25	30	35	≥40
1,000	0.0006	0.0006	0.0006	0.0006	0.0007	0.0007	0.0007	0.0007
5,000	0.0026	0.0026	0.0027	0.0027	0.0028	0.0028	0.0028	0.0028
10,000	0.0042	0.0043	0.0044	0.0045	0.0045	0.0045	0.0046	0.0046
15,000	0.0051	0.0053	0.0054	0.0054	0.0055	0.0055	0.0056	0.0056
20,000	0.0055	0.0057	0.0058	0.0059	0.0060	0.0060	0.0060	0.0061
24,000–47,000	0.0056	0.0058	0.0059	0.0060	0.0061	0.0061	0.0062	0.0062
50,000	0.0060	0.0062	0.0064	0.0065	0.0065	0.0066	0.0066	0.0067
55,000	0.0066	0.0069	0.0070	0.0071	0.0072	0.0072	0.0073	0.0073
60,000	0.0072	0.0075	0.0076	0.0077	0.0078	0.0079	0.0079	0.0080
65,000	0.0078	0.0081	0.0083	0.0084	0.0085	0.0085	0.0086	0.0087
70,000	0.0084	0.0087	0.0089	0.0090	0.0091	0.0092	0.0093	0.0093
75,000	0.0090	0.0094	0.0095	0.0097	0.0098	0.0099	0.0099	0.0100
80,000	0.0096	0.0100	0.0102	0.0103	0.0104	0.0105	0.0106	0.0107
85,000	0.0102	0.0106	0.0108	0.0110	0.0111	0.0112	0.0113	0.0113
≥ 90,000	0.0108	0.0112	0.0115	0.0116	0.0117	0.0118	0.0119	0.0120

Note: Encroachment data are not available for one-way roadways. One-way roadways should be evaluated using the encroachment model for divided highways where the one-way AADT value should be multiplied by 2 and used to determine HVE$_i$ for use in the calculations.

Source: [11, Table 13]

Table A.41: Site-specific adjustment factor (N_i)

Major Accesses‡			Lane Width			Horizontal Curve Radius†	
Number of access points within 300ft upstream of the pier system	Undivided	Divided and one-way	Avg. lane width in ft	Undivided	Divided and one-way	Horizontal curve radius at centerline in ft	All highway types
0	1.0	1.0	≤9	1.50	1.25	AR > 10,000	1.00
1	1.5	2.0	10	1.30	1.15	10,000 ≥ AR > 432	Exp(474.4/AR)
2≤	2.2	4.0	11	1.05	1.03	432 ≥ AR > 0	3.00
			≥12	1.00	1.00	TR > 10,000	1.00
						10,000 ≥ TR > 432	Exp(173.6/TR)
						432 ≥ TR > 0	1.50
f_{ACC} =			f_{LW} =			f_{HC} =	
Lanes in One Direction			**Posted Speed Limit¶**			**Grade Approaching the Pier System††**	
No. of through lanes in one direction	Undivided	Divided and one-way	Posted speed limit	Undivided	Divided and one-way	Percent grade	All highway types
1	1.00	1.00	<65	1.42	1.18	-6 ≥ G	2.00
2	0.76	1.00	≥65	1.00	1.00	-6 < G < -2	0.5 – G/4
≥3	0.76	0.91				-2 ≤ G	1.00
f_{LN} =			f_{PSL} =			f_G =	
$N_i = f_{ACC} \cdot f_{LN} \cdot f_{LW} \cdot f_G \cdot f_{HC} \cdot f_{PSL}$ =							

Notes: ACC = major accesses, LW = lane width, HC = horizontal curve radius, LN = lanes in one direction, and G = grade approaching the pier system.
‡Major accesses include ramps and intersections. Commercial and residential driveways should not be included as access points unless they are signalized or stop-sign controlled.
†The horizontal curve radius may either curve away (AR) from the pier system under consideration or toward it (TR). When the driver is turning the wheel of the vehicle away from the pier, the AR adjustments should be used. When the driver is turning the wheel of the vehicle toward the pier, the TR adjustments should be used. This adjustment must be considered for each direction of travel (i) where an encroaching vehicle could approach the pier system.
††The grade (G) approaching the pier system must be considered for each direction of travel (i). Positive values indicate an uphill grade, and negative values indicate a downhill grade.
¶ For roads with unposted speed limits, use the adjustment for <65 mph.

Source: [11, Table 15]

Table A.42: Probability of a heavy-vehicle collision given a heavy-vehicle encroachment as a function of pier column diameter or wall thickness and offset from the direction of travel [P(C|HVEi)].

Offset ‡ (ft)	**Pier Column Size** (ft) †				
	1	2	3	4	6
2	0.1763	0.1868	0.1978	0.2093	0.2337
4	0.1650	0.1750	0.1855	0.1964	0.2198
6	0.1543	0.1638	0.1738	0.1842	0.2064
8	0.1442	0.1532	0.1626	0.1725	0.1937
10	0.1347	0.1432	0.1521	0.1614	0.1816
15	0.1131	0.1204	0.1282	0.1363	0.1539
20	0.0946	0.1009	0.1075	0.1145	0.1297
25	0.0789	0.0842	0.0899	0.0958	0.1088
30	0.0656	0.0701	0.0749	0.0799	0.0910
35	0.0544	0.0582	0.0622	0.0665	0.0758
40	0.0450	0.0482	0.0515	0.0551	0.0630

$$P(C|\mathrm{HVE}_i) = \frac{e^{-0.0398\,P_i + 0.0709\,D_i - 1.5331}}{1 + e^{-0.0398\,P_i + 0.0709\,D_i - 1.5331}}$$

‡ P_i = Offset to critical pier component in direction (i) in ft,where the distance is from the face of the critical pier component to the closest edge of travel lane (i).

† D_i = Size of the critical component of the pier in direction (i), where size is either the diameter of the critical circular column or the smallest cross-sectional dimension of a rectangular column.

Source: [11, Table 19]

Table A.43: Probability of impact force (QCT) exceeding critical pier component nominal lateral resistance (RCPC).

	Rural Interstates and Primaries							**Rural Collectors**						
	Posted Speed Limit (mi/hr)							Posted Speed Limit (mi/hr)						
R_{CPC}	≤45	50	55	60	65	70	≥75	≤45	50	55	60	65	70	≥75
100	0.9999	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
150	0.9939	0.9989	0.9999	1.0000	1.0000	1.0000	1.0000	0.9817	0.9969	0.9993	1.0000	1.0000	1.0000	1.0000
200	0.9063	0.9629	0.9890	0.9966	0.9992	0.9996	0.9999	0.6980	0.8826	0.9609	0.9892	0.9960	0.9994	0.9998
250	0.8058	0.8422	0.9049	0.9565	0.9824	0.9935	0.9974	0.3710	0.5055	0.7018	0.8602	0.9431	0.9792	0.9930
300	0.7931	0.7928	0.8125	0.8566	0.9116	0.9533	0.9771	0.3322	0.3429	0.4023	0.5462	0.7134	0.8523	0.9283
350	0.7892	0.7884	0.7907	0.7996	0.8279	0.8684	0.9142	0.3302	0.3315	0.3350	0.3657	0.4455	0.5800	0.7291
400	0.7584	0.7832	0.7886	0.7902	0.7978	0.8079	0.8370	0.3179	0.3294	0.3300	0.3374	0.3464	0.3897	0.4873
450	0.6440	0.7550	0.7820	0.7887	0.7931	0.7914	0.7990	0.2720	0.3177	0.3280	0.3358	0.3327	0.3357	0.3622
500	0.4232	0.6620	0.7552	0.7817	0.7912	0.7894	0.7901	0.1797	0.2770	0.3163	0.3328	0.3313	0.3296	0.3360
550	0.1964	0.4754	0.6731	0.7570	0.7843	0.7879	0.7888	0.0817	0.1993	0.2837	0.3213	0.3290	0.3285	0.3323
600	0.0597	0.2628	0.5216	0.6903	0.7602	0.7810	0.7870	0.0254	0.1086	0.2163	0.2895	0.3183	0.3261	0.3313
650	0.0125	0.1054	0.3292	0.5582	0.6999	0.7584	0.7790	0.0056	0.0432	0.1397	0.2356	0.2942	0.3174	0.3287
700	0.0016	0.0312	0.1614	0.3816	0.5883	0.7076	0.7586	0.0008	0.0130	0.0657	0.1645	0.2463	0.2956	0.3193
750	0.0002	0.0067	0.0584	0.2132	0.4338	0.6144	0.7095	0.0000	0.0028	0.0253	0.0916	0.1833	0.2550	0.2998
800	0.0000	0.0008	0.0177	0.0958	0.2706	0.4781	0.6263	0.0000	0.0005	0.0070	0.0429	0.1129	0.1975	0.2666
850	0.0000	0.0001	0.0048	0.0361	0.1390	0.3246	0.5072	0.0000	0.0001	0.0016	0.0158	0.0610	0.1343	0.2167
900	0.0000	0.0000	0.0007	0.0098	0.0594	0.1934	0.3692	0.0000	0.0000	0.0003	0.0048	0.0269	0.0796	0.1571
950	0.0000	0.0000	0.0001	0.0024	0.0224	0.0988	0.2362	0.0000	0.0000	0.0001	0.0012	0.0107	0.0400	0.0998
1,000	0.0000	0.0000	0.0000	0.0006	0.0065	0.0431	0.1363	0.0000	0.0000	0.0001	0.0002	0.0033	0.0165	0.0559
1,050	0.0000	0.0000	0.0000	0.0000	0.0018	0.0155	0.0670	0.0000	0.0000	0.0000	0.0000	0.0010	0.0063	0.0260
1,100	0.0000	0.0000	0.0000	0.0000	0.0006	0.0054	0.0285	0.0000	0.0000	0.0000	0.0000	0.0002	0.0018	0.0117
1,150	0.0000	0.0000	0.0000	0.0000	0.0001	0.0015	0.0102	0.0000	0.0000	0.0000	0.0000	0.0000	0.0005	0.0042
1,200	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0034	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0014
1,250	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0011	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0004
1,300	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001

	Urban Interstates and Primaries							**Urban Collectors**						
	Posted Speed Limit (mi/hr)							Posted Speed Limit (mi/hr)						
R_{CPC}	≤45	50	55	60	65	70	≥75	≤45	50	55	60	65	70	≥75
100	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
150	0.9924	0.9986	0.9996	0.9999	1.0000	1.0000	1.0000	0.9798	0.9961	0.9996	0.9997	0.9999	1.0000	1.0000
200	0.8599	0.9419	0.9813	0.9947	0.9987	0.9995	0.9998	0.6462	0.8638	0.9551	0.9870	0.9969	0.9990	0.9996
250	0.7093	0.7597	0.8573	0.9322	0.9743	0.9903	0.9966	0.2676	0.4239	0.6550	0.8368	0.9350	0.9763	0.9908
300	0.6915	0.6837	0.7196	0.7815	0.8863	0.9264	0.9673	0.2228	0.2396	0.3082	0.4745	0.6701	0.8248	0.9155
350	0.6876	0.6769	0.6858	0.6962	0.7394	0.7954	0.8723	0.2211	0.2260	0.2274	0.2599	0.3610	0.5123	0.6816
400	0.6622	0.6728	0.6832	0.6832	0.6890	0.7054	0.7587	0.2129	0.2245	0.2228	0.2237	0.2410	0.2901	0.3987
450	0.5611	0.6504	0.6791	0.6816	0.6826	0.6795	0.6997	0.1798	0.2166	0.2210	0.2216	0.2258	0.2295	0.2579
500	0.3724	0.5678	0.6562	0.6764	0.6812	0.6764	0.6869	0.1187	0.1887	0.2139	0.2199	0.2248	0.2223	0.2245
550	0.1718	0.4055	0.5845	0.6542	0.6758	0.6751	0.6850	0.0552	0.1377	0.1914	0.2128	0.2227	0.2211	0.2208
600	0.0513	0.2231	0.4522	0.5932	0.6544	0.6681	0.6833	0.0161	0.0742	0.1486	0.1932	0.2151	0.2191	0.2200
650	0.0110	0.0886	0.2836	0.4795	0.6024	0.6485	0.6775	0.0029	0.0284	0.0937	0.1574	0.1975	0.2134	0.2180
700	0.0010	0.0252	0.1410	0.3302	0.5068	0.6042	0.6589	0.0003	0.0079	0.0461	0.1071	0.1666	0.1998	0.2118
750	0.0002	0.0051	0.0529	0.1847	0.3724	0.5194	0.6170	0.0000	0.0019	0.0172	0.0592	0.1246	0.1741	0.1992
800	0.0000	0.0005	0.0155	0.0851	0.2344	0.4050	0.5437	0.0000	0.0003	0.0054	0.0266	0.0758	0.1356	0.1761
850	0.0000	0.0001	0.0038	0.0315	0.1200	0.2770	0.4387	0.0000	0.0000	0.0012	0.0100	0.0417	0.0924	0.1435
900	0.0000	0.0000	0.0008	0.0092	0.0529	0.1636	0.3200	0.0000	0.0000	0.0003	0.0026	0.0182	0.0554	0.1055
950	0.0000	0.0000	0.0001	0.0022	0.0184	0.0836	0.2076	0.0000	0.0000	0.0000	0.0005	0.0067	0.0279	0.0698
1,000	0.0000	0.0000	0.0000	0.0003	0.0055	0.0356	0.1186	0.0000	0.0000	0.0000	0.0001	0.0018	0.0116	0.0411
1,050	0.0000	0.0000	0.0000	0.0000	0.0016	0.0138	0.0584	0.0000	0.0000	0.0000	0.0000	0.0006	0.0041	0.0210
1,100	0.0000	0.0000	0.0000	0.0000	0.0004	0.0042	0.0252	0.0000	0.0000	0.0000	0.0000	0.0001	0.0015	0.0088
1,150	0.0000	0.0000	0.0000	0.0000	0.0000	0.0010	0.0104	0.0000	0.0000	0.0000	0.0000	0.0000	0.0004	0.0038
1,200	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0031	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0013
1,250	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0008	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0004
1,300	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001

Source: [11, Table 7]

- $P(C \mid PVE_i)$: probability that a passenger-vehicle encroachment results in a collision with the (unshielded) pier component, from Table A.45.
- $P(KA_{\text{CUSP}} \mid C)$: probability of severe or fatal injury (KA) given a collision with an unshielded pier component, from Table 26 (or the report's speed-scaling model).
- AF_{CUSP}^{KA}: annual expected frequency of severe/fatal crashes with the unshielded pier system (1/yr).

Table A.44: Base annual passenger vehicle encroachments in direction (i): PVE

Two-Way AADT	Undivided Highways							
	PT							
veh/day	5	10	15	20	25	30	35	40
1,000	0.0165	0.0157	0.0148	0.0139	0.0130	0.0122	0.0113	0.0104
2,000	0.0268	0.0254	0.0240	0.0226	0.0212	0.0198	0.0183	0.0169
3,000	0.0326	0.0309	0.0292	0.0275	0.0258	0.0240	0.0223	0.0206
4,000	0.0353	0.0334	0.0316	0.0297	0.0279	0.0260	0.0241	0.0223
5,000–41,000	0.0358	0.0339	0.0320	0.0301	0.0282	0.0264	0.0245	0.0226
42,000	0.0371	0.0351	0.0332	0.0312	0.0293	0.0273	0.0254	0.0234
43,000	0.0380	0.0360	0.0340	0.0320	0.0300	0.0280	0.0260	0.0240
44,000	0.0389	0.0368	0.0348	0.0327	0.0307	0.0286	0.0266	0.0245
45,000	0.0397	0.0377	0.0356	0.0335	0.0314	0.0293	0.0272	0.0251
≥46,000	0.0406	0.0385	0.0364	0.0342	0.0321	0.0299	0.0278	0.0257
Two-Way AADT	**Divided Highways**							
	PT							
veh/day	5	10	15	20	25	30	35	40
1,000	0.0114	0.0108	0.0102	0.0096	0.0090	0.0084	0.0078	0.0072
5,000	0.0485	0.0459	0.0434	0.0408	0.0383	0.0357	0.0332	0.0306
10,000	0.0789	0.0747	0.0706	0.0664	0.0623	0.0581	0.0540	0.0498
15,000	0.0962	0.0912	0.0861	0.0810	0.0760	0.0709	0.0658	0.0608
20,000	0.1044	0.0989	0.0934	0.0879	0.0824	0.0769	0.0714	0.0659
24,000–47,000	0.1062	0.1006	0.0950	0.0894	0.0838	0.0782	0.0727	0.0671
50,000	0.1143	0.1082	0.1022	0.0962	0.0902	0.0842	0.0782	0.0722
55,000	0.1257	0.1191	0.1125	0.1058	0.0992	0.0926	0.0860	0.0794
60,000	0.1371	0.1299	0.1227	0.1155	0.1082	0.1010	0.0938	0.0866
65,000	0.1485	0.1407	0.1329	0.1251	0.1173	0.1094	0.1016	0.0938
70,000	0.1600	0.1515	0.1431	0.1347	0.1263	0.1179	0.1094	0.1010
75,000	0.1714	0.1624	0.1533	0.1443	0.1353	0.1263	0.1173	0.1082
80,000	0.1828	0.1732	0.1636	0.1540	0.1443	0.1347	0.1251	0.1155
85,000	0.1942	0.1840	0.1738	0.1636	0.1533	0.1431	0.1329	0.1227
≥ 90,000	0.2057	0.1948	0.1840	0.1732	0.1624	0.1515	0.1407	0.1299

†Encroachment data are not available for one-way roadways.Traditionally, one-way roadways have been evaluated using the encroachment model for divided highways.The one-way AADT value should be multiplied by 2 and used to determine PV $\text{ENCR}_{\text{DIV BASE}}$ for use in the remaining calculations.

Source: [11, Table 24]

A. Heavy-vehicle pier-collision collapse-risk procedure (LRFD pier protection) Step 1. Assemble desk-based inputs for each pier system.

- Roadway/traffic by direction: highway type, functional class, two-way AADT, truck percentage PT, posted speed limit PSL, lane width, number of lanes in the approach direction, grade, horizontal curve information, and number of major access points upstream (within the report's defined influence distance for Table A.41).
- Pier-system geometry: number of columns n, the face offset O_i for the leading/critical component in each direction, and the component size D_i.
- Structural capacity: nominal lateral resistance R_{CPC} of the critical pier component.

Step 2. Define directions $i = 1, \ldots, m$ that can generate an encroachment toward the pier system. For each direction, identify the leading/critical pier component to be evaluated, and record (O_i, D_i).

Step 3. Compute the site-specific adjustment factor N_i for each direction. Using Table A.41, obtain the

Table A.45: Probability of a collision given a passenger vehicle encroachment: P(C|PVEi).

Offset (ft)[‡]	Pier Column Size (ft)[†]				
	1	2	3	4	6
2	0.1125	0.1242	0.1369	0.1507	0.1818
4	0.1066	0.1178	0.1300	0.1432	0.1730
6	0.1011	0.1117	0.1233	0.1360	0.1646
8	0.0957	0.1059	0.1170	0.1291	0.1565
10	0.0907	0.1004	0.1109	0.1225	0.1487
15	0.0790	0.0876	0.0970	0.1073	0.1307
20	0.0688	0.0763	0.0846	0.0937	0.1146
25	0.0598	0.0664	0.0737	0.0817	0.1002
30	0.0519	0.0577	0.0641	0.0712	0.0875
35	0.0450	0.0501	0.0557	0.0619	0.0762
40	0.0390	0.0434	0.0483	0.0537	0.0663

$$P(C|\text{PVE}_i) = \frac{e^{-0.0300\,P_i+0.1122\,D_i-2.1177}}{1+\,e^{-0.0300\,P_i+0.1122\,D_i-2\,1177}}$$

[‡]P_i = Offset to critical pier component in direction (i) in ft where the distance is from the face of the critical pier component to the closest edge of travel lane (i).

[†]D_i = Size of the critical component of the pier in direction (i) where size is either the diameter of the critical circular column or the smallest cross-sectional dimension of a rectangular column.

Source: [11, Table 25]

encroachment adjustment multipliers for direction i:

$$f_{\text{ACC},i},\ f_{\text{LW},i},\ f_{\text{HC},i},\ f_{\text{LN},i},\ f_{\text{PSL},i},\ f_{\text{G},i}$$

Then compute

$$N_i = f_{\text{ACC},i}\ f_{\text{LW},i}\ f_{\text{HC},i}\ f_{\text{LN},i}\ f_{\text{PSL},i}\ f_{\text{G},i} \tag{A.169}$$

Step 4. Obtain the base annual frequency of heavy-vehicle encroachments HVE_i. For each direction i, use Table A.40 with (highway type, PT, AADT) to look up HVE_i (veh/yr).

Step 5. Compute the conditional probability of collision given a heavy-vehicle encroachment, $P(C \mid HVE_i)$. Two equivalent implementations are provided in the report:

- Table-based: Use Table A.42 with (O_i, D_i) to obtain $P(C \mid HVE_i)$ (interpolate if needed).
- Model-based: Use the fitted logit model

$$P(C \mid HVE_i) = \frac{\exp(\eta_i)}{1+\exp(\eta_i)}, \tag{A.170}$$

$$\eta_i = \beta_0 + \beta_O\, O_i + \beta_D\, D_i \tag{A.171}$$

where $(\beta_0, \beta_O, \beta_D)$ are taken from Table A.46 for the heavy-vehicle model.

Table A.46: Heavy-vehicle model coefficients on crash proportion.

	Estimate	Std. Error	z value	Pr(>|z|)	2.50%	97.50%
(Intercept)	-1.5331	0.02	-70.97	<2E-16	-1.5755	-1.4908
offset	-0.0398	0.00	-43.96	<2E-16	-0.0416	-0.0381
diameter	0.0709	0.01	10.77	<2E-16	0.0580	0.0839

Source: [11, Table 18]

Step 6. Compute $P(Q_{\text{CT}} > R_{\text{CPC}} \mid C)$ for each direction. First, determine R_{CPC} for the critical compo-

nent (per the report's structural-capacity guidance). Then use Table 7 to obtain the conditional exceedance probability based on:

$$\text{roadway/traffic class}, \quad PSL_i, \quad R_{\text{CPC}}$$

Step 7. Compute the annual frequency of bridge collapse for the pier system.

$$AF_{\text{BC}} = \sum_{i=1}^{m} [N_i \; HVE_i \; P(C \mid HVE_i) \; P(Q_{\text{CT}} > R_{\text{CPC}} \mid C)_i] \tag{A.172}$$

Step 8. Screening decision and ranking metric. Use AF_{BC} as the pier-system collapse-risk score for network ranking. The report's proposed acceptance checks compare AF_{BC} to a threshold that depends on whether the bridge is classified as typical or critical.

A.9 Overloads Risk

A.9.1 Introduction

Overload risk refers to the expected impact on a bridge arising from trucks whose weights exceed the bridge's allowable capacity, including both potential structural damage and traffic disruption from load posting and detours.

Representative Case Studies in Texas

- Wooden Neighborhood Bridge Collapse (Willis, Montgomery County, TX) (2023)
  - A concrete truck with an estimated weight of 20–40 tons crossed a wooden neighborhood bridge posted for a 4-ton limit. The truck broke through the deck, and the structure collapsed into the water. The bridge required replacement [143].
- Recurring Bridge Strikes - Houston Avenue Bridge (I-10, Houston, TX) (2025)
  - The Houston Avenue overpass bridge above I-10 in Houston has experienced dozens of vehicle strikes, many involving trucks that are overloads or overheight beyond the safe design clearance [144].

A.9.2 Methods Reported in the Literature

A.9.2.1 FDOT Assessment Procedure [1]

The report treats truck overloads as a man-made extreme event whose risk can be assessed at the network level using (1) an exposure-driven hazard proxy and (2) a bridge-specific vulnerability (consequence) model.

(1) Hazard likelihood: Instead of modeling illegal loads directly, the overload hazard is assumed proportional to the fraction of trucks that would be detoured because the bridge has insufficient operating rating. This detour fraction is computed from a Florida truck-weight model as a piecewise function of the bridge operating rating (NBI Item 66), with different relationships for Interstate bridges versus other functional classes. The detour fraction is then combined with traffic volume (ADT) and truck percentage to estimate the number of

Table A.47: Selected U.S. bridge damage/collapse incidents

Date	State (Area)	Bridge / Facility	Vehicle	Evidence of overload	Outcome / damage
2023-09-14	TX (Montgomery Co., Walnut Cove)	Neighborhood wooden bridge at Maywood Dr & Sassafras	Loaded concrete truck	Bridge posted **4-ton** limit; loaded concrete truck ∼**20 tons**	Bridge crushed/sank; hazmat precautions for leakage into waterway [135]
2024-08-23	ME (Gorham–Windham)	Babb's Bridge (covered bridge over Presumpscot River)	Ford F-750 dump truck (gravel)	Truck ≈**36,000 lb** vs posted **3-ton (6,000 lb)** limit	Deck/beams failed; catastrophic deck damage and prolonged closure [136]
2024-12-30	WV (Roane Co.)	Bridge on Route 26 (Lick Fork Road), milepost 5.7	Tow truck	Reported as an **overweight** tow truck that broke through the deck	Bridge closed indefinitely; repair/replace assessment; detours established [137, 138]
2025-04-29	PA (Emmaus / Lower Macungie)	Wooden bridge to Dorothy Rider Pool Wildlife Sanctuary (Little Lehigh Creek)	Propane tanker truck	Posted **5 tons max**; tanker likely **2–3**× heavier per report	Span section collapsed during crossing; site access closed; investigation ongoing [139, 140]
2025-07-21	SC (Pickens Co.)	Crowe Creek Road bridge	Heavy truck (type not specified)	SCDOT: damage/collapse caused by a truck **exceeding the restricted weight limit**	Portion collapsed; closure expected to last months [141]
2025-11-18	WV (Monongalia Co., near Morgantown)	Small bridge at beginning of Birchfield Run Road	Truck (overweight)	WVDOH: an **overweight truck** broke through the bridge deck	Partial collapse; emergency deck replacement planned [142]

trucks ”unable” to use the bridge. Because the fractions of (a) trucks that fail to detour and (b) those that actually overload the bridge are not observed, these unknown factors are absorbed into a parameter in the damage term rather than estimated explicitly.

(2) Consequence / vulnerability. The overload consequence model is taken to depend primarily on superstructure condition (NBI Item 59) and bridge age. The idea is that weaker/older superstructures are more susceptible to overload-induced damage.

Under the report's general framework, a unitless risk index represents the combined effect of hazard likelihood and bridge vulnerability, and is then converted to annualized economic impact by scaling with (i) agency consequence proportional to replacement cost and (ii) user consequence proportional to detour cost.

Step-by-step procedure **Step 1**. Obtain the bridge inventory (FDOT/Pontis) as the baseline asset set for overload-risk computation, including operating rating r_i (NBI item 66, Table A.48), functional class (interstate vs. other), average daily traffic ADT_i, and truck fraction of traffic τ_i.

Table A.48: How NBI Item 66 is interpreted in this report

A. Bridges carrying Interstate highways	
$r < 10{,}000$	Extremely low load-carrying capacity (very restrictive). In the model, this implies essentially all trucks must detour (i.e., the bridge cannot accommodate typical truck weights).
$10{,}000 \leq r < 80{,}000$	Low-to-moderate capacity range. As r increases, fewer trucks are expected to be forced to detour; the model represents this as a smooth decreasing relationship in this interval.
$80{,}000 \leq r < 91{,}000$	Moderate-to-high capacity range. The model continues decreasing the detour tendency until it reaches zero at the upper threshold.
$r \geq 91{,}000$	High capacity relative to expected truck weights; the model treats this as no detouring triggered by rating limits (detour fraction = 0).
B. All other functional classes	
$r < 3{,}725$	Extremely low capacity for non-Interstate functional classes; the model treats this as essentially all trucks detour.
$3{,}725 \leq r < 85{,}000$	Low-to-high capacity range where increasing r reduces the expected detouring; the model uses a smooth decreasing relationship over this interval.
$r \geq 85{,}000$	High capacity; the model treats this as no detouring triggered by rating limits (detour fraction = 0).

In the report, r is explicitly defined as the bridge “operating rating” in pounds and identified as **NBI Item 66**; it is used as a continuous load-capacity indicator to estimate the fraction of trucks that would be detoured due to rating limits.

Step 2. Compute the percent of trucks detoured due to operating rating limits, denoted d_i (in percent), using a piecewise rule based on r_i. For bridges carrying interstate highways:

$$d_i = \begin{cases} 100, & r_i < 10000 \\ 102.24 - (8.982 \times 10^{-5}) r_i - (1.4336 \times 10^{-8}) r_i^2, & r_i < 80000 \\ 18.976 - (2.083 \times 10^{-4}) r_i, & r_i < 91000 \\ 0, & r_i \geq 91000 \end{cases} \tag{A.173}$$

and for all other functional classes:

$$d_i = \begin{cases} 100, & r_i < 3725 \\ 107.26 - (1.9743 \times 10^{-3}) r_i + (6.5265 \times 10^{-9}) r_i^2 + (2.2256 \times 10^{-14}) r_i^3, & r_i < 85000 \\ 0, & r_i \geq 85000 \end{cases} \tag{A.174}$$

Step 3. Convert the detoured-truck percentage to an exposure proxy: the estimated number of trucks unable to use the bridge per day,

$$N_i = \left(\frac{d_i}{100} \right) \cdot ADT_i \cdot \tau_i \tag{A.175}$$

This N_i is an intermediate quantity. The model assumes an unknown fraction of these trucks will fail to detour, and an unknown fraction of those will overload the bridge. These unknown fractions are absorbed later through calibration.

Step 4. Define a relative overload hazard likelihood (reported as "hazard prob (relative)"), denoted $p_{\text{overload}}(i)$, as proportional to N_i:

$$p_{\text{overload}}(i) \propto N_i \tag{A.176}$$

Step 5. The report relates overload consequences to superstructure condition (NBI Item 59) and bridge age. The reconstructed consequence multiplier uses the product of an element term and an age term calibrated to the magnitude of the reported Elem VulnC values.

Age multiplier. Let Y_i be the year built. Define

$$V_{\text{overload}}^{\text{age}}(i) = \begin{cases} 0.75, & Y_i \geq 1998, \\ 1 + \dfrac{1990 - Y_i}{35}, & 1970 < Y_i < 1998, \\ 1.25, & Y_i \leq 1970, \end{cases} \tag{A.177}$$

Element multiplier. Let $SC_i \in \{0, \ldots, 9\}$ be the superstructure condition rating (NBI Item 59):

$$V_{\text{overload}}^{\text{elem}}(i) = \begin{cases} 0.85, & SC_i \geq 9, \\ 1 + \dfrac{7 - SC_i}{40}, & 1 < SC_i < 9, \\ 1.15, & SC_i \leq 1. \end{cases} \tag{A.178}$$

Step 6. Compute the overload risk index and rank bridges in descending order:

$$\text{Risk}_{\text{overload}}(i) = p_{\text{overload}}(i) \cdot V_{\text{overload}}^{\text{elem}}(i) \cdot V_{\text{overload}}^{\text{age}}(i) \tag{A.179}$$

Agency cost. First, compute replacement cost for bridge i:

$$R_i = A_i c \tag{A.180}$$

where A_i is deck area and c is the unit replacement cost per area. Define the agency allocation weight over the bridge inventory set $\mathcal{E}$:

$$w_i = \frac{R_i, \text{Risk}_{\text{overload}}(i)}{\sum j \in \mathcal{E} R_j, \text{Riskoverload}(j)} \tag{A.181}$$

Given the statewide average annual agency loss from overloads L^Aoverload (annual historical agency cost), allocate it using w_i:

$$\text{AgencyCost}_{\text{overload}}(i) = L^A_{\text{overload}} \cdot w_i \tag{A.182}$$

User cost. For bridge i, define the daily detour cost:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}}; \frac{\ell_i}{v_i} \right) \tag{A.183}$$

where ℓ_i is detour length and v_i is detour speed. Define the user allocation weight:

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{overload}}(i)}{\sum_{j \in \mathcal{B}} U_j \cdot \text{Risk}_{\text{overload}}(j)} \tag{A.184}$$

Given the statewide annual control total for overload user losses L^U_{overload} (annual historical user cost), allocate it using u_i:

$$\text{UserCost}_{\text{overload}}(i) = L^U_{\text{overload}}, u_i \tag{A.185}$$

Total cost.

$$\text{TotalCost}_{\text{overload}}(i) = \text{AgencyCost}_{\text{overload}}(i) + \text{UserCost}_{\text{overload}}(i) \tag{A.186}$$

A.9.2.2 NCHRP Report 495: Effect of Truck Weight on Bridge Network Costs [12]

In NCHRP Report 495, the network-level (Level I) approach for identifying bridges vulnerable to truck overload/overstress is aligned with the AASHTO load rating concept: a bridge is considered overstressed for a specified evaluation truck if its rating factor is below 1.0. To avoid full re-rating of every bridge in a large network, Level I screening estimates the Alternative-Scenario rating factor using (i) existing rating factors in the bridge inventory and (ii) a moment-effect ratio between the Base Case rating vehicle and the Alternative Scenario vehicle, together with (iii) an adjustment to account for changes in the live-load factor implied by the truck weight histogram (TWH).

Step-by-step procedure **Step 1. Define scenarios and the governing rating vehicle.** Define a Base Case (BC) and an Alternative Scenario (AS) representing the overload condition. Here, BC denotes the current (status-quo) truck-loading environment under existing legal limits, permit policies, enforcement levels, and observed weight spectra (i.e., the present-day truck fleet and its associated truck weight histogram,

TWH_{BC}). AS denotes a changed truck-loading environment that is hypothesized for evaluation. For example, increased legal/permit limits, increased frequency or magnitude of overweight/overload events, or the introduction of a heavier representative permit vehicle. It is represented by a corresponding alternative weight spectrum TWH_{AS} (and/or alternative rating vehicle). Select the rating vehicle(s) used by the agency and identify the governing rating vehicle for each bridge (inventory or operating rating, consistent with agency practice). Bridges already deficient under the Base Case (i.e., with $RF_{BC} < 1.0$ for the governing rating vehicle) are excluded from the incremental-deficiency impact analysis, which focuses on deficiencies newly induced by the transition from BC to AS.

Step 2. Assemble network-wide inputs from the bridge inventory (or NBI).

For each bridge i, obtain:

- The existing rating factor under the Base Case, $RF_{BC}(i)$, for the governing rating vehicle;
- Span type (simple/continuous) and span length class (for moment ratios);
- Annual daily truck traffic, $ADTT(i)$ (truck AADT), used in the live-load-factor adjustment;
- Deck area A_i (if later converting vulnerability to a cost-impact metric).

Step 3. Develop truck loading statistics for BC and AS (TWH-based).

Obtain or predict the truck weight histogram (TWH) for the Base Case (BC) and the Alternative Scenario (AS). Here, a TWH is a binned representation of the distribution of gross vehicle weight (GVW) for the truck stream relevant to a bridge/site (e.g., from WIM observations, permit records, or an agency/federal default by functional class). Denote these scenario-specific histograms by TWH_{BC} and TWH_{AS}, respectively.

For each scenario, compute summary statistics of the upper tail of the GVW distribution, defined as the heaviest 20% of the trucks in the corresponding TWH. Specifically, let

$$W^*_{BC},\ \sigma^*_{BC}, \qquad W^*_{AS},\ \sigma^*_{AS}$$

where:

- W^*_{BC} is the mean GVW of the top 20% of the truck weights under the Base Case histogram TWH_{BC}.
- σ^*_{BC} is the standard deviation of GVW within the top 20% of the Base Case histogram TWH_{BC}.
- W^*_{AS} is the mean GVW of the top 20% of the truck weights under the Alternative Scenario histogram TWH_{AS}.
- σ^*_{AS} is the standard deviation of GVW within the top 20% of the Alternative Scenario histogram TWH_{AS}.

W^* and σ^* summarize the upper-tail (heavier-than-typical) portion of the truck stream, which is most influential for rating-related demand and overstress screening.

Next, determine the traffic-dependent multiplier $t(ADTT)$ from the tabulated relationship in Table A.49. for each bridge i, using its truck traffic level:

- $ADTT(i)$ is the Average Daily Truck Traffic for bridge i (typically in trucks/day), i.e., the daily volume of trucks (not total vehicles) expected to cross the bridge.
- $t(ADTT)$ is a dimensionless traffic multiplier that accounts for the increased likelihood of critical loading effects (e.g., multiple-presence / concurrent heavy trucks) as truck traffic increases; it is obtained by looking up (or interpolating) the value corresponding to $ADTT(i)$.

Table A.49 is a lookup table that converts a bridge's truck traffic level (ADTT) and number of lanes into the multiplier $t(ADTT)$, which is a traffic multiplier that makes the load estimate larger when more trucks use

the bridge, because heavy trucks are more likely to be on the bridge at the same time.

Table A.49: Function t

ADTT	t(ADTT)	
	Two or more lanes	One lane
5000	4.3	4.9
1000	3.3	4.5
100	1.5	3.9

Source: [12, Table A-3.1]

Step 4. Compute moment-effect ratios for the rating vehicle under BC vs. AS. For each bridge (or span class), determine the maximum bending moment due to:

$$M_{BC,\text{ rating}} \quad \text{and} \quad M_{AS,\text{ rating}}$$

where $M_{BC,\text{ rating}}$ is due to the current governing rating vehicle and $M_{AS,\text{ rating}}$ is due to the new rating vehicle under the Alternative Scenario. For continuous spans, evaluate multiple commonly critical sections and take the maximum ratio:

$$\left(\frac{M_{BC,\text{ rating}}}{M_{AS,\text{ rating}}}\right)_i = \max_{s\in\mathcal{S}_i}\left(\frac{M_{BC,\text{ rating}}(s)}{M_{AS,\text{ rating}}(s)}\right)$$

Cap the ratio at 1.0:

$$\left(\frac{M_{BC}}{M_{AS}}\right)_i \leftarrow \min\left\{1,\ \left(\frac{M_{BC}}{M_{AS}}\right)_i\right\}$$

because a new vehicle that induces a smaller moment effect leaves the current rating vehicle as the controlling case.

In this report, the bending moments $M_{BC,\text{rating}}$ and $M_{AS,\text{rating}}$ are obtained by applying the Base-Case and Alternative-Scenario rating vehicles as moving axle-load trains on representative simple or continuous span models. The maximum bending-moment effect at the applicable critical sections supplies the ratio between the BC and AS vehicles used in the screening calculation.

Step 5. Compute the live-load-factor adjustment ratio AF_{rating}. Using the TWH tail statistics and traffic multiplier, compute:

$$AF_{\text{rating}}(i) = \frac{2W^*_{AS} + 1.41\,t(ADTT_{AS}(i))\,\sigma^*_{AS}}{2W^*_{BC} + 1.41\,t(ADTT_{BC}(i))\,\sigma^*_{BC}} \tag{A.187}$$

In typical applications, $ADTT_{AS}(i)$ may be set equal to $ADTT_{BC}(i)$ unless the scenario explicitly changes truck volumes. The expression $2W^* + 1.41\,t(ADTT)\,\sigma^*$ can be interpreted as a simple "extreme live-load" index for multiple-presence truck loading. The term $2W^*$ represents the mean total weight of two heavy trucks present on the bridge at the same time (using the upper-tail mean W^* of the truck-weight distribution). The additive term $1.41\,t(ADTT)\,\sigma^*$ is an uncertainty/extreme-event margin: σ^* measures the variability of heavy-truck weights (upper-tail standard deviation), $t(ADTT)$ increases this margin when truck traffic is higher (more opportunities for unusually heavy concurrent trucks), and $1.41 \approx \sqrt{2}$ reflects the statistical fact that when two independent truck weights are added, the standard deviation of their sum increases by about $\sqrt{2}$.

Step 6. Estimate the Alternative-Scenario rating factor RF_{AS}. Compute the screened rating factor under

the Alternative Scenario:

$$RF_{AS}(i) = RF_{BC}(i) \left(\frac{M_{BC,\text{ rating}}}{M_{AS,\text{ rating}}} \right)_i \Big/ AF_{\text{rating}}(i) \tag{A.188}$$

In this equation, $RF_{BC}(i)$ is the bridge's current rating factor under the Base Case, meaning the safety margin for the governing rating vehicle under today's loading environment. The ratio $\left(M_{BC,\text{rating}}/M_{AS,\text{rating}}\right)_i$ accounts for the change in vehicle demand: if the Alternative Scenario vehicle produces larger bending moments, then $M_{AS,\text{rating}}$ is larger and the ratio is less than 1, which lowers the rating. The factor $AF_{\text{rating}}(i)$ accounts for changes in the truck weight spectrum and concurrent-truck loading: if the Alternative Scenario has heavier trucks or higher likelihood of simultaneous heavy trucks, then $AF_{\text{rating}}(i) > 1$, and dividing by it further lowers the rating. Therefore, $RF_{AS}(i)$ estimates the bridge's safety margin under the overload scenario; $RF_{AS}(i) < 1.0$ indicates the bridge would be overstressed under Alternative Scenario conditions.

Step 7. Flag newly vulnerable/deficient bridges (overstress screening). A bridge is considered overstressed for the scenario vehicle if:

$$RF_{AS}(i) < 1.0$$

If an agency applies an allowance (e.g., 5%) for certain decision contexts, a practical screening threshold may be used (e.g., $RF_{AS}(i) < 0.95$), with the caveat that operating rating is conceptually the absolute maximum and is typically not paired with an allowance.

Step 8. Produce a network-wide vulnerability score and ranking. Use the computed $RF_{AS}(i)$ to screen and rank overload-vulnerable bridges across the network. The key is to construct the Alternative Scenario (AS) so that it represents an overload environment. In this report, AS can be built in two practical ways.

First, build AS as an overload truck stream by modifying the truck weight histogram. Use WIM, enforcement, or permit data to create an AS truck weight histogram TWH_{AS} that includes overweight vehicles or a heavier upper tail than the Base Case histogram TWH_{BC}. Then compute the upper tail statistics W^*_{AS} and σ^*_{AS} from the heaviest 20% of TWH_{AS} and use them in $AF_{\text{rating}}(i)$.

Second, build AS as an overload vehicle model by selecting one or more representative overweight or permit trucks. Choose the practical maximum vehicle or a small set of vehicles that forms a bending moment envelope over representative spans. Use these vehicles to compute $M_{AS,\text{rating}}$ for the moment ratio term.

After AS is defined, bridges with smaller $RF_{AS}(i)$ have less safety margin under overload conditions and are more vulnerable. Bridges with $RF_{AS}(i) < 1.0$ are flagged as overstressed under AS conditions. For prioritization, rank bridges by $RF_{AS}(i)$ in ascending order, or use a severity score such as

$$VI(i) = \max\{0,\ 1 - RF_{AS}(i)\}$$

and rank bridges in descending order of $VI(i)$.

A.10 Over-Height Collision Risk

A.10.1 Introduction

Over-height collision risk is the probability that a vehicle taller than the posted vertical clearance will collide with a bridge structure, multiplied by the severity of potential consequences on structural integrity, traffic flow, and safety.

Table A.50: Selected U.S. bridge damage / major disruption events caused by overheight vehicle collisions (2021–2025).

Date	State	Bridge / Route	Overheight vehicle (load) and impact summary	Source
2021-07-19	MA	Roosevelt Circle Overpass (I-93 S, Medford)	Overheight permitted-load carrier struck the overpass; required temporary repairs and lane restrictions; MassDOT indicated permanent repairs would take months.	[145]
2022-01-13	NY	Onondaga Lake Parkway railroad bridge (Salina / Syracuse area)	Truck wedged under low-clearance bridge (bridge strike / lodging); recurring overheight issue; roadway impacts reported.	[146]
2023-08-31	NY	South Broadway overpass (South Nyack), I-87/I-287	Overheight tractor-trailer strike; subsequent operation reported to remove damaged girders and stabilize the overpass, with extended lane closures.	[147]
2023-12-21	NY	Railroad bridge (Glenville)	Truck carrying compressed natural gas struck railroad bridge and exploded while a train passed overhead; major incident response and disruption.	[148]
2024-04-01	PA	Conrail bridge over I-95 (Philadelphia)	Truck hauling an oversized load struck the bridge; I-95 closure and structural inspection/repairs reported.	[149]
2025-05-27	TX	FM 928 overpass / roadway along I-27 (Swisher County)	Crane arm (overheight) struck an overpass/roadway; reported bridge/roadway damage and resulting traffic impacts.	[150]
2025-08-26	MI	Overpass on US-131 (Allegan County)	Semi-truck hit an overpass bridge; debris/damage prompted partial closure and traffic disruption.	[151]
2025-09-02	TX	Hwy 29 (SH 29 / W. University) bridge over I-35 (Georgetown)	18-wheeler hauling an excavator struck the bridge; city report describes beam damage and multi-week repair timeline.	[152]
2025-09-04	TX	Houston Avenue bridge (Houston)	Overheight load struck the Houston Avenue bridge; reported as another bridge-hit incident with traffic impacts.	[153]
2025-10-21	WA	Bullfrog Road overpass (I-90 WB, Exit 80 near Cle Elum)	Over-height load struck overpass causing significant damage; WSDOT reported the damaged portion was not repairable and required demolition.	[154]
2025-12-18	OK	Will Rogers Turnpike (I-44): multiple overpasses	Overheight truck strike damaged multiple (reported six) overpasses on the turnpike, prompting closures and repairs.	[155]

Representative Case Studies in Texas

- Overheight Truck Strikes Two Bridges (I-20, West Texas) (2018)
  - A truck driver hauling over-sized, over-height equipment along I-20 in West Texas struck two separate bridges during a single run. The vehicle first hit the Business 20 bridge in Stanton and then went on to strike the County Road 1150 bridge near Midland, prompting closures and requiring structural inspections [156].
- Houston Avenue Bridge Struck (I-10, Houston, TX) (2018)
  - On June 25, 2018, an over-height truck slammed into the Houston Avenue bridge over I-10 in Houston, causing traffic snarls and forcing two lanes to be blocked while debris was cleared [157].
- 18-Wheeler Strikes SH 29 Bridge (I-35, Georgetown, TX) (2023)
  - On September 2, 2023, an 18-wheeler carrying an excavator exceeding the height limit struck the State Highway 29 bridge over I-35 in Georgetown, injuring two people and causing significant damage to the overpass. The impact led to lane closures and emergency repairs [158].
- Pedestrian Bridge Struck by Over-height Truck (I-610 South Loop, Houston) (2024)
  - In September 2024, an 18-wheeler exceeded clearance limits and struck a pedestrian bridge over the southbound lanes of the I-610 South Loop near Fannin and Kirby in Houston. The collision blocked traffic until the over-height truck could be removed. TxDOT crews later closed lanes for repair work in early 2025 [159].
- Houston Avenue Bridge Frequent Over-height Hits (I-10, Houston, TX) (2025)
  - The Houston Avenue bridge over I-10 - one of Houston's low-clearance bridges (14 ft 3 in) - has been struck repeatedly by over-height trucks, with 67+ collisions involving tall vehicles in 2025. Many collisions occur despite height detection systems and warning signage, and they frequently cause lane closures and debris on the roadway. Plans are underway for a 400 million USD project that will reconstruct the area to eliminate the clearance issue in 2027 [160].

A.10.2 Methods Reported in the Literature

A.10.2.1 FDOT Assessment Procedure: Over-height Collisions [1]

The report treats over-height collisions as a man-made (unintentional) hazard driven primarily by insufficient vertical clearance. The likelihood component is formulated from a truck-height exceedance model: for each bridge, the vertical clearance (NBI Item 10) is mapped to an estimated percent of trucks that should detour because they are too tall to safely pass under (or through) the structure, with separate piecewise relationships for interstate bridges versus other functional classes. This detoured-truck fraction is then combined with traffic exposure (ADT and truck percentage) to estimate the volume of trucks that are effectively "in conflict" with the clearance restriction; the framework assumes that only an unknown fraction of these trucks fail to detour, and only an unknown fraction of those failures result in an actual strike, yielding an annual over-height impact hazard level for ranking.

On the consequence side, the report uses historical accident/inspection records (Pontis inspevent narratives) to characterize typical damage outcomes of over-height impacts. Observed cases are classified into qualitative damage levels (e.g., minor, moderate, severe/extreme), and the distribution of affected bridge elements

is summarized to identify which components are most exposed (with beams/girders dominating, and occasional deck/slab involvement). These empirical summaries establish relative consequence and vulnerability factors at the bridge-element and aggregate levels. The factors are combined in a bridge-level index for network screening and prioritization of over-height collision risk.

Step-by-step procedure **Step 1**. Obtain the bridge inventory (FDOT/Pontis bridge GIS layer) as the baseline asset set for risk computation.

Step 2. For each bridge, obtain (i) vertical clearance VC over the roadway (NBI Item 10, in feet), (ii) functional class indicator (to distinguish interstate highways vs. all other functional classes), and (iii) traffic inputs on the affected roadway: average daily traffic ADT_i and truck fraction f_i^{truck} (fraction of ADT that are trucks).

Step 3. Estimate the *percent of trucks detoured due to vertical clearance restrictions* using the truck height model. Let $d(VC_i)$ denote the percent detoured (in %). The model is piecewise by functional class:

$$d(VC) = \begin{cases} 100, & \text{Interstate: } VC < 9.65 \\ 855.91 - 223.43\,VC + 22.199\,VC^2 - 0.74236\,VC^3, & \text{Interstate: } 9.65 \le VC < 13 \\ 1.0956 \times 10^{56}\,VC^{-48.683}, & \text{Interstate: } 13 \le VC < 14 \\ 14.567 - 0.9046\,VC, & \text{Interstate: } 14 \le VC < 16.1 \\ 0, & \text{Interstate: } VC \ge 16.1 \\ 100, & \text{Other classes: } VC < 7.3 \\ -26.275 + 34.692\,VC - 2.3894\,VC^2, & \text{Other classes: } 7.3 \le VC < 13.5 \\ 138.86 - 9.886\,VC, & \text{Other classes: } 13.5 \le VC < 14 \\ 0, & \text{Other classes: } VC \ge 14 \end{cases} \tag{A.189}$$

Step 4. Convert the detour percent to an exposure proxy: estimate the number of trucks unable to use the crossing (per day) as

$$N_i^{\text{detour}} = \frac{d(VC_i)}{100}\, ADT_i\, f_i^{\text{truck}} \tag{A.190}$$

Step 5. Model the over-height collision hazard likelihood. The report assumes the hazard probability for over-height collisions is proportional to the fraction of trucks detoured due to vertical clearance restrictions, i.e., proportional to the exposure proxy above. Since the fraction of trucks that fail to detour and the fraction that actually strike the bridge are unknown, the hazard term is used as a relative likelihood indicator (and can later be scaled through risk allocation / calibration):

$$p_{\text{height}}(i) \;\propto\; N_i^{\text{detour}} \tag{A.191}$$

Step 6. Compute the over-height collision element vulnerability multiplier $V_{\text{height}}^{\text{elem}}(i)$. For each bridge, let $\mathcal{E}$ denote the set of bridge elements recorded for that bridge. For each element e, obtain its overhead-collision element score $s_{\text{height}}(e)$ from Table A.51, where $s_{\text{height}}(e) \in \{0, 1, 2, 3, 4, 5\}$. The bridge-level raw element

score is the simple average:

$$\bar{s}_{\text{height}} = \frac{1}{|\mathcal{E}|} \sum_{e \in \mathcal{E}} s_{\text{height}}(e) \tag{A.192}$$

Then rescale to obtain the reported multiplier:

$$V^{\text{elem}}_{\text{height}}(i) = \frac{\bar{s}_{\text{height}} + 1}{3} \tag{A.193}$$

Table A.51: Levels of vulnerability of bridge elements to damage from overhead collisions [1]

Table 7.26. Levels of vulnerability of bridge elements to damage from overhead collisions

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	3	216	Timber Abutment	1	476	Timber Walls	1
13	Unp Conc Deck/AC Ovl	3	217	Other Mtl Abutment	1	477	Other Walls	1
28	Steel Deck/Open Grid	3	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	1
29	Steel Deck/Conc Grid	3	230	Unpnt Stl Cap	1	480	Mast Arm Found	0
30	Corrug/Orthotpc Deck	3	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	0
31	Timber Deck	3	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	0
32	Timber Deck/AC Ovly	3	234	R/Conc Cap	1	483	Other Mast Arm Vert	0
38	Bare Concrete Slab	3	235	Timber Cap	1	484	Paint Mast Arm Horzn	0
39	Unp Conc Slab/AC Ovl	3	240	Metal Culvert	1	485	Galvan Mast Arm Horz	0
54	Timber Slab	3	241	Concrete Culvert	0	486	Other Mast Arm Horzn	0
55	Timber Slab/AC Ovly	3	242	Timber Culvert	0	487	Sign Member Horiz	0
98	Conc Deck on PC Pane	3	243	Misc Culvert	0	488	Sign Member Vertical	0
99	PS Conc Slab	3	290	Channel	1	489	Sign Foundation	0
101	Unpnt Stl Box Girder	5	298	Pile Jacket Bare	1	495	Uncoat High Mast L.	0
102	Paint Stl Box Girder	5	299	Pile Jacket/Cath Pro	1	496	Painted High Mast L.	0
104	P/S Conc Box Girder	5	300	Strip Seal Exp Joint	3	497	Galvan. High Mast L.	0
105	R/Conc Box Girder	5	301	Pourable Joint Seal	3	498	Other High Mast L.P.	0
106	Unpnt Stl Opn Girder	5	302	Compressn Joint Seal	3	499	H. M. L. P. Found.	0
107	Paint Stl Opn Girder	5	303	Assembly Joint/Seal	3	540	Open Gearing	0
109	P/S Conc Open Girder	5	304	Open Expansion Joint	3	541	Speed Reducers	0
110	R/Conc Open Girder	5	310	Elastomeric Bearing	3	542	Shafts	0
111	Timber Open Girder	5	311	Moveable Bearing	3	543	Shaft Brgs and Coupl	0
112	Unpnt Stl Stringer	5	312	Enclosed Bearing	3	544	Brakes	0
113	Paint Stl Stringer	5	313	Fixed Bearing	3	545	Emergency Drive	0
115	P/S Conc Stringer	5	314	Pot Bearing	3	546	Span Drive Motors	0
116	R/Conc Stringer	5	315	Disk Bearing	3	547	Hydraulic Power Unit	0
117	Timber Stringer	5	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	0
120	U/Stl Thru Truss/Bot	3	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	0
121	P/Stl Thru Truss/Bot	3	330	Metal Rail Uncoated	3	550	Hopkins Frame	0
125	U/Stl Thru Truss/Top	3	331	Conc Bridge Railing	3	560	Locks	0
126	P/Stl Thru Truss/Top	3	332	Timb Bridge Railing	3	561	Live Load Shoes	0
130	Unpnt Stl Deck Truss	3	333	Other Bridge Railing	3	562	Counterweight Suppor	0
131	Paint Stl Deck Truss	3	334	Metal Rail Coated	3	563	Acc Ladd & Plat	0
135	Timber Truss/Arch	3	356	Steel Fatigue SmFlag	0	564	Counterweight	0
140	Unpnt Stl Arch	3	357	Pack Rust Smart Flag	0	565	Trun/Str and Cur Trk	0
141	Paint Stl Arch	3	358	Deck Cracking SmFlag	0	570	Transformers	0
143	P/S Conc Arch	3	359	Soffit Smart Flag	0	571	Submarine Cable	0
144	R/Conc Arch	3	360	Settlement SmFlag	0	572	Conduit & Junc. Box	0
145	Other Arch	3	361	Scour Smart Flag	0	573	PLCs	0
146	Misc Cable Uncoated	3	362	Traf Impact SmFlag	0	574	Control Console	0
147	Misc Cable Coated	3	363	Section Loss SmFlag	0	580	Navigational Lights	0
151	Unpnt Stl Floor Beam	3	369	Sub.Sect Loss SmFlag	0	581	Operator Facilities	0
152	Paint Stl Floor Beam	3	370	Alert Smart Flag	0	582	Lift Bridge Spec. Eq	0
154	P/S Conc Floor Beam	3	386	Fender/Dolphin Uncoa	0	583	Swing Bridge Spec. E	0
155	R/Conc Floor Beam	5	387	P/S Fender/Dolphin	0	590	Resistance Barriers	0
156	Timber Floor Beam	5	388	R/Conc Fender/Dolphi	0	591	Warning Gates	0
160	Unpnt Stl Pin/Hanger	3	389	Timber Fender/Dolphi	0	592	Traffic Signals	0
161	Paint Stl Pin/Hanger	3	390	Other Fender/Dolphin	0			
201	Unpnt Stl Column	3	393	Blkhd Sewl Metal Unc	0			
202	Paint Stl Column	3	394	R/Conc Abut Slope Pr	1			
204	P/S Conc Column	3	395	Timber Abut Slope Pr	1			
205	R/Conc Column	3	396	Other Abut Slope Pro	1			
206	Timber Column	3	397	Drain. Syst Metal	1			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	1			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	1			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	1			
215	R/Conc Abutment	1	475	R/Conc Walls	1			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Source: [1, Table 7.26]

Note: The vulnerability values in Table A.51 were assigned as an ordinal (0-5) rating for each Pontis element key using FDOT historical damage evidence from two internal data streams and a judgmental

synthesis step:

(1) the report reviews Pontis accident-inspection records (the **inspevent** table), reading the inspectors' notes to identify the crash type and which bridge elements were damaged for each event. This produces the over-height collision sample (154 over-height crashes within 398 reviewed accident records) and the list of damaged element categories;

(2) for the over-height subset, the report summarizes (a) the distribution of damage levels observed in the over-height crash narratives (Table A.52 and Table A.53) and (b) which element types were hit most often (Table A.54), i.e., an empirical damage likelihood by element group in over-height events.

(3) the "mapping" is a judgment-based translation: the report uses the over-height crash summaries (e.g., Table A.54) to rank element groups by observed damage frequency, then assigns higher ordinal ratings (closer to 5) to the element keys in those frequently damaged, structurally critical groups, while assigning lower ratings to rarely affected or noncritical elements. The MMS repair-cost/damage-level summaries (Table A.55, Table A.56, Table A.57) are used as a qualitative check to keep elements with more severe/expensive repairs in higher bins; elements with no observed damage are given a minimum score of 1 to avoid zero vulnerability.

Table A.52: Established damage levels to bridges due to vehicular over-height crashes

Table 7.16. Established damage levels to bridges due to vehicular over-height crashes

Level of Damage	Bridge ID	Incident Description	Elements Affected	Comments	MMS No.#
Severe	010088	track hoe mounted on a semi rig lowboy trailer traveling northbound struck this bridge at span 5 south side, midspan. The collision severely damaged the span necessitating closure of Rampart Blvd over I-75 by first responders. The roadway underneath also sustained some damage.	98, 109		
Moderate	720092	northbound long-bed dump truck struck and damaged/destroyed 2 of 6 beams in Span 3.	109		9190714
Minor	720177	unknown overheight vehicle traveling westbound on 20th St. struck Beams 2-1, 2-2, 2-4, 2-6, 2-14, 2-16, 2-18, 2-20 and 2-24. The bottom east flange of these beams have spalls averaging 12 in. x 4 in. x 1/2 in. with no exposed steel.	Beams	no repairs are requested	
No Damage	290082	an overheight vehicle was reported to be stuck under one of the overpasses at I-75 and US-441 in Columbia County. Beams 2-9 and 2-1appeared to have the most recent damage which consisted of minor scrapes on bottom flange, over outside lane, of Northbound 441.	109	No damage. No evidence of damage/debris on under route roadway.	
# FDOT's Maintenance Management System (MMS): work order prepared for repair.					

Source: [1, Table 7.16]

Step 7. Compute the over-height collision risk index and rank bridges in descending order of $\text{Risk}_{\text{height}}$:

$$\text{Risk}_{\text{height}}(i) = p_{\text{height}}(i) \cdot V^{\text{elem}}_{\text{height}}(i) \tag{A.194}$$

Agency cost. First, compute the replacement cost for bridge i:

$$R_i = A_i c \tag{A.195}$$

Table A.53: Levels of damage on Bridges from vehicular over-height crashes

Table 7.17. Levels of damage on Bridges from vehicular over-height crashes

Level of damage	No. of bridge crashes	% of bridge crashes
No Damage	5	3.2%
Minor	43	27.9%
Moderate	90	58.4%
Severe	2	1.3%
Unknown	14	9.1%
Totals all crashes	154	

Source: [1, Table 7.17]

Table A.54: Effect of vehicular over-height crashes on bridge elements

Table 7.18. Effect of vehicular over-height crashes on bridge elements

	Element Type Affected	No. of element in crashes	% of all elements in crashes
Core	Deck and Slab	12	7.4%
	Beam: Girder and Stringer	137	84.0%
	Column	1	0.6%
	Abutment/Retaining Walls	1	0.6%
	Bearing	1	0.6%
	Railing	5	3.1%
	Signs, Electrical conduit/junction box.	6	3.7%
	Totals all elements in crashes	163	

Source: [1, Table 7.18]

Table A.55: Summary of bridge accidents repair costs from FDOT‟s MMS Cost Data

Table 7.21. Summary of bridge accidents repair costs from FDOT's MMS Cost Data

Bridge Element Type	Mean Cost ($)	Min Cost ($)	Max Cost ($)	Std Dev ($)	Coeff. Of variation	Count
Barrier	2,578.41	310.45	6,460.00	2,333.83	0.91	6
Beam	1,203.80	16.64	8,530.13	1,686.86	1.40	41
Fender	1,272.28	51.16	4,224.89	1,993.72	1.57	4
Guardrail/Handrail/Post	1,440.48	33.21	8,045.78	2,081.27	1.44	28
All Bridge Elements	1,307.70	14.55	8,530.13	1,844.27	1.41	100

Source: [1, Table 7.21]

Table A.56: Summary of repair total costs for bridge elements after accidents

Table 7.22. Summary of repair total costs for bridge elements after accidents

Bridge element no.	Damage level	mean ($)	stdev ($)	min ($)	max ($)	count
12	slight	1,002.11	NA	1,002.11	1,002.11	1
12	moderate	1,312.85	498.87	776.10	1,762.33	3
107	moderate	772.75	471.57	271.10	1,379.91	6
109	negligible	1,598.48	1,127.77	690.27	2,860.81	3
109	slight	1,165.91	1,582.89	142.95	4,629.87	7
109	moderate	3,085.51	3,579.48	18.85	17,418.25	48
109	severe	2,042.07	872.98	962.68	3,366.02	6
121	moderate	129.97	20.70	109.91	161.87	5
121	moderate	186.08	NA	186.08	186.08	1
205	moderate	3,959.59	NA	3,959.59	3,959.59	1
215	moderate	717.46	822.90	135.58	1,299.34	2
234	slight	562.40	NA	562.40	562.40	1
301	moderate	365.95	NA	365.95	365.95	1
321	moderate	275.70	NA	275.70	275.70	1
330	moderate	529.25	543.18	9.94	3,344.42	38
331	slight	486.66	265.75	298.74	674.57	2
331	moderate	2,703.80	2,594.38	16.31	13,431.80	54
333	moderate	2,478.94	2,628.92	254.01	11,277.61	22
334	moderate	712.00	627.99	18.10	1,542.94	4
386	moderate	3,100.50	4,053.35	497.29	7,770.65	3
387	moderate	539.74	481.76	8.60	948.54	3
394	moderate	1,563.15	1,580.63	339.08	3,315.47	5
396	moderate	910.47	NA	910.47	910.47	1
475	moderate	920.38	1,343.66	94.25	2,926.42	4
482	moderate	704.79	NA	704.79	704.79	1
487	moderate	2,145.15	NA	2,145.15	2,145.15	1
489	slight	433.83	NA	433.83	433.83	1
489	moderate	195.61	NA	195.61	195.61	1
563	moderate	1,495.85	176.56	1,371.00	1,620.70	2
572	moderate	329.03	388.52	8.60	1,234.04	12
580	moderate	309.29	401.46	27.36	1,080.11	6
railings, guardrails & handrails	moderate	3,270.79	2,608.82	461.69	6,988.64	7
Signs and gates	moderate	2,453.64	2,922.90	163.24	6,294.58	4

Source: [1, Table 7.22]

Table A.57: Summary of repair unit costs for bridge elements after accidents

Table 7.23. Summary of repair unit costs for bridge elements after accidents

Bridge element	Damage level	unit	mean ($)	stdev ($)	min ($)	max ($)	count
12	slight	SF	0.46	NA	0.46	0.46	1
12	moderate	SF	180.05	69.71	125.88	258.70	3
107	moderate	MH	28.55	2.19	25.82	31.67	6
109	negligible	MH	49.31	32.51	27.61	86.69	3
109	slight	MH	61.05	99.25	16.93	285.90	7
109	moderate	MH	37.57	46.14	17.93	345.76	48
109	severe	MH	27.31	4.15	19.85	31.95	6
121	moderate	MH	31.23	3.38	28.29	36.64	5
121	moderate	MH	18.61	NA	18.61	18.61	1
205	moderate	MH	21.70	NA	21.70	21.70	1
215	moderate	MH	26.98	4.45	23.84	30.13	2
234	slight	MH	22.27	NA	22.27	22.27	1
301	moderate	LF	91.49	NA	91.49	91.49	1
321	moderate	SF	275.70	NA	275.70	275.70	1
330	moderate	LF	22.98	38.21	1.74	222.96	37
331	slight	LF	NA	NA	NA	NA	0
331	moderate	LF	115.24	130.01	8.40	854.05	47
333	moderate	LF	53.60	38.79	4.10	132.26	21
334	moderate	LF	41.61	25.13	16.83	67.08	3
386	moderate	MH	28.54	3.07	26.70	32.08	3
387	moderate	MH	32.87	1.66	31.10	34.40	3
394	moderate	MH	34.11	33.23	9.41	92.04	5
396	moderate	MH	10.65	NA	10.65	10.65	1
475	moderate	MH	26.73	1.84	25.00	29.22	4
482	moderate	MH	35.24	NA	35.24	35.24	1
487	moderate	UN	71.51	NA	71.51	71.51	1
489	slight	UN	144.61	NA	144.61	144.61	1
489	moderate	UN	195.61	NA	195.61	195.61	1
563	moderate	MH	36.09	4.87	32.64	39.53	2
572	moderate	MH	31.74	3.84	25.37	37.88	12
580	moderate	MH	64.67	35.11	24.69	121.40	6
railings, guardrails & handrails	moderate	LF	102.12	92.59	9.62	240.99	7
Signs and gates	moderate	MH	27.16	3.34	23.81	31.70	4

Source: [1, Table 7.23]

where A_i is deck area and c is the unit replacement cost per area. Define the agency weight as the normalized share over the bridge inventory set $\mathcal{E}$:

$$w_i = \frac{R_i \cdot \text{Risk}_{\text{height}}(i)}{\sum_{j \in \mathcal{E}} R_j \cdot \text{Risk}_{\text{height}}(j)} \tag{A.196}$$

Given the statewide average annual agency loss from over-height collisions L_{height} (risk allocation control total), allocate it using w_i:

$$\text{AgencyCost}_{\text{height}}(i) = L_{\text{height}} w_i \tag{A.197}$$

User cost. For bridge i, define the daily detour cost:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}} \frac{\ell_i}{v_i} \right) \tag{A.198}$$

where ADT_i is average daily traffic, ℓ_i is detour length, v_i is detour speed, c_{op} is unit operating cost per mile, and c_{time} is value of time per hour. Define the user weight as the normalized share over the bridge inventory:

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{height}}(i)}{\sum_{j \in \mathcal{B}} U_j \cdot \text{Risk}_{\text{height}}(j)} \tag{A.199}$$

Given the statewide annual control total for over-height collision user losses L^U_{height}, allocate it using u_i:

$$\text{UserCost}_{\text{height}}(i) = L^U_{\text{height}} u_i \tag{A.200}$$

Total cost.

$$\text{TotalCost}_{\text{height}}(i) = \text{AgencyCost}_{\text{height}}(i) + \text{UserCost}_{\text{height}}(i) \tag{A.201}$$

A.10.2.2 NCHRP Project 08-139 [13]

This guide frames risk using two components: (i) likelihood, the chance of a bridge-and-tunnel-strike (BrTS) event occurring, and (ii) consequences, the impacts should an event occur. The risk assessment workflow in this chapter operationalizes likelihood primarily through a location-level risk score derived from risk factors (binary or model-based), and incorporates consequences using crash-severity costs (as implemented in the Maine example risk score).

Because OHVs are most directly associated with under-bridge (or tunnel entrance) collisions with the superstructure (e.g., girder), the steps below are written for that focus crash-type (Type B in Figure A.8).

Step-by-step procedure **Step 1.** Identify the candidate elements (sites) to analyze and create an element table.

Choose the element set for screening (e.g., all bridges on the network with under-bridge roadways, or a filtered subset such as bridges on particular corridors). Construct a table where each row is a site (bridge-under-bridge-roadway pair, or tunnel entrance), and columns include the variables needed for scoring.

The guide provides example element-table formats:

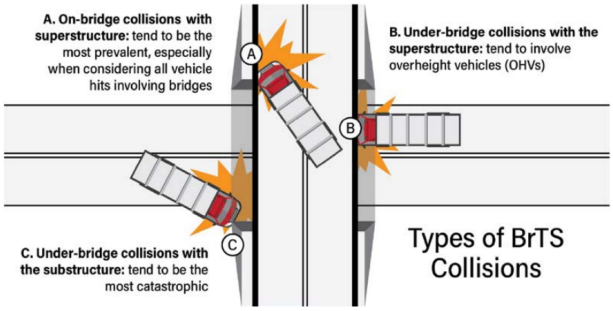


Figure A.8: Types of bridge collisions
Source: [13, Figure 1]

- Table A.58: bridges with under-bridge roadways (recommended for OHV under-bridge strikes).

Table A.58: Example element table for bridges and under-bridge roadways

Bridge Characteristics				Under-Bridge Roadway Characteristics					
Bridge ID	Age (year)	Max. Span Length (m)	Crash Count in 5-years	Functional Classification	AADT	% Trucks	# Lanes	Min. Vertical Clearance (m)	National Network for Truck
630280	42	20.4	12	Urban – Local	11,000	12	2	3.74	No
670265	59	19.8	2	Urban – Collector	38,500	12	4	4.49	Yes
510129	4	28.0	1	Urban – Principal Arterial – Other Freeways or Expressways	3,500	6	2	4.81	No

Source: [13, Table 5]

Step 2. Assemble an integrated dataset for each element.

For each site i, compile the risk-factor variables (bridge, roadway, and traffic attributes) into the element table. The guide recommends using GIS or spreadsheets to integrate spatial and tabular data to support scoring.

Step 3. Select risk factors relevant to OHV under-bridge (overhead-structure) or tunnel-entrance strikes.

Start from the guide's candidate and observed risk-factor lists:

- Table A.59: potential risk factors considered (exposure/design/condition and related variables),
- Table A.60: observed risk factors summarized by focus crash-type (use the column for "Under-Bridge Strikes with Overhead Structure (e.g., girder) or Tunnel Entrance").

Let the selected risk-factor set be $\mathcal{K}$, and define the site-level feature vector

$$\mathbf{x}_i = (x_{i1}, x_{i2}, \ldots, x_{iq})$$

where each x_{ij} is a quantitative or coded representation of risk-factor j for site i (e.g., minimum vertical clearance, AADT, truck AADT, functional class indicators, whether on the national truck network).

Step 4. Compute the location-level likelihood score (two options).

Table A.59: Potential risk factors for BrTS.

Bridge or Roadway Features	Description	On-Bridge	Under-Bridge or Tunnel Entrance
AADT	Annual average daily traffic	•	•
Approach rail (Y/N)	Presence of guardrail on approach	•	—
Approach rail end (Y/N)	Presence of guardrail end on approach	•	—
Approach roadway width	Width of usable roadway approaching bridge	•	—
Average shoulder width	Average shoulder width of roadway	•	•
Bridge age	Bridge age	•	•
Bridge length	Bridge length	•	•
Bridge serviceability and functional obsolescence	Bridge serviceability and functional obsolescence	•	•
Bypass, detour length	Bypass, detour length for roadway	•	•
Curb (Y/N)	Presence of curb on roadway	•	•
Curb width	Curb width	•	•
Curve (degree)	Degree of horizontal curve of roadway	•	•
Curve (k-value)	k-value of vertical curve of roadway	—	•
Curve length	Length of curve	•	•
Deck condition rating	Condition rating of bridge deck	•	•
Deck width	Deck width	•	•
Flare (Y/N)	Presence of flare (variable width of structure)	•	—
Functional classification	Functional classification of roadway	•	•
Horizontal clearance	Total horizontal clearance for roadway	•	•
Lateral clearance	Lateral clearance	•	•
Lighting	Type of lighting present on roadway	•	•
Maximum span	Length of maximum span of bridge	•	•
Median width	Width of median grass or sod	•	•
Narrow approach (Y/N)	Indicator for structure narrower than approach	•	—
National truck network	Indicator if roadway is part of national truck network	•	•
Number of lanes	Total number of lanes on roadway	•	•
Operation (1-way/2-way)	Direction of traffic (1-way/2-way) on roadway	•	•
Paved road (Y/N)	Presence of paved road	•	•
Railing (Y/N)	Presence of railings on bridge	•	—
Roadway width	Width of roadway	•	•
Rumble strip (Y/N)	Presence of longitudinal rumble strips	•	•
Shoulder type	Pavement type of shoulders on roadway	•	•
Speed limit	Speed limit of roadway	•	•
Surface type	Pavement type of lanes on bridge	•	•
Transition (Y/N)	Presence of transition rails	•	—
Truck AADT	Truck annual average daily traffic	•	•
Truck percentage	Percentage of truck traffic	•	•
Variable vertical clearance	Vertical clearance varies by lane	•	•
Vertical clearance	Vertical clearance over roadway	•	•

Note: — = Not a risk factor.

Source: [13, Table 2]

Table A.60: Summary of observed risk factors for BrTS focus crash-types.

Category	Variable/Risk Factors	On-Bridge Strikes (e.g., truss, parapet, railing)	Under-Bridge Strikes with Overhead Structure (e.g., girder) or Tunnel Entrance	Under-Bridge Strikes with Pier or Support (e.g., bent, support, pier, abutment) or Tunnel Entrance
Exposure	One-way traffic/dual carriage	•	—	•
	Larger AADT	•	•	•
	Larger truck AADT	•	•	•
	More than two travel lanes	—	•	•
	Part of national truck network	—	—	•
	Rural arterial	•	—	—
	Rural local	—	—	•
	Rural major collector	•	—	—
	Rural minor collector	•	—	—
	Urban local	—	•	•
	Urban minor arterial	—	—	•
	Urban other principal arterial	—	—	•
Design	Concrete shoulder	—	—	•
	Concrete surface	—	—	•
	Curve (larger degree)	—	—	•
	Larger bridge roadway width	•	—	—
	Larger deck width	•	—	—
	Larger lateral clearance	—	•	—
	Longer bridge length	•	—	—
	Lower vertical clearance	—	•	—
	Approach road narrower than bridge	•	—	—
	No transition from road to bridge	•	—	—
	Presence of longitudinal rumble strip	—	—	•
	Presence of railing on bridge	•	—	—
	Presence of guardrail on approach	•	—	—
	Structure flared	•	—	•
	Unpaved road	•	—	—
Condition	Lower deck condition rating	•	—	—
	Older bridge	•	•	—

Note: — = Not a risk factor.

Source: [13, Table 3]

Option A: Binary risk-factor score (screening score). For each site i and risk factor $k \in \mathcal{K}$, define a binary indicator:

$$I_{ik} = \begin{cases} 1, & \text{if risk factor } k \text{ is present at site } i, \\ 0, & \text{otherwise.} \end{cases}$$

A basic unweighted risk score is the sum of present factors:

$$R_i = \sum_{k \in \mathcal{K}} I_{ik}$$

If the agency applies factor importance weights w_k (user-defined or guided by overrepresentation evidence), use:

$$R_i = \sum_{k \in \mathcal{K}} w_k I_{ik}$$

Option B: Regression-based risk-factor score. Use a predictive crash model to estimate the expected number of BrTS crashes and use the prediction as the risk score. The guide presents the industry-standard negative binomial (NB) crash-count formulation in a multiplicative form. A general prediction equation consistent with the guide's formulation is:

$$\widehat{N}_i = n \cdot L_i \cdot AADT_i^{\alpha} \ \exp\left(\beta_0 + \sum_{j=1}^{q} \beta_j x_{ij}\right) \tag{A.202}$$

where

- $\widehat{N}_i$ is the estimated number of BrTS crashes at site i over the study period,
- n is the number of years in the study period (e.g., $n = 5$),
- L_i is the relevant structure length exposure term (for OHV under-bridge overhead-structure strikes, the guide specifies using the length of deck),
- $AADT_i$ is the annual average daily traffic associated with the crash type being modeled (for under-bridge strikes, use the under-bridge roadway AADT),
- x_{ij} are explanatory variables (risk factors),
- $\alpha, \beta_0, \{\beta_j\}$ are estimated coefficients from the fitted model.

If an annualized likelihood measure is desired, convert the n-year prediction to an annual expected count:

$$\widehat{\lambda}_i = \frac{\widehat{N}_i}{n}$$

Step 5. Incorporate consequences via severity-cost weighting (optional; Maine implementation). If the objective is a cost-based risk score rather than a purely likelihood-based score, the guide illustrates a severity-cost approach. Let $\widehat{N}_i$ be the predicted crash count (e.g., from the regression-based method), and let p_s denote the predicted proportion in severity category $s \in \mathcal{S}$ (KABCO categories). Then:

$$\widehat{N}_{i,s} = \widehat{N}_i \, p_s$$

Let c_s be the comprehensive unit crash cost for severity s. Compute the total crash cost at site i:

$$C_i = \sum_{s \in \mathcal{S}} c_s \, \widehat{N}_{i,s}$$

Finally, normalize to a standardized risk score on $[0, 100]$ across the analysis set:

$$RS_i = 100 \cdot \frac{C_i - C_{\text{min}}}{C_{\text{max}} - C_{\text{min}}}$$

where C_{min} and C_{max} are the minimum and maximum C_i values across all candidate sites being scored.

Step 6. Prioritize elements for diagnosis and treatment. Rank sites in descending order of the chosen score:

- likelihood-based ranking using R_i (binary) or $\widehat{N}_i$ / $\widehat{\lambda}_i$ (regression-based), or
- consequence-weighted ranking using C_i or standardized RS_i.

The guide characterizes this as a simple ranking step following scoring.

A.11 Seismic Risk

A.11.1 Introduction

Seismic risk refers to the probability and potential consequences of damage or failure of a bridge due to earthquake ground motions. Unlike other risks (flood, truck collisions, over-height loads), seismic risk arises from natural ground shaking and related geo-technical effects that can stress or destabilize bridge structures.

Representative Case Studies in Texas

- Texas is in a low-seismic region. Although there are earthquakes in Texas (e.g., 1931 Valentine earthquake (M5.8), 1995 Marathon Earthquake (M5.7), and West Texas quakes of about M5.0 in 2025), their magnitudes are not high enough to damage bridgesin Texas. The Research Team did not find any bridge damage reported in association with earthquakes.

A.11.2 Methods Reported in the Literature

A.11.2.1 FHWA Bridge Risk Report [14]

This procedure follows a risk = likelihood × consequence formulation suitable for statewide bridge screening and prioritization. For each bridge, seismic risk is computed by combining (i) a site-specific seismic hazard representation (annual exceedance rates for an intensity measure) with (ii) an expected consequence model (e.g., repair-cost ratio, closure days, or a proxy impact score). The output is a bridge-level annual risk score that can be used to rank bridges and identify the highest-risk assets for more detailed evaluation.

Table A.61: U.S. bridge earthquake-damage events with publicly documented, bridge-specific reporting (2021–2025).

Date (local)	Mw	Epicentral area (State)	Damaged bridge(s) (owner / route)	Sources
2022-12-20	6.4	Near Ferndale / Humboldt County (CA)	**Eel River Bridge ("Fernbridge")**, Bridge No. 04-0134 (Caltrans / SR-211). Reported as having *substantial/extensive* earthquake damage; closed for safety evaluation and emergency repairs; later restored to two-way traffic (repairs and monitoring actions described in project documentation).	[161–163]
2023-01-01	5.x (aftershock)	Humboldt County (CA)	**Eel River Bridge (Fernbridge)**, Bridge No. 04-0134 (Caltrans / SR-211). A follow-on earthquake in the same sequence was reported to add stress and contribute to additional documented damage and continued response/repairs.	[161, 162]

Step-by-step procedure **Step 1**. Obtain the statewide bridge inventory as the baseline asset set (bridge IDs, locations, key attributes such as age, type, deck area, ADT, detour length, route importance, etc.).

Step 2 (Definition and selection of the intensity measure). Define the seismic intensity measure (IM), i.e., the scalar ground-motion parameter used to represent earthquake loading at bridge sites. The intensity measure (IM) is selected from the set $\{\text{PGA}, S_a(T)\}$. PGA is the maximum ground acceleration at a site during an earthquake; it describes how hard the ground shakes at its strongest instant. $S_a(T)$ is a structure-related acceleration measure: it is the peak acceleration response of an idealized structure (a linear oscillator) with natural period T subjected to the same ground motion. Informally, it describes how strong the shaking is for a structure whose natural period is T. A structure has a natural period T, which is the time it takes to complete one full cycle of free vibration after being disturbed. Equivalently, the corresponding natural frequency is $f = 1/T$. Thus, "a structure with period T" means a structure whose dominant vibration cycle lasts T seconds. For example, $T = 1.0\,\text{s}$ means the structure tends to oscillate with a cycle of about one second. In spectral acceleration notation, $S_a(T)$ is the peak response of such a period-T oscillator to the ground motion. The natural period T describes how fast a structure tends to vibrate. A small T means the structure is stiff and vibrates quickly, so earthquake shaking tends to create larger accelerations and forces. A large T means the structure is more flexible and vibrates slowly, so earthquake shaking tends to create larger movements (displacements).

Step 3 (Site-specific hazard curve for the selected IM). For each bridge site i, obtain (or generate) the seismic hazard curve for the selected intensity measure (IM), defined as the annual exceedance-rate function

$$\lambda_{\text{ex},i}(\text{im}) = \nu(\text{IM}_i \geq \text{im})$$

over a suitable range of intensity levels im, where:

- IM_i is the random ground-motion intensity measure at the location of bridge i (e.g., $\text{IM}_i = \text{PGA}_i$ or $\text{IM}_i = S_{a,i}(T)$, depending on Step 2);
- im is a deterministic intensity threshold (a numerical level of the chosen IM);
- $\lambda_{\text{ex},i}(\text{im})$ is the annual exceedance rate (units: yr^{-1}) of the event $\{\text{IM}_i \geq \text{im}\}$ at bridge site i;
- $\nu(\cdot)$ denotes the annual occurrence rate of the event in parentheses.

This curve may be obtained from probabilistic seismic hazard analysis (PSHA) products (e.g., site-specific USGS hazard curves) or generated from an internal PSHA model. Figure A.9 illustrates a typical hazard curve for IM = PGA (left panel), which reports the annual rate of exceeding a PGA level. For subsequent risk integration/discretization, the absolute slope $\left|\frac{d}{d\text{im}}\lambda_{\text{ex},i}(\text{im})\right|$ (right panel) can be interpreted as a nonnegative "weight function" indicating which intensity ranges contribute most strongly to the total annual risk.

Step 4 (Discretize the hazard curve into representative intensity scenarios). Select a finite set of M intensity levels $\{\text{im}_j\}_{j=1}^{M}$ to represent the continuous hazard curve $\lambda_{\text{ex},i}(\text{im})$ in a tabulated form for subsequent risk calculation. Each im_j defines a "hazard scenario" corresponding to ground motions of approximately that intensity. A common implementation is to choose im_j associated with prescribed return periods RP_j (or exceedance probabilities), i.e., determine im_j from the site hazard curve such that

$$\lambda_{\text{ex},i}(\text{im}_j) \approx \frac{1}{RP_j}$$

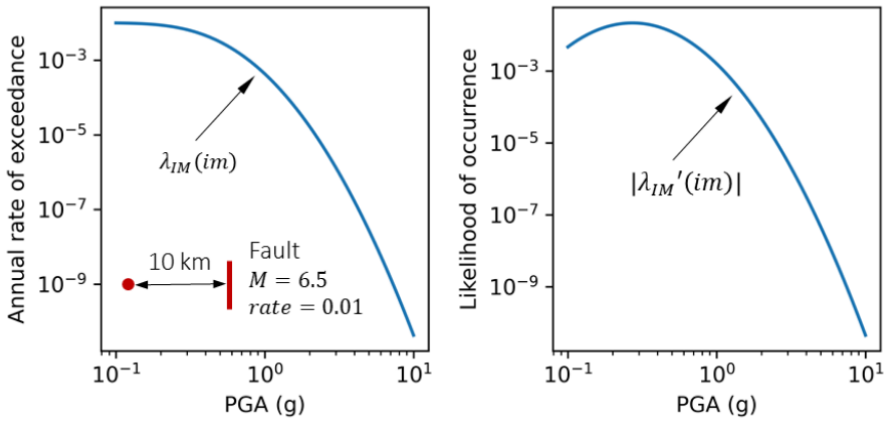


Figure A.9: Example hazard curve $\lambda_{\text{ex}}(\text{im})$ (left) and the absolute value of its gradient $|\lambda'_{\text{ex}}(\text{im})|$ (right) for IM = PGA. Source: [14, Figure 2.3]

for $j = 1, \ldots, M$. Assuming exceedances follow a Poisson process, for a return period RP (years),

$$p_T = \Pr(\text{exceed in } T \text{ years}) = 1 - \exp\left(-\frac{T}{RP}\right), \qquad p_{1\text{yr}} = 1 - \exp\left(-\frac{1}{RP}\right)$$

Return period RP (yr)	p_{50} (exceed in 50 yr)	$p_{1\text{yr}}$ (exceed in 1 yr)
50	63.21%	1.980%
75	48.66%	1.324%
100	39.35%	0.995%
225	19.93%	0.443%
475	9.99%	0.210%
975	5.00%	0.103%
2475	2.00%	0.0404%

Table A.62: Commonly used return periods and their corresponding exceedance probabilities under a Poisson model.

Step 5. Convert the hazard curve to scenario likelihood weights. One convenient discrete approximation is to define incremental annual rates over intensity bins:

$$w_j \;=\; \lambda_{\text{ex}}(\text{im}_j) - \lambda_{\text{ex}}(\text{im}_{j+1}), \quad j = 1, \ldots, M-1 \tag{A.203}$$

with $\text{im}_{j+1} > \text{im}_j$, and for the last bin take

$$w_M \;=\; \lambda_{\text{ex}}(\text{im}_M) \tag{A.204}$$

Step 6 (Consequence model specification). Specify the consequence variable C and the conditional expected-consequence function

$$\bar{C}(\text{im}) \;=\; \mathbb{E}[C \,|\, \text{IM} = \text{im}]$$

to be used for screening and ranking, where:

- C is the consequence/impact metric associated with a seismic event at a bridge (units defined by the

analyst; e.g., USD, days, delay-hours, or a dimensionless index);

- $\bar{C}(\text{im})$ is the conditional mean consequence when the site shaking intensity equals the threshold level im;
- $\mathbb{E}[\cdot]$ denotes expectation with respect to the uncertainty in damage/impact given IM $=$ im;
- IM is the selected intensity measure from Step 2 (e.g., PGA or $S_a(T)$);
- im is a deterministic intensity level (a numerical value of the selected IM).

Typical specifications of C include:

- **Agency consequence** C^A: expected repair/recovery cost (USD) or repair cost ratio (dimensionless) at intensity im. In this case, $\bar{C}(\text{im})$ may be denoted $\bar{C}^A(\text{im})$.
- **User consequence** C^U: expected service disruption at intensity im, such as closure duration (days) or user delay cost (USD). In this case, $\bar{C}(\text{im})$ may be denoted $\bar{C}^U(\text{im})$.
- **Composite consequence** C^{comp}: a weighted combination of multiple consequence components, e.g.,

$$C^{\text{comp}} = \alpha\, C^A + \beta\, C^U$$

with nonnegative weights α, β prescribed by the decision objective; correspondingly, $\bar{C}^{\text{comp}}(\text{im}) = \alpha\, \bar{C}^A(\text{im}) + \beta\, \bar{C}^U(\text{im})$.

Step 7 (Evaluate the bridge-specific expected consequence at each scenario intensity). For each bridge i and each hazard scenario j (defined by an intensity level im_j in Step 4), compute the conditional expected consequence

$$\bar{C}_{i,j} \;=\; \mathbb{E}[C_i \,|\, \text{IM}_i = \text{im}_j] \tag{A.205}$$

where C_i is the consequence metric selected in Step 6 (e.g., repair cost ratio, repair cost in USD, closure days, or user delay cost). This step produces a bridge-by-scenario consequence matrix $\{\bar{C}_{i,j}\}$ that is subsequently combined with scenario likelihood weights $\{w_j\}$ in Step 8 to compute risk.

In this report, $\bar{C}_{i,j}$ is computed by mapping damage states (DS) to consequences. Let $\text{DS} \in \{0, 1, \ldots, K\}$ denote discrete damage states, where $\text{DS} = 0$ denotes no damage and larger values denote more severe damage. The expected consequence at intensity im_j is obtained by

$$\bar{C}_{i,j} = \sum_{k=0}^{K} c_k \;\Pr(\text{DS} = k \,|\, \text{IM}_i = \text{im}_j) \tag{A.206}$$

where c_k is the consequence assigned to damage state k.

Fragility curves provide exceedance probabilities of the form

$$F_k(\text{im}) \;=\; \Pr(\text{DS} \geq k \,|\, \text{IM} = \text{im})\,, \qquad k = 1, \ldots, K \tag{A.207}$$

Figure A.10 shows typical fragility curves versus IM $=$ PGA for four thresholds (Slight, Moderate, Extensive, Complete). Each curve is a monotone increasing function of PGA and represents the probability that damage reaches at least the corresponding level.

The damage-state probabilities $\Pr(\text{DS} = k \mid \text{IM} = \text{im})$ are calculated by differencing adjacent exceedance

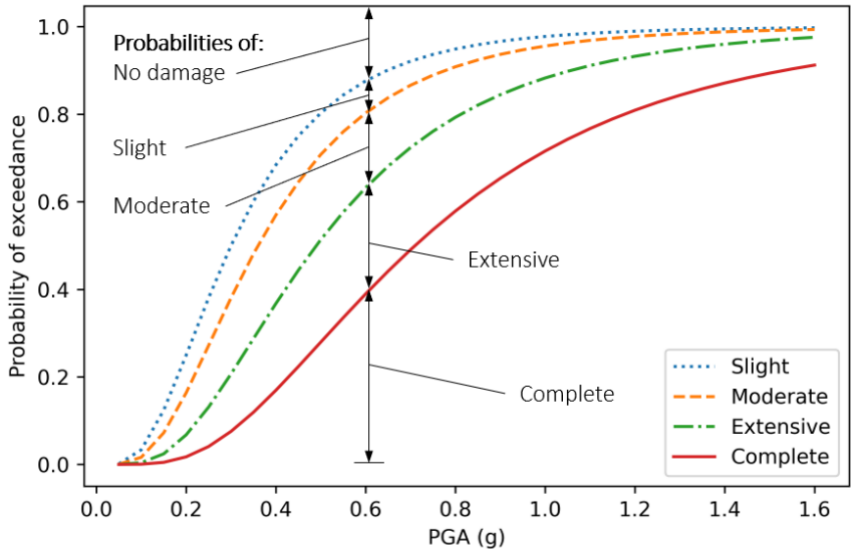


Figure A.10: Fragility curves and damage state probabilities
Source: [14, Figure 2.4]

probabilities $\{F_k(\text{im})\}$ from Equation A.207:

$$\Pr(\text{DS} = 0 \mid \text{IM} = \text{im}) = 1 - F_1(\text{im}), \tag{A.208}$$

$$\Pr(\text{DS} = k \mid \text{IM} = \text{im}) = F_k(\text{im}) - F_{k+1}(\text{im}), \qquad k = 1, \ldots, K-1, \tag{A.209}$$

$$\Pr(\text{DS} = K \mid \text{IM} = \text{im}) = F_K(\text{im}) \tag{A.210}$$

This is exactly what Figure A.10 illustrates: at a fixed PGA level, the vertical differences between the exceedance curves yield the probabilities of being in each discrete damage state (including the probability of no damage).

In this report, the consequence is defined as a repair cost ratio (dimensionless), i.e., repair cost divided by replacement cost. Accordingly, c_k is taken from Table A.63 for the four damaged states, and $c_0 = 0$ for no damage:

$$c_0 = 0, \quad c_{\text{Slight}} = 0.12, \quad c_{\text{Moderate}} = 0.19, \quad c_{\text{Extensive}} = 0.48, \quad c_{\text{Complete}} = 1.00$$

Thus, $\bar{C}_{i,j}$ computed from Equation A.206 is an expected repair cost ratio at intensity im_j.

Table A.63: Average cost ratios for different damage states

Damage state	Slight	Moderate	Extensive	Complete
Cost ratio c_k	0.12	0.19	0.48	1.00

Source: [14, Table 2.3]

For each scenario intensity level im_j:

1. Evaluate the exceedance fragilities $F_k(\text{im}_j) = \Pr(\text{DS} \geq k \mid \text{IM} = \text{im}_j)$ for all thresholds k.
2. Convert $\{F_k(\text{im}_j)\}$ to state probabilities $\Pr(\text{DS} = k \mid \text{IM} = \text{im}_j)$ using Equation A.208–Equation A.210.

3. Compute $\bar{C}_{i,j}$ by the weighted sum in Equation A.206 using the damage-state consequences c_k from Table A.63.

Step 8. Compute the bridge-level expected annual seismic risk score by summing likelihood-weighted consequences:

$$R_i = \sum_{j=1}^{M} w_j \bar{C}_{i,j} \tag{A.211}$$

Step 9. Normalize to support comparison across bridges and decision contexts. Examples:

$$R_i^{\text{ratio}} = \frac{R_i}{V_i}, \quad S_i = 100 \cdot \frac{R_i - \min_k R_k}{\max_k R_k - \min_k R_k} \tag{A.212}$$

where V_i may be replacement value (for monetary consequences), and S_i is a 0–100 score.

Step 10. Rank bridges in descending order of R_i (or S_i) to identify the highest-risk subset for detailed assessment, retrofit planning, or corridor-level programming.

A.11.2.2 Washington State Highway Bridge Seismic Screening Tool [15]

The BSST is a scenario-based screening methodology (not a probabilistic annualized risk model) developed to evaluate bridge damage states and approximate reopening times for Washington State highway bridges under a full-length M9.0 Cascadia Subduction Zone (CSZ) rupture scenario. It evaluates (i) direct seismic overloading relative to an approximate seismic design capacity, (ii) liquefaction-triggered damage, and (iii) tsunami-related damage (overtopping and scour), and then maps the projected damage outcomes to reopening times (Table A.64).

Table A.64: Bridge Reopening Times and Repair Types Based on Damage and Bridge Length

Damage Level	Damage Type	Consideration	Bridge Length (ft)	Reopening Time	Repair Type
None	None	None	N/A	0 days	None
Moderate	Minor or None	None	N/A	2 weeks	Bridge inspection and minor or no repairs
Significant	Any significant damage type	Bridge not over waterway or impassable topography	> 50	2 weeks per 50 ft of bridge length	Temporary road
			≤ 50	2 weeks	
	Significant damage without soil liquefaction	Bridge over waterway or impassable topography	> 150	2 years	Major bridge rehabilitation or replacement
			$\leq 150, > 50$	14 months	
			≤ 50	7 months	
	Significant damage with soil liquefaction	Bridge over waterway or impassable topography	> 150	2.5 years	Major bridge rehabilitation or replacement and subsurface strengthening
			$\leq 150, > 50$	1.5 years	
			≤ 50	8 months	

Source: [15, Table 5]

Step-by-step procedure **Step 1**. Assemble bridge inventory and attribute datasets as the baseline asset set. Use the bridge asset datasets provided by WSDOT (summarized in Table A.65), including: (i) bridge location (point/line GIS), (ii) bridge inventory attributes (e.g., year built, design indicators, length), (iii) seismic retrofit information (e.g., superstructure retrofit completion year, design PGA when available), and (iv) relevant bridge condition/auxiliary codes needed by the screening logic (e.g., waterway adequacy code, scour critical flag).

Table A.65: Bridge Data Provided by WSDOT

Dataset Name	Dataset Type	Description
Bridge Lines	Geographic Information System (GIS) shapefile	Polyline GIS data developed by WSDOT for some bridges in Washington State using linear referencing system (LRS) codes
BMS Elements	Spreadsheet	Bridge data is from WSDOT's Bridge Management Software database identifying the existence and quantity of various bridge systems and structural components
Inventory Data	Spreadsheet	Bridge data is from WSDOT's bridge inventory database containing various bridge design parameters and location information
Seismic Data	Spreadsheet	Bridge data created and maintained by WSDOT throughout the bridge seismic retrofit program containing information on the types and dates of seismic retrofits conducted on bridge superstructures and substructures, the expected 1,000-year earthquake PGA value for several bridges, information about the seismic retrofits needed/types based on WSDOT assessment

Source: [15, Table 1]

Step 2. Assemble hazard layers for the CSZ scenario and secondary hazards. Collect the following spatial layers used by the methodology: (i) projected CSZ scenario peak ground acceleration (PGA) field (Figure A.11), (ii) tsunami inundation extents Figure A.12; 1A Scenario inundation polygons), (iii) liquefaction susceptibility information from Washington State DNR (Figure A.13).

Step 3. Classify bridges as standard or special. The BSST first flags each bridge as either a standard bridge (screened with simplified capacity-vs-demand rules) or a special bridge requiring detailed analysis outside the BSST capacity check. Five bridge types are classified as special bridges using codes/fields described in Table A.66 (e.g., floating, movable, cable-stayed, suspension, steel tied-arch). In addition, bridges with hollow-core concrete piles are treated as special bridges and assumed to suffer significant damage regardless of PGA. For special bridges, the reopening time is treated as an input; a default reopening time of 2.5 years was assumed in the BSST (subject to later expert adjustment).

Step 4. Define the effective bridge design year used for capacity assignment. For each bridge i, define its effective design year as the more recent of: (1) the construction year and (2) the superstructure seismic retrofit completion year (Table A.67):

$$Y_i^{\text{des}} = \max\left(Y_i^{\text{built}},\, Y_i^{\text{retro,super}}\right) \tag{A.213}$$

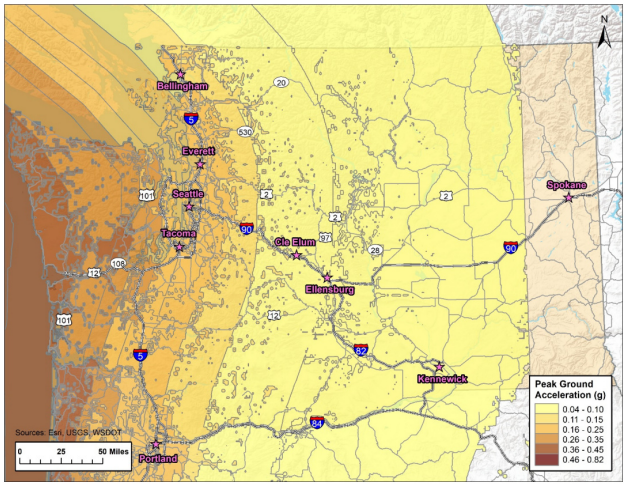


Figure A.11: Projected Peak Ground Acceleration (PGA) for Washington State under the USGS M9.0 CSZ Scenario
Source: [15, Figure 1]

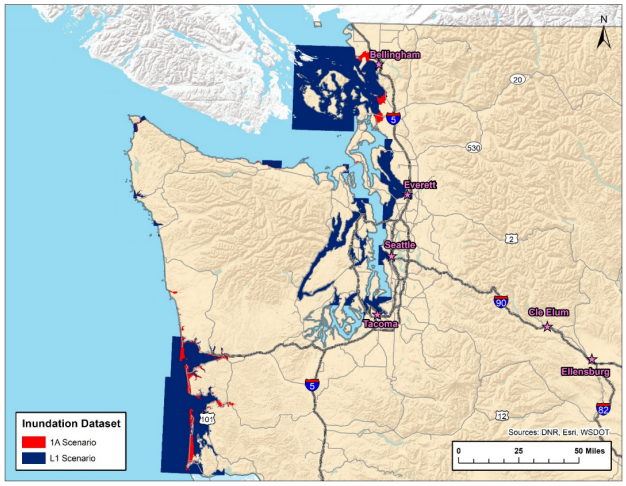


Figure A.12: Washington State DNR Tsunami Inundation Datasets Coverage Comparison-the 1A Scenario was used in this Bridge Analysis

Source: [15, Figure 2]

Table A.66: Bridge Types Classified as Special Bridges, and Sources Used for Identifying Each Bridge Type

Bridge Type	Number of Bridges	"alphabetic_span" Values [from the Inventory Data dataset provided by WSDOT]	"fed_main_design_code" Values [from the Inventory Data dataset provided by WSDOT]	"element_id" Values [from the BMS Elements dataset provided by WSDOT]
Floating	8	• CFP ("Concrete Floating Pontoon") • SFP ("Steel Floating Pontoon")		• 236 ("Concrete Floating Pontoon") • 238 ("Floating Bridge-Anchor Cable")
Movable	13	• BAS ("Bascule Lift Span") • SLS ("Steel Lift Span") • SSwS ("Steel Swing Span")	• 15 ("Movable-Lift") • 16 ("Movable-Bascule") • 17 ("Movable Swing")	• 367 ("Movable Bridge") • 501 ("Movable Bridge Steel Tower")
Cable Stayed	1	• CSS ("Cable Stayed Span")		• 149 ("Cable Stayed Bridge-Cable")
Steel Suspension	3	• SSuS ("Steel Suspension Span")		• 146 ("Suspension-Main Cable") • 147 ("Suspension-Suspender Cable")
Steel Tied Arch	10	• STA ("Steel Tied Arch")		• 143 ("Steel Suspender")

Source: [15, Table 2]

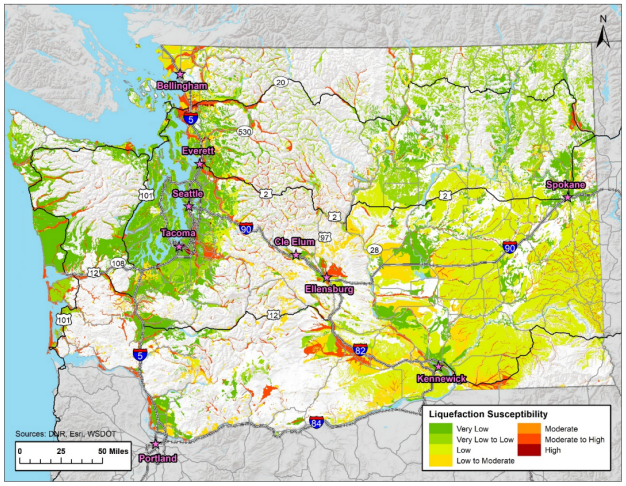


Figure A.13: Soil Liquefaction Susceptibility in Washington State
Source: [15, Figure 3]

Table A.67: Determination of Bridge Seismic Design Capacity Based on Bridge Design Year

Bridge Design Year	Document/Methodology for Determining Bridge Seismic Design Capacity
1983 or earlier	Based on historical observation of bridge performance during earthquakes; seismic design capacity of 0.15 g used for all bridges.
1984–1998	Digitized Figure 3 from the 1983 AASHTO *Guide Specifications for Seismic Design of Highway Bridges* (AASHTO 1983) in ArcGIS to determine PGA design values for each bridge in the WSDOT-provided Inventory Dataset (WSDOT 2017c).
1999–2007	Digitized Figure 3.10.2-1 from the 1998 *AASHTO LRFD Bridge Design Specifications* (AASHTO 1998, 2004) in ArcGIS to determine PGA design values (WSDOT 2017c).
2008–2014	Capacities provided by WSDOT in "1000 YR PGA" column (WSDOT 2017d). For 148 bridges missing values, 1,000-year PGA design values were extracted from U.S. Seismic Design Maps using 2009 AASHTO Guide Specs.
2015 or later	Capacities provided by WSDOT in "2014 PGA" column (WSDOT 2017d). For 44 bridges missing values, 1,000-year PGA design values were extracted from U.S. Seismic Design Maps using 2009 AASHTO Guide Specs.

Source: [15, Table 3]

Step 5. Determine the bridge seismic design capacity (in terms of PGA). For each standard bridge i, assign an approximate seismic design capacity C_i (units of g) based on Y_i^{des} and the location-dependent design PGA implied by the governing design guidance. Conceptually,

$$C_i = \begin{cases} 0.15g, & Y_i^{\text{des}} \le 1983, \\ \text{PGA}_{\text{AASHTO}}^{1983}(\text{loc}_i), & 1984 \le Y_i^{\text{des}} \le 1998, \\ \text{PGA}_{\text{AASHTO}}^{1998/2004}(\text{loc}_i), & 1999 \le Y_i^{\text{des}} \le 2007, \\ C_i^{\text{WSDOT}}, & 2008 \le Y_i^{\text{des}} \le 2014, \\ C_i^{\text{WSDOT}} \text{ or } C_i^{\text{USGSmap}}, & Y_i^{\text{des}} \ge 2015. \end{cases} \tag{A.214}$$

Here $\text{PGA}_{\text{AASHTO}}(\text{loc}_i)$ denotes the digitized contour-map PGA value assigned by spatial lookup at the

bridge location, with conservative assignment when the bridge falls between contours.

Step 6. Extract scenario PGA demand at each bridge site. For each bridge i, obtain the scenario PGA at the bridge site from the CSZ M9.0 hazard field:

$$D_i = \text{PGA}^{\text{CSZ}}(\text{loc}_i) \tag{A.215}$$

Step 7. Apply the earthquake-duration adjustment to obtain an effective (modified) PGA demand. The BSST accounts for long-duration shaking (3-6 minutes for the full-length CSZ rupture) by increasing projected PGA demand by 33%, which corresponds to an approximate 25% reduction in capacity, but only when shaking is strong enough to induce inelastic deformation. A PGA of $0.15g$ is used as the elastic/inelastic threshold. Define the effective PGA demand $\widetilde{D}_i$ as:

$$\widetilde{D}_i = \begin{cases} 1.33\, D_i, & D_i \geq 0.15g, \\ D_i, & D_i < 0.15g. \end{cases} \tag{A.216}$$

Step 8. Evaluate direct seismic overloading damage (capacity vs. modified demand). For each standard bridge i, compare the modified demand $\widetilde{D}_i$ to the design capacity C_i.

Define indicator functions:

$$\mathbb{I}_i^{\text{over}} = \mathbb{I}\left(\widetilde{D}_i > C_i\right), \qquad \mathbb{I}_i^{0.15} = \mathbb{I}\left(\widetilde{D}_i > 0.15g\right) \tag{A.217}$$

The BSST assigns damage levels using rule-based logic described in the report:

- If $\mathbb{I}_i^{\text{over}} = 1$, the bridge is assumed to sustain significant damage due to seismic overloading.
- Exception: bridges with single-span configurations (no columns) or with pier-wall supports are treated as more resilient; if such a bridge satisfies $\mathbb{I}_i^{\text{over}} = 1$, it is classified as moderately damaged due to overloading (and then still assessed for liquefaction/tsunami).
- If $\mathbb{I}_i^{\text{over}} = 0$ but $\mathbb{I}_i^{0.15} = 1$, the bridge is classified as moderately damaged (shaking exceeds the minimal damage threshold but does not exceed capacity).
- If $\mathbb{I}_i^{0.15} = 0$, the bridge is classified as having no direct seismic overloading damage.

Step 9. Evaluate liquefaction-triggered damage using PGA thresholds by soil category. Using DNR liquefaction susceptibility categories, the BSST maps each bridge site to a liquefaction category and assigns a liquefaction PGA threshold T_i^{liq} according to Table 4 in the report (simplified categories):

$$T_i^{\text{liq}} \in \{\text{N/A (not vulnerable)},\ 0.15g,\ 0.10g\} \tag{A.218}$$

Define the liquefaction occurrence indicator as:

$$\mathbb{I}_i^{\text{liq}} = \begin{cases} 1, & \widetilde{D}_i \geq T_i^{\text{liq}}, \\ 0, & \widetilde{D}_i < T_i^{\text{liq}}, \end{cases} \tag{A.219}$$

with the convention that if the site is classified as not vulnerable (N/A), then $\mathbb{I}_i^{\text{liq}} = 0$.

If $\mathbb{I}_i^{\text{liq}} = 1$, the bridge is assumed to sustain significant damage due to liquefaction at the bridge location.

Step 10. Evaluate tsunami-triggered damage (overtopping proxy and scour-critical rule). First determine whether bridge i lies inside any mapped tsunami inundation zone:

$$\mathbb{I}_i^{\text{tsu,zone}} = \mathbb{I}\left(\text{loc}_i \in \Omega^{\text{tsu}}\right) \tag{A.220}$$

where Ω^{tsu} denotes the union of available tsunami inundation polygons.

Because inundation depth was not available in the dataset used, overtopping is proxied using the bridge waterway adequacy code. Define:

$$\mathbb{I}_i^{\text{tsu,over}} = \mathbb{I}_i^{\text{tsu,zone}} \cdot \mathbb{I}\left(\text{WaterwayCode}_i \in \mathcal{W}_{\text{over}}\right) \tag{A.221}$$

where $\mathcal{W}_{\text{over}}$ denotes the set of waterway adequacy codes indicating "occasional overtopping of bridge deck and roadway approaches with significant traffic delays" or more severe/frequent overtopping, as described in the report. If $\mathbb{I}_i^{\text{tsu,over}} = 1$, the bridge is assumed to sustain significant damage due to tsunami overtopping.

Next assess scour potential using the scour-critical flag:

$$\mathbb{I}_i^{\text{tsu,scour}} = \mathbb{I}_i^{\text{tsu,zone}} \cdot \mathbb{I}\left(\text{ScourCritical}_i = 1\right) \tag{A.222}$$

If $\mathbb{I}_i^{\text{tsu,scour}} = 1$, the bridge is assumed to sustain significant damage due to tsunami-related scour.

Step 11. Aggregate damage mechanisms into a single bridge damage level. Define the significant-damage indicator as:

$$\mathbb{I}_i^{\text{sig}} = \mathbb{I}_i^{\text{sig,seis}} \vee \mathbb{I}_i^{\text{liq}} \vee \mathbb{I}_i^{\text{tsu,over}} \vee \mathbb{I}_i^{\text{tsu,scour}} \tag{A.223}$$

where $\mathbb{I}_i^{\text{sig,seis}}$ is the significant damage flag from Step 8 after applying the pier-wall/single-span exception logic, and $\vee$ denotes logical OR.

Define the moderate-damage indicator as:

$$\mathbb{I}_i^{\text{mod}} = \left(\mathbb{I}_i^{\text{mod,seis}} = 1\right) \wedge \left(\mathbb{I}_i^{\text{sig}} = 0\right) \tag{A.224}$$

where $\mathbb{I}_i^{\text{mod,seis}}$ is the moderate damage flag from Step 8.

Then the overall damage level is:

$$\text{DamageLevel}_i = \begin{cases} \text{Significant}, & \mathbb{I}_i^{\text{sig}} = 1, \\ \text{Moderate}, & \mathbb{I}_i^{\text{mod}} = 1, \\ \text{None}, & \text{otherwise}. \end{cases} \tag{A.225}$$

Special bridges are handled separately (Step 3) and treated as significant damage by assumption in the BSST.

Step 12. Determine reopening time and repair type using rule-based mappings. The BSST converts damage level and damage-type considerations to an approximate reopening time based on Table A.64, with additional logic to determine whether a temporary bypass road is feasible.

Step 12a. Determine whether a temporary road is feasible for significantly damaged bridges. A temporary bypass road is considered for bridges over passable ground and outside waterway crossings. The waterway adequacy code is used as a proxy for whether the bridge crosses a waterway. Topographic feasibility is based

on elevation variation beneath the bridge computed from a DEM:

$$\Delta z_i = z_{i,\max} - z_{i,\min}, \qquad \mathbb{I}_i^{\text{imp}} = \mathbb{I}\left(\Delta z_i > 30 \text{ ft}\right) \tag{A.226}$$

If $\Delta z_i \leq 30$ ft, temporary road construction is considered feasible (subject to the waterway condition).

Step 12b. Assign reopening time T_i. Let L_i denote bridge length (ft). Define the temporary-road construction time as:

$$T_i^{\text{temp}} = 2 \text{ weeks} \cdot \max\left(1, \left\lceil \frac{L_i}{50 \text{ ft}} \right\rceil\right) \tag{A.227}$$

For standard bridges, the reopening time is assigned as:

$$T_i = \begin{cases} 0, & \text{DamageLevel}_i = \text{None}, \\ 2 \text{ weeks}, & \text{DamageLevel}_i = \text{Moderate}, \\ T_i^{\text{temp}}, & \text{DamageLevel}_i = \text{Significant, and temporary road feasible}, \\ T_i^{\text{repl}}(L_i, \mathbb{I}_i^{\text{liq}}), & \text{DamageLevel}_i = \text{Significant, and temporary road not feasible}, \end{cases} \tag{A.228}$$

where $T_i^{\text{repl}}(L_i, \mathbb{I}_i^{\text{liq}})$ is taken from Table 5 of the report and depends on: (i) bridge length bin (≤ 50 ft, 50–150 ft, > 150 ft), (ii) whether significant damage includes soil liquefaction ($\mathbb{I}_i^{\text{liq}} = 1$).

Specifically, Table A.64 assigns:

$$T_i^{\text{repl}}(L_i, 0) = \begin{cases} 7 \text{ months}, & L_i \leq 50 \text{ ft}, \\ 14 \text{ months}, & 50 < L_i \leq 150 \text{ ft}, \\ 2 \text{ years}, & L_i > 150 \text{ ft}, \end{cases} \qquad T_i^{\text{repl}}(L_i, 1) = \begin{cases} 8 \text{ months}, & L_i \leq 50 \text{ ft}, \\ 1.5 \text{ years}, & 50 < L_i \leq 150 \text{ ft}, \\ 2.5 \text{ years}, & L_i > 150 \text{ ft}. \end{cases} \tag{A.229}$$

For special bridges, the BSST uses an input reopening time; a default value of 2.5 years is used in the report.

Step 12c. Assign repair type. Repair type is mapped in tandem with reopening time:

- None damage: no repair.
- Moderate damage: bridge inspection and minor or no repairs.
- Significant damage with feasible bypass: temporary road.
- Significant damage without feasible bypass: major bridge rehabilitation/replacement; if liquefaction occurs, include subsurface strengthening.
- Special bridges: treated as requiring special-bridge repair/replacement consistent with assumed reopening time.

A.12 Tornado and Strong Wind Risk

A.12.1 Introduction

Tornado and strong wind risk is the likelihood that a high-wind event will subject the bridge to lateral or uplift forces exceeding its structural capacity, multiplied by the expected consequences of such events (damage, collapse, closure, or casualties).

Table A.68: U.S. bridge damage incidents associated with high-wind events (last 5 years).

Date	Bridge	State	High-wind event	Reported damage (bridge)
2025-01-12	Rabbit Creek Pedestrian Bridge (over Seward Hwy)	AK	Hurricane-force windstorm in Anchorage	Partial collapse; roof/fencing/walkway sections fell onto/near highway; bridge closed for assessment [164, 165]
2024-09 (late)	Lake Lure Flowering Bridge	NC	Remnants of Hurricane Helene (winds + flooding)	One end collapsed; subsequent failures; engineers deemed bridge beyond repair; demolition scheduled Aug 18, 2025 [166, 167]
2023-03-21	Third Street Bridge (Lefty O'Doul Bridge)	CA	Windstorm: barges broke loose in high winds/surf and allided with bridge	"Substantial structural damage" including damaged steel beams/handrail/timber walkway and damage near operator's house; emergency repairs initiated [168, 169]
2023-01 (month)	Pfeiffer Falls Trail pedestrian bridge (Pfeiffer Big Sur SP)	CA	Winter storm (high-wind storm; tree fall cited as damage trigger)	Pedestrian bridge damaged/destroyed after a fallen redwood tree during storm; trail closed until reconstruction [170]
2022-09-28	Sanibel Causeway (Sanibel Island access)	FL	Hurricane Ian (Cat 4; extreme winds + surge)	Severe damage; sections of the causeway destroyed/collapsed, cutting off roadway access; temporary reopening Oct 19, 2022 [171, 172]

Representative Case Studies in Texas

- Multiple Bridge Infrastructure Damaged by 2008 Hurricane Ike (Gulf Coast Region) (2012)
  - Hurricane Ike brought sustained winds of about 110 mph and extensive storm surge along the Texas Gulf Coast, severely impacting transportation infrastructure near Houston and Galveston. A comprehensive study of Ike's effects found that at least 53 bridges in the Houston/Galveston region were damaged, especially smaller timber structures and low-clearance rural bridges. Damage mechanisms included wind-driven waves, wind forces, debris impact, and scour around support elements. Many bridges were completely destroyed or required major repair following the storm [173].
- World Trade Bridge (Laredo, Tx) Closed After Strong Storm Winds (2017)
  - A severe storm with high winds (downburst) caused structural damage to the World Trade Bridge - one of the region's busiest commercial crossings in Laredo, Tx. Strong straight-line winds damaged bridge facilities and infrastructure, prompting indefinite closure of the bridge to commercial traffic until a structural inspection supported reopening. Wind gusts and the storm's destructive path forced rerouting of freight traffic to other border crossings [174].
- Severe Storm Damage Around World Trade Bridge (Laredo, Tx) (2024)
  - A severe storm in mid-September 2024 brought damaging winds that affected the World Trade Bridge area, including damage to roof structures and facilities associated with the bridge and halted commercial processing temporarily [175].

A.12.2 Methods Reported in the Literature

A.12.2.1 FDOT Assessment Procedure [1]

The report estimates tornado risk at each bridge by combining a data-driven likelihood model with an expert-informed consequence/vulnerability model. Historical tornado touchdown points are spatially linked to the bridge inventory using a GIS overlay and a fixed buffer around each bridge (e.g., one mile). For each bridge i, the number of historical touchdowns N_i observed within the buffer over the observation period T (about 60 years) is converted to a mean annual occurrence rate $\lambda_i = \frac{N_i}{T}$. Assuming tornado touchdowns follow a Poisson process, the annual probability of at least one tornado occurrence near bridge i is $p_{\text{tor}}(i) = 1 - \exp(-\lambda_i)$.

Because Florida-specific bridge damage data from tornadoes are limited, consequences are represented using relative vulnerability scores (on an ordinal scale) assigned to (i) individual bridge elements (with non-structural and movable components such as signs and railings scored as highly vulnerable), and (ii) bridge type/geometry classes (with long-span, narrow, high-underclearance truss bridges identified as especially vulnerable). A unitless tornado risk index is then formed by multiplying the likelihood $p_{\text{tor}}(i)$ by appropriate vulnerability multipliers derived from these element- and type-based scores.

Step-by-step procedure **Step 1**. Obtain the bridge inventory (FDOT/Pontis bridge GIS layer) as the baseline asset set for risk computation.

Step 2. Obtain historical tornado data from the National Weather Service GIS Data Portal for 1950–2010, including GIS shapefiles and tabular database records.

Step 3. Perform a spatial overlay (spatial join) in ArcGIS between the tornado GIS layer and the FDOT bridge GIS layer.

Step 4. For each bridge, create a 1-mile buffer zone around its location (the report notes that the 1-mile distance is chosen arbitrarily). Identify tornado touch-down points within the 1-mile buffer of each bridge and assign those events to the corresponding bridge.

Step 5. For each bridge, count the number of tornado occurrences within the buffer during the observation period.

Step 6. Convert the event count to an annual occurrence rate λ (the Poisson intensity) by dividing by the observation duration (60 years for 1950–2010).[6]

Step 7. Assume tornadoes follow a Poisson process. Convert λ to the annual probability of at least one occurrence:

$$p_{\text{tornado}} = 1 - e^{-\lambda} \tag{A.230}$$

Step 8. Compute the tornado risk index by multiplying the annual probability by three vulnerability multipliers, and rank bridges in descending order of $\text{Risk}_{\text{tornado}}$:

[6] The report also presents an alternative approach to estimate λ using county-level tornado records (NOAA): compute a county-level λ and then use the same probability formula. In addition, because tornado records are sparse at the bridge scale, the report discusses smoothing/accumulating historical incidence by FDOT district to obtain a more reasonable forecast of future tornado probability for any particular bridge.

$$\text{Risk}_{\text{tornado}} = p_{\text{tornado}} \cdot V^{\text{elem}}_{\text{tornado}} \cdot V^{\text{age}}_{\text{tornado}} \cdot V^{\text{des}}_{\text{tornado}} \tag{A.231}$$

The calculations of $V^{\text{elem}}_{\text{tornado}}, V^{\text{age}}_{\text{tornado}}, V^{\text{des}}_{\text{tornado}}$ are shown below:

$V^{\text{elem}}_{\text{tornado}}$ is the tornado element vulnerability. For each bridge, let $\mathcal{E}$ denote the set of bridge elements recorded for that bridge. For each element e, obtain its tornado element score $s_{\text{tornado}}(e)$ from Table A.69, where $s_{\text{tornado}}(e) \in \{0, 1, 2, 3, 4, 5\}$. The bridge-level raw element score is the simple average:

$$\bar{s}_{\text{tornado}} = \frac{1}{|\mathcal{E}|} \sum_{e \in \mathcal{E}} s_{\text{tornado}}(e) \tag{A.232}$$

Table A.69: Levels of vulnerability of bridge elements to damage from tornadoes and strong winds

Table 3.2. Levels of vulnerability of bridge elements to damage from tornadoes and strong winds.

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	2	476	Timber Walls	3
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	1	477	Other Walls	2
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	2
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	1	480	Mast Arm Found	4
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	4
31	Timber Deck	2	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	4
32	Timber Deck/AC Ovly	2	234	R/Conc Cap	1	483	Other Mast Arm Vert	4
38	Bare Concrete Slab	1	235	Timber Cap	2	484	Paint Mast Arm Horzn	4
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	1	485	Galvan Mast Arm Horz	4
54	Timber Slab	2	241	Concrete Culvert	1	486	Other Mast Arm Horzn	4
55	Timber Slab/AC Ovly	2	242	Timber Culvert	3	487	Sign Member Horiz	4
98	Conc Deck on PC Pane	1	243	Misc Culvert	2	488	Sign Member Vertical	4
99	PS Conc Slab	1	290	Channel	3	489	Sign Foundation	4
101	Unpnt Stl Box Girder	1	298	Pile Jacket Bare	3	495	Uncoat High Mast L.	4
102	Paint Stl Box Girder	1	299	Pile Jacket/Cath Pro	3	496	Painted High Mast L.	4
104	P/S Conc Box Girder	1	300	Strip Seal Exp Joint	2	497	Galvan. High Mast L.	4
105	R/Conc Box Girder	1	301	Pourable Joint Seal	2	498	Other High Mast L.P.	4
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	2	499	H. M. L. P. Found.	4
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	2	540	Open Gearing	3
109	P/S Conc Open Girder	1	304	Open Expansion Joint	2	541	Speed Reducers	3
110	R/Conc Open Girder	1	310	Elastomeric Bearing	2	542	Shafts	3
111	Timber Open Girder	2	311	Moveable Bearing	2	543	Shaft Brgs and Coupl	3
112	Unpnt Stl Stringer	3	312	Enclosed Bearing	2	544	Brakes	3
113	Paint Stl Stringer	2	313	Fixed Bearing	2	545	Emergency Drive	3
115	P/S Conc Stringer	2	314	Pot Bearing	2	546	Span Drive Motors	3
116	R/Conc Stringer	2	315	Disk Bearing	2	547	Hydraulic Power Unit	3
117	Timber Stringer	3	320	P/S Conc Appr Slab	1	548	Hydraulic Piping Sys	3
120	U/Stl Thru Truss/Bot	4	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	3
121	P/Stl Thru Truss/Bot	4	330	Metal Rail Uncoated	4	550	Hopkins Frame	3
125	U/Stl Thru Truss/Top	4	331	Conc Bridge Railing	4	560	Locks	3
126	P/Stl Thru Truss/Top	4	332	Timb Bridge Railing	4	561	Live Load Shoes	3
130	Unpnt Stl Deck Truss	4	333	Other Bridge Railing	4	562	Counterweight Suppor	3
131	Paint Stl Deck Truss	4	334	Metal Rail Coated	4	563	Acc Ladd & Plat	3
135	Timber Truss/Arch	2	356	Steel Fatigue SmFlag		564	Counterweight	3
140	Unpnt Stl Arch	2	357	Pack Rust Smart Flag		565	Trun/Str and Cur Trk	3
141	Paint Stl Arch	2	358	Deck Cracking SmFlag		570	Transformers	3
143	P/S Conc Arch	1	359	Soffit Smart Flag		571	Submarine Cable	3
144	R/Conc Arch	1	360	Settlement SmFlag		572	Conduit & Junc. Box	3
145	Other Arch	1	361	Scour Smart Flag		573	PLCs	3
146	Misc Cable Uncoated	4	362	Traf Impact SmFlag		574	Control Console	3
147	Misc Cable Coated	4	363	Section Loss SmFlag		580	Navigational Lights	4
151	Unpnt Stl Floor Beam	3	369	Sub.Sect Loss SmFlag		581	Operator Facilities	4
152	Paint Stl Floor Beam	3	370	Alert Smart Flag		582	Lift Bridge Spec. Eq	3
154	P/S Conc Floor Beam	2	386	Fender/Dolphin Uncoa	2	583	Swing Bridge Spec. E	3
155	R/Conc Floor Beam	2	387	P/S Fender/Dolphin	2	590	Resistance Barriers	3
156	Timber Floor Beam	3	388	R/Conc Fender/Dolphi	2	591	Warning Gates	4
160	Unpnt Stl Pin/Hanger	3	389	Timber Fender/Dolphi	2	592	Traffic Signals	4
161	Paint Stl Pin/Hanger	3	390	Other Fender/Dolphin	2			
201	Unpnt Stl Column	3	393	Blkhd Sewl Metal Unc	2			
202	Paint Stl Column	3	394	R/Conc Abut Slope Pr	2			
204	P/S Conc Column	2	395	Timber Abut Slope Pr	3			
205	R/Conc Column	2	396	Other Abut Slope Pro	2			
206	Timber Column	3	397	Drain. Syst Metal	2			
207	P/S Conc Holl Pile	2	398	Drain. Syst Other	2			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	2			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	2			
215	R/Conc Abutment	1	475	R/Conc Walls	2			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Source: [1, Table 3.2]

The reported element vulnerability multiplier is obtained by rescaling the average score:

$$V_{\text{tornado}}^{\text{elem}} = \frac{\bar{s}_{\text{tornado}} + 1}{3} \tag{A.233}$$

$V_{\text{tornado}}^{\text{age}}$ is the tornado age vulnerability. Let Y be the year the bridge was built. The tornado age vulnerability multiplier is modeled as a linear function of bridge age:

$$V_{\text{tornado}}^{\text{age}} = 1 + \frac{1990 - Y}{35} \tag{A.234}$$

$V_{\text{tornado}}^{\text{des}}$ is the tornado design vulnerability. For a given bridge, identify its superstructure design type (as coded in the bridge inventory) and its vertical underclearance class (the report grades consequences by design type and by range of underclearance). The report assigns a tornado design score $s_{\text{tornado}}^{\text{des}}$ using Table A.70, which provides a discrete rating (0–5) for each (design type, underclearance range) combination. Then

$$V_{\text{tornado}}^{\text{des}} = \frac{s_{\text{tornado}}^{\text{des}} + 1}{3} \tag{A.235}$$

Table A.70: Levels of vulnerability of bridge types to damage from tornadoes and strong winds [1]

Table 3.3. Levels of vulnerability of bridge types to damage from tornadoes and strong winds.

Superstructure design - main unit	Low underclearance	High underclearance
Slab	1	2
Stringer/MultiBeam/Girder	1	2
Girder & Floorbeam	1	2
Tee Beam	1	2
Box Beam or Girders - Multiple	1	2
Box Beam/Girders - Single or Spread	1	2
Frame (except frame culverts)	1	2
Orthotropic	1	2
Truss - Deck	4	5
Truss - Thru	4	5
Arch - Deck	1	2
Arch - Thru	1	2
Suspension	4	4
Stayed Girder	4	4
Movable - Lift	3	4
Movable - Bascule	3	4
Movable - Swing	3	4
Tunnel	2	4
Culvert	2	3
Mixed Types	2	2
Segmental Box Girder	1	2
Channel Beam	1	2

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Source: [1, Table 3.3]

Agency cost. First, compute the replacement cost for bridge i:

$$R_i = A_i c \tag{A.236}$$

where A_i is deck area and c is the unit replacement cost per area. Define the agency weight as the normalized share over the bridge inventory set $\mathcal{E}$:

$$w_i = \frac{R_i \cdot \text{Risk}_{\text{tornado}}(i)}{\sum_{j \in \mathcal{E}} R_j \cdot \text{Risk}_{\text{tornado}}(j)} \tag{A.237}$$

Given the statewide average annual agency loss from tornadoes L_{tornado} (annual historical agency cost in Table A.15), allocate it using w_i:

$$\text{AgencyCost}_{\text{tornado}}(i) = L_{\text{tornado}}\, w_i \tag{A.238}$$

User cost. For bridge i, define the daily detour cost:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}} \frac{\ell_i}{v_i} \right) \tag{A.239}$$

where ADT_i is average daily traffic, ℓ_i is detour length, v_i is detour speed, c_{op} is unit operating cost per mile, and c_{time} is value of time per hour.

Define the user weight as the normalized share over the bridge inventory:

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{tornado}}(i)}{\sum_{j \in \mathcal{E}} U_j \cdot \text{Risk}_{\text{tornado}}(j)} \tag{A.240}$$

Given the statewide annual control total for tornado user losses L^U_{tornado} (annual historical user cost in Table A.15), allocate it using u_i:

$$\text{UserCost}_{\text{tornado}}(i) = L^U_{\text{tornado}}\, u_i \tag{A.241}$$

Total cost. After computing $\text{AgencyCost}_{\text{tornado}}(i)$ and $\text{UserCost}_{\text{tornado}}(i)$ for each bridge i, form the total annual expected impact (USD/year) by summing the two components:

$$\text{TotalCost}_{\text{tornado}}(i) = \text{AgencyCost}_{\text{tornado}}(i) + \text{UserCost}_{\text{tornado}}(i) \tag{A.242}$$

A.12.2.2 NCHRP 20-07 Task 378 [2]

In this report, the likelihood of a hazard scenario h affecting a bridge b is modeled as the product of (i) an extreme-event likelihood term and (ii) a service-disruption likelihood term that is conditional on the occurrence of the hazard scenario:

$$L_{bh} \;=\; L^E_{bh}\, L^D_{bh} \tag{A.243}$$

where L^E_{bh} is the extreme event likelihood and L^D_{bh} is the likelihood of service disruption conditional on the occurrence of the extreme event.

For high winds, **the Guidelines note that agencies more commonly plan mitigation actions on specific structures with unusual wind response behavior** (rather than implementing inventory-wide risk programs), and the methods can support benefit/cost prioritization of such mitigation actions.

The procedure below follows the report's high-wind worksheet, and the report states that estimation for high winds uses the same general approach as storm surge (polling plus supporting records and GIS exposure/probability inputs).

Step-by-step procedure **Step 1**. Define the high-wind hazard scenario h and the analysis unit.

- Select a hazard scenario such as High wind-100 (a specified high-wind event magnitude/recurrence) and define the disruption threshold (e.g., damaged bridge, closure, or delay) consistent with the report's hazard-scenario concept.
- Select the organizational/geographic unit used for polling and scaling (the worksheet example uses District).

Step 2. Estimate the extreme-event likelihood L_{bh}^{E} (wind event probability) using geographic analysis.

- Develop or obtain geographic zones/polygons with uniform probabilities of damaging wind events for the selected hazard scenario; the report notes this can be helpful in the analysis.
- For each bridge (or for the district-level approximation used in the worksheet), obtain the annual wind-event probability from GIS analysis and enter it as the wind event probability:

$$p_{\text{wind}}^{E} \equiv L_{bh}^{E} \tag{A.244}$$

- If the hazard scenario is defined by a return period R (e.g., a 100-year event), the annual exceedance probability is

$$p_{\text{wind}}^{E} = \frac{1}{R} \tag{A.245}$$

  (The Guidelines illustrate this concept for a 100-year event as a one-percent annual probability.)

Step 3. Collect historical incident information via polling (and supporting records).

- Using NCHRP 20-07 (378) Risk Analysis, Sheet LD - Wind, list as many relevant incidents as possible when "tornadoes or other high winds" damaged a bridge or forced a bridge closure or delay.

Step 4. Define the recollection period and adjust for incomplete recall.

- Let T_{recall} be the number of years included in the respondent's recollection period.
- Let N_{add} be the number of additional incidents believed to have occurred during this period.

Step 5. Compute the total number of bridges affected during the recall window.

- Let N_{listed} be the number of distinct affected bridges listed in the incident table on Sheet LD - Wind.
- Compute the estimated total number of bridges affected over the recall period:

$$N_{\text{aff}} \;=\; N_{\text{listed}} + N_{\text{add}} \tag{A.246}$$

Step 6. Determine the exposed bridge population used for scaling.

- Let N_{dist} be the number of bridges in the district.
- (Storm-surge guidance adopted by the report for high winds) GIS-based geographic analysis may be used to identify the total number of bridges exposed to the extreme event for scaling purposes; in the

storm-surge example, FEMA data were used for this purpose.

Step 7. Estimate the conditional likelihood of service disruption for high winds, L_{bh}^{D}.

- Using Sheet LD - Wind, compute the likelihood of service disruption as indicated by the worksheet:

$$L_{\text{wind}}^{D} \;=\; \frac{N_{\text{aff}}}{N_{\text{dist}}\; T_{\text{recall}}\; p_{\text{wind}}^{E}} \;=\; \frac{\text{Line 13}}{\text{Line 14} \cdot \text{Line 11} \cdot \text{Line 15}} \tag{A.247}$$

  The worksheet defines this quantity as the likelihood of service disruption conditional on occurrence of the wind-hazard scenario.

Step 8. Combine with the extreme-event likelihood (if needed) to obtain the total likelihood for use in risk computations.

- If the analysis requires the total (unconditional) likelihood of disruption for bridge b under scenario h, compute

$$L_{bh} = L_{bh}^{E}\, L_{bh}^{D} \quad \text{with} \quad L_{bh}^{E} = p_{\text{wind}}^{E} \tag{A.248}$$

A.13 Vessel Collision Risk

A.13.1 Introduction

Vessel collision risk refers to the likelihood and potential consequences of a waterborne vessel (ship, barge, boat) striking a bridge or its supporting structures. This is a major concern for bridges over navigable waterways, such as rivers, ports, and coastal channels, where vessel traffic is common.

Table A.71: U.S. bridge damage/collapse events attributed to vessel collisions (2021-2025).

Date	Bridge (State)	Location / Waterway	Vessel(s) involved	Reported bridge impacts (and other key impacts)
2021-01-12	CSX Railway Rigolets Swing Bridge (LA)	The Rigolets, near Slidell, LA	Towboat *Robert Cenac* pushing 1 empty barge	Barge struck the swing bridge; bridge damage reported; total bridge and barge damages estimated at $1.105M; no injuries/pollution. [176]
2021-12-23	Route 182 Bridge (LA)	Bayou Boeuf / Bayou Ramos, near Amelia, LA	Towboat *Miss Mollye D* pushing 6 barges	Tow left channel and struck the bridge; bridge damage estimated at $2M; incident discovered later; no injuries/pollution. [177]
2022-03-06	Houma Twin Span Bridges (LA)	Gulf Intracoastal Waterway, Houma, LA	Towboat *Robert Cenac* with crane barge *Mr. Dawg*	Crane contacted eastbound span; traffic reduced for 10 days; bridge damage estimated at $1.5–$2.0M; no injuries/pollution. [178]
2023-05-07	Mackinac Bridge (MI)	Straits of Mackinac, MI	Towboat *Nickelena* towing barge with crawler crane	Crane boom struck main span; bridge damage estimated at $145,000; no injuries/pollution. [179]
2023-07-13	Pelican Island Causeway Bridge (TX)	Galveston Channel, Galveston, TX	Towboat *Duke* with 2 barges (plus nearby power poles)	Tow contacted power poles; tug subsequently contacted the bridge; combined damage to vessels/poles/bridge estimated > $300,000; no injuries/pollution. [180]
2024-03-26	Francis Scott Key Bridge (MD)	Patapsco River, Baltimore, MD	Containership *Dali*	Vessel struck bridge pier; substantial portion of bridge collapsed; 6 roadway workers killed; vessel damage > $18M (bridge replacement costs not yet listed on the page at that time). [181]
2024-05-09	Fort Madison Bridge protection cell & fendering system (IA)	Upper Mississippi River, near Fort Madison, IA	Towboat *Joe B. Wyatt* pushing 15 barges	Tow struck bridge protection cell/fendering system; damage to protection system/barges/towboat estimated $3.28M; no injuries/pollution. [182]
2024-05-15	Pelican Island Causeway Bridge (TX)	Galveston, TX	Tug pushing fuel barge(s) (AP report)	Barge struck a support pillar; partial bridge collapse reported; oil spill response and closures; no injuries reported. [183]
2024-06-06	Bayou Gross Tete Swing Bridge / Highway 77 (LA)	Near Plaquemine, LA (Port Allen Lock / ICW)	Towboat *Josset* pushing 3 barges	Swing bridge severely damaged and stuck closed; highway impassable and waterway closed; damages estimated > $500,000. [184]

Representative Case Studies in Texas

- Ship's boom hits Sidney Sherman bridge (2001)
  - In May 2001, a freighter's cargo boom caused severe damage to a girder and put another gaping hole in the span. Several lanes of the bridge were closed for two months while repairs were completed [185].
- 2001 Queen Isabella Causeway Collapse (Port Isabel, TX) (2024)
  - On September 15, 2001, the Queen Isabella Causeway connecting Port Isabel to South Padre Island was struck by a towboat pushing barges. One of the barges collided with a span of the causeway, causing two 80-foot sections to collapse into the Laguna Madre Bay. Vehicles driving on the bridge fell into the water; eight people died and 15 were injured in the disaster [186].
- Barge Collides with I-10 Bridge Pier in 2019 (San Jacinto River, Channelview, TX) (2020)
  - On February 11, 2019, the towing vessel Lindberg Crosby lost engine control while attempting to dock in the San Jacinto River near Channelview. It pushed an empty tank barge into a support beam / column of the Interstate 10 bridge. The resulting impact caused structural damage requiring extensive repairs, with estimated damages of about 1.59 million USD. No injuries or pollution were reported, but the bridge closure and repair work highlighted collision risk in an active waterway [187].
- Barge Hits Pelican Island Causeway Bridge (Galveston, TX) (2024)
  - On May 15, 2024, a barge being pushed by a tugboat lost control in strong current and high tide near the Galveston Ship Channel and collided with a support pillar of the Pelican Island Causeway Bridge, which connects Galveston Island to Pelican Island. The impact caused partial collapse of the bridge structure, forcing closure of the roadway. The collision also spilled oil (est. up to 2,000 gallons) into surrounding water, leading to environmental cleanup efforts. One person was knocked into the water but rescued; no serious injuries were reported [188].

A.13.2 Methods Reported in the Literature

A.13.2.1 FDOT Assessment Procedure [1]

The FDOT network screening assigns an equal hazard likelihood to bridges on navigable waterways that require pier protection and differentiates the bridges through vulnerability terms. In particular, the pier-protection indicator (NBI Item 111) is used to distinguish consequences, and bridge age is used to reflect that newer bridges are more likely to have been designed for vessel impact (whereas older bridges, especially pre-1985, are treated as more vulnerable).

Step-by-step procedure **Step 1**. Obtain the bridge inventory (FDOT/Pontis bridge GIS layer) as the baseline asset set for risk computation.

Step 2. Identify candidate bridges exposed to vessel impact. FDOT's Project Level Analysis Tool (PLAT) implementation uses NBI Item 111 (Pier Protection) as shown in Table A.72 to flag bridges on navigable waterways where pier protection is applicable.

Step 3. Assign the vessel-collision annual hazard likelihood. Due to limited data availability for bridge-by-bridge vessel-traffic modeling, the hazard likelihood is assumed equal for all bridges identified in Step

Table A.72: NBI Item 111: Pier or Abutment Protection (for Navigation)

Code	Description
1	Navigation protection not required.
2	In place and functioning.
3	In place but in a deteriorated condition.
4	In place but reevaluation of design suggested.
5	None present but reevaluation suggested.
N	Not Applicable.

2:

$$p_{\text{vessel}}(i) = \begin{cases} p_0, & \text{if bridge } i \text{ is flagged as vessel-impact exposed (Item 111)} \in \{2,3,4,5\} \\ 0, & \text{otherwise} \end{cases} \tag{A.249}$$

where p_0 is a statewide-calibrated baseline annual probability (**0.381%** is used in this report).

Note: FDOT assigns an equal likelihood to bridges on navigable waterways that require pier protection, identified using NBI Item 111. The baseline value $p_0 = 0.381\%$ is calibrated to historical vessel-collision experience in the Pontis records. The report identifies 31 vessel-collision events in the database. Let T denote the length of the observation period in years and let N denote the number of exposed bridges requiring pier protection. The statewide average annual event rate is approximated by $31/T$, and PLAT allocates this rate uniformly across the exposed bridge set, yielding

$$p_0 \approx \frac{31/T}{N} = \frac{31}{TN} \tag{A.250}$$

Expressed as a percentage, this calibrated per-bridge annual probability is reported in PLAT as 0.381%.

Step 4. Compute the vessel-collision element vulnerability multiplier $V^{\text{elem}}_{\text{vessel}}(i)$. For each bridge, let $\mathcal{E}$ denote the set of bridge elements recorded for that bridge. For each element e, obtain its vessel-collision element score $s_{\text{vessel}}(e) \in \{0,1,2,3,4,5\}$ from the FDOT element vulnerability table for vessel collisions (Table A.73). The bridge-level raw element score is the simple average:

$$\bar{s}_{\text{vessel}}(i) = \frac{1}{|\mathcal{E}|} \sum_{e \in \mathcal{E}} s_{\text{vessel}}(e) \tag{A.251}$$

Rescale to the multiplier:

$$V^{\text{elem}}_{\text{vessel}}(i) = \frac{\bar{s}_{\text{vessel}}(i) + 1}{3} \tag{A.252}$$

Note: The report first compiles historical vessel-impact crash records (31 crashes) and extracts which bridge elements were reported as damaged in each crash, producing 44 "element-in-crash" observations in total. The resulting frequencies by element type (e.g., fenders dominating the observations) are summarized in Table A.74. Because the vessel-impact dataset is small, the report notes it is difficult to establish detailed damage levels/costs directly from the records; therefore the element consequence/vulnerability levels in Table A.73 are assigned using engineering judgment informed by the observed damage patterns (especially the high observed involvement of fender systems, and smaller involvement of items like

Table A.73: Levels of vulnerability of bridge elements to damage from vessel collisions

Table 7.25. Levels of vulnerability of bridge elements to damage from vessel collisions

Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability	Element key	Element short decription	Vulnerability
12	Bare Concrete Deck	1	216	Timber Abutment	1	476	Timber Walls	0
13	Unp Conc Deck/AC Ovl	1	217	Other Mtl Abutment	1	477	Other Walls	0
28	Steel Deck/Open Grid	1	220	R/C Sub Pile Cap/Ftg	1	478	MSE Walls	0
29	Steel Deck/Conc Grid	1	230	Unpnt Stl Cap	1	480	Mast Arm Found	0
30	Corrug/Orthotpc Deck	1	231	Paint Stl Cap	1	481	Paint Mast Arm Vert	0
31	Timber Deck	1	233	P/S Conc Cap	1	482	Galvan Mast Arm Vert	0
32	Timber Deck/AC Ovly	1	234	R/Conc Cap	1	483	Other Mast Arm Vert	0
38	Bare Concrete Slab	1	235	Timber Cap	1	484	Paint Mast Arm Horzn	0
39	Unp Conc Slab/AC Ovl	1	240	Metal Culvert	1	485	Galvan Mast Arm Horz	0
54	Timber Slab	1	241	Concrete Culvert	1	486	Other Mast Arm Horzn	0
55	Timber Slab/AC Ovly	1	242	Timber Culvert	1	487	Sign Member Horiz	0
98	Conc Deck on PC Pane	1	243	Misc Culvert	1	488	Sign Member Vertical	0
99	PS Conc Slab	1	290	Channel	1	489	Sign Foundation	0
101	Unpnt Stl Box Girder	2	298	Pile Jacket Bare	1	495	Uncoat High Mast L.	0
102	Paint Stl Box Girder	2	299	Pile Jacket/Cath Pro	1	496	Painted High Mast L.	0
104	P/S Conc Box Girder	2	300	Strip Seal Exp Joint	1	497	Galvan. High Mast L.	0
105	R/Conc Box Girder	2	301	Pourable Joint Seal	1	498	Other High Mast L.P.	0
106	Unpnt Stl Opn Girder	2	302	Compressn Joint Seal	1	499	H. M. L. P. Found.	0
107	Paint Stl Opn Girder	2	303	Assembly Joint/Seal	1	540	Open Gearing	1
109	P/S Conc Open Girder	2	304	Open Expansion Joint	1	541	Speed Reducers	1
110	R/Conc Open Girder	2	310	Elastomeric Bearing	1	542	Shafts	1
111	Timber Open Girder	2	311	Moveable Bearing	1	543	Shaft Bigs and Coupl	1
112	Unpnt Stl Stringer	2	312	Enclosed Bearing	1	544	Brakes	1
113	Paint Stl Stringer	2	313	Fixed Bearing	1	545	Emergency Drive	1
115	P/S Conc Stringer	2	314	Pot Bearing	1	546	Span Drive Motors	1
116	R/Conc Stringer	2	315	Disk Bearing	1	547	Hydraulic Power Unit	1
117	Timber Stringer	2	320	P/S Conc.Appr Slab	1	548	Hydraulic Piping Sys	1
120	U/Stl Thru Truss/Bot	1	321	R/Conc Approach Slab	1	549	Hydraulic Cylinders	1
121	P/Stl Thru Truss/Bot	1	330	Metal Rail Uncoated	1	550	Hopkins Frame	1
125	U/Stl Thru Truss/Top	1	331	Conc Bridge Railing	1	560	Locks	1
126	P/Stl Thru Truss/Top	1	332	Timb Bridge Railing	1	561	Live Load Shoes	1
130	Unpnt Stl Deck Truss	1	333	Other Bridge Railing	1	562	Counterweight Suppor	1
131	Paint Stl Deck Truss	1	334	Metal Rail Coated	1	563	Acc Ladd & Plat	1
135	Timber Truss/Arch	1	356	Steel Fatigue SmFlag	0	564	Counterweight	1
140	Unpnt Stl Arch	1	357	Pack Rust Smart Flag	0	565	Trun/Str and Cur Trk	1
141	Paint Stl Arch	1	358	Deck Cracking SmFlag	0	570	Transformers	1
143	P/S Conc Arch	1	359	Soffit Smart Flag	0	571	Submarine Cable	1
144	R/Conc Arch	1	360	Settlement SmFlag	0	572	Conduit & Junc. Box	2
145	Other Arch	1	361	Scour Smart Flag	0	573	PLCs	1
146	Misc Cable Uncoated	1	362	Traf Impact SmFlag	0	574	Control Console	1
147	Misc Cable Coated	1	363	Section Loss SmFlag	0	580	Navigational Lights	2
151	Unpnt Stl Floor Beam	1	369	Sub.Sect Loss SmFlag	0	581	Operator Facilities	1
152	Paint Stl Floor Beam	1	370	Alert Smart Flag	0	582	Lift Bridge Spec. Eq	1
154	P/S Conc Floor Beam	1	386	Fender/Dolphin Uncoa	5	583	Swing Bridge Spec. E	1
155	R/Conc Floor Beam	1	387	P/S Fender/Dolphin	5	590	Resistance Barriers	1
156	Timber Floor Beam	1	388	R/Conc Fender/Dolphi	5	591	Warning Gates	1
160	Unpnt Stl Pin/Hanger	1	389	Timber Fender/Dolphi	5	592	Traffic Signals	1
161	Paint Stl Pin/Hanger	1	390	Other Fender/Dolphin	5			
201	Unpnt Stl Column	1	393	Blkhd Sewl Metal Unc	1			
202	Paint Stl Column	1	394	R/Conc Abut Slope Pr	1			
204	P/S Conc Column	1	395	Timber Abut Slope Pr	1			
205	R/Conc Column	1	396	Other Abut Slope Pro	1			
206	Timber Column	1	397	Drain. Syst Metal	0			
207	P/S Conc Holl Pile	1	398	Drain. Syst Other	0			
210	R/Conc Pier Wall	1	399	Other Xpansion Joint	0			
211	Other Mtl Pier Wall	1	474	Walls Uncoated	0			
215	R/Conc Abutment	1	475	R/Conc Walls	0			

* Level of vulnerability: 5 - Extremely High; 4 - Very High; 3 - High; 2 - Moderate; 1 - Low; 0 - Negligible); to avoid zero vulnerability, min of 1 indicated for those elements with no data of damages observed.

Source: [1, Table 7.25]

conduit/junction boxes and navigation lights). Numerically, each entry in Table A.73 is the assigned vulnerability level on the stated ordinal scale (5 = Extremely High, …, 0 = Negligible). The table footnote also states a rule-of-thumb used in these parameter tables: when no damage was observed for an element in the available records, a minimum level may be assigned (often 1) to avoid a strict zero vulnerability purely due to missing observations.

Table A.74: Effect of vessel impact crashes on bridge elements

Table 7.20. Effect of vessel impact crashes on bridge elements

	Element Type Affected	No. of element in crashes	% of all elements in crashes
Core	Deck overhang	1	2.3%
	Railing	1	2.3%
	Beam: girder, bascule span, chord	3	6.8%
	Fender	26	59.1%
	Conduit & Junc. Box	6	13.6%
	Navigational Lights	6	13.6%
	Platform	1	2.3%
	Totals all elements in crashes	44	

Source: [1, Table 7.20]

Step 5. Compute the vessel-collision design/protection vulnerability multiplier $V^{\text{des}}_{\text{vessel}}(i)$ from pier-protection characteristics (NBI Item 111). Let $I_{111}(i)$ denote the NBI Item 111 code for bridge i (pier/abutment protection for navigation). Define the vessel-collision design/protection score $s^{\text{des}}_{\text{vessel}}(i) \in \{0, 1, 2, 3, 4, 5\}$ by the following monotone mapping (better protection $\Rightarrow$ lower vulnera-

bility):

$$s^{\text{des}}_{\text{vessel}}(i) = \begin{cases} 0, & I_{111}(i) = 1 \quad \text{(protection not required)}, \\ 2, & I_{111}(i) = 2 \quad \text{(in place and functioning)}, \\ 3, & I_{111}(i) = 3 \quad \text{(in place, deteriorated)}, \\ 4, & I_{111}(i) = 4 \quad \text{(in place, reevaluation suggested)}, \\ 5, & I_{111}(i) = 5 \quad \text{(none present, reevaluation suggested)}. \end{cases} \tag{A.253}$$

Then the design/protection vulnerability multiplier is

$$V^{\text{des}}_{\text{vessel}}(i) = \frac{s^{\text{des}}_{\text{vessel}}(i) + 1}{3} \tag{A.254}$$

If $I_{111}(i)$ is `N/A` (not applicable), the bridge is typically treated as not exposed to vessel collision and one may set $p_{\text{vessel}}(i) = 0$ in the hazard-likelihood step.

Step 6. Compute the vessel-collision age vulnerability multiplier $V^{\text{age}}_{\text{vessel}}(i)$ as a linear function of bridge age (year built Y_i from NBI Item 27). Following the PLAT convention for age-based vulnerability (i.e., a base year y at which the multiplier equals 1.0), set the vessel-collision base year to the mid-1980s shift in vessel-impact-aware design practice:

$$y = 1985, \qquad m = 30 \text{ years}$$

Then

$$V^{\text{age}}_{\text{vessel}}(i) = \max\left(1,\, 1 + \frac{y - Y_i}{m}\right) = \max\left(1,\, 1 + \frac{1985 - Y_i}{30}\right) \tag{A.255}$$

The lower bound enforces a minimum multiplier of 1.0 so that newer bridges are not assigned a reduced vulnerability below the baseline, consistent with PLAT's use of multipliers as vulnerability amplifiers.

Step 7. Compute the vessel-collision risk index by multiplying the annual hazard likelihood by the vulnerability multipliers, and rank bridges in descending order:

$$\text{Risk}_{\text{vessel}}(i) = p_{\text{vessel}}(i) \cdot V^{\text{elem}}_{\text{vessel}}(i) \cdot V^{\text{age}}_{\text{vessel}}(i) \cdot V^{\text{des}}_{\text{vessel}}(i) \tag{A.256}$$

Agency cost. First, compute the replacement cost for bridge i:

$$R_i = A_i c \tag{A.257}$$

where A_i is deck area and c is the unit replacement cost per area. Define the agency weight as the normalized share over the bridge inventory set $\mathcal{E}$:

$$w_i = \frac{R_i \cdot \text{Risk}_{\text{vessel}}(i)}{\sum_{j \in \mathcal{E}} R_j \cdot \text{Risk}_{\text{vessel}}(j)} \tag{A.258}$$

Given the statewide average annual agency loss from vessel collisions L_{vessel} (annual historical agency cost in Table A.15), allocate it using w_i:

$$\text{AgencyCost}_{\text{vessel}}(i) = L_{\text{vessel}}\, w_i \tag{A.259}$$

User cost. For bridge i, define the daily detour cost:

$$U_i = ADT_i \left(c_{\text{op}} \ell_i + c_{\text{time}} \frac{\ell_i}{v_i} \right) \tag{A.260}$$

where ADT_i is average daily traffic, ℓ_i is detour length, v_i is detour speed, c_{op} is unit operating cost per mile, and c_{time} is value of time per hour. Define the user weight as the normalized share over the bridge inventory:

$$u_i = \frac{U_i \cdot \text{Risk}_{\text{vessel}}(i)}{\sum_{j \in \mathcal{E}} U_j \cdot \text{Risk}_{\text{vessel}}(j)} \tag{A.261}$$

Given the statewide annual control total for vessel-collision user losses L^U_{vessel} (annual historical user cost in Table A.15), allocate it using u_i:

$$\text{UserCost}_{\text{vessel}}(i) = L^U_{\text{vessel}}\, u_i \tag{A.262}$$

Total cost. After computing $\text{AgencyCost}_{\text{vessel}}(i)$ and $\text{UserCost}_{\text{vessel}}(i)$ for each bridge i, form the total annual expected impact (USD/year) by summing the two components:

$$\text{TotalCost}_{\text{vessel}}(i) = \text{AgencyCost}_{\text{vessel}}(i) + \text{UserCost}_{\text{vessel}}(i) \tag{A.263}$$

A.13.2.2 FDOT Vessel Collision Risk Assessment (Modified AASHTO Guide Method II) [16]

This procedure estimates the annual frequency of bridge collapse due to vessel collision, expressed as the Annual Frequency of Collapse (AFC). The FDOT implementation adapts AASHTO Method II for network-level use by substituting detailed site-specific inputs with proxy variables available in NBI/BMS databases.

Step-by-step procedure Step 1. Define the analysis asset set (navigable bridges) from the FDOT/NBI bridge inventory. Use the NBI inventory field Navigation Control (NBI Item 38 = 1) to identify bridges that accommodate vessel traffic. Retain only bridges with valid Navigation Control coding as the baseline "navigable bridge inventory" for subsequent computations and for determining inventory-based normalization bounds (e.g., min/max values used later for rescaling proxy variables).

Step 2. For each bridge, compile required proxy attributes from the bridge inventory and BMS databases:

- NBI Item 45: Number of spans in main unit (MAINSPANS).
- NBI Item 39: Navigation vertical clearance (NAVVC).
- NBI Item 40: Navigation horizontal clearance (NAVHC).
- NBI Item 48: Length of maximum bridge span.
- NBI Item 59: Superstructure condition rating.

- NBI Item 60: Substructure condition rating.
- NBI Item 111: Pier or abutment protection.
- NBI Item 113: Scour critical.
- BMS: Scour evaluation rating.

Step 3. Estimate the annual number of vessels, N, at each bridge location using FDOT Past Point vessel traffic data. For a desired design year y, define time relative to a datum year y_0:

$$t = y - y_0 \tag{A.264}$$

and compute annual vessel trips (both directions) using the Past Point projection model:

$$N(t) = N(y - y_0) = a_0\, t^{a_1} + a_2 \tag{A.265}$$

where (a_0, a_1, a_2) are Past Point parameters provided for each bridge location.

Step 4. Compute the geometric probability PG as a proxy for the likelihood that an aberrant vessel will geometrically hit the bridge (Equation 2 and Figure 4).

Define four unitless proxy scores on $[0, 1]$:

$$S_{\text{CVA}} = \frac{\theta}{90^\circ}, \tag{A.266}$$

$$S_{45} = \frac{\text{MAINSPANS} - \text{MAINSPANS}_{\text{min}}}{\text{MAINSPANS}_{\text{max}} - \text{MAINSPANS}_{\text{min}}}, \tag{A.267}$$

$$S_{39} = \frac{\text{NAVVC}_{\text{max}} - \text{NAVVC}}{\text{NAVVC}_{\text{max}} - \text{NAVVC}_{\text{min}}}, \tag{A.268}$$

$$S_{40} = \frac{\text{NAVHC}_{\text{max}} - \text{NAVHC}}{\text{NAVHC}_{\text{max}} - \text{NAVHC}_{\text{min}}} \tag{A.269}$$

Here θ is the channel turning angle at the bridge location (degrees). The quantities MAINSPANS, NAVVC, and NAVHC are taken from the bridge inventory (NBI Items 45, 39, and 40). The bounds $(\cdot)_{\text{min}}$ and $(\cdot)_{\text{max}}$ are computed over the navigable-bridge inventory used in the analysis. The score meanings are: S_{CVA} (larger turn angle $\Rightarrow$ harder maneuvering), S_{45} (more main spans $\Rightarrow$ more in-water obstruction), S_{39} (smaller vertical clearance $\Rightarrow$ tighter envelope), and S_{40} (smaller horizontal clearance $\Rightarrow$ narrower envelope).

Compute PG using a conservative worst-factor rule:

$$PG = \max\{S_{\text{CVA}}, S_{45}, S_{39}, S_{40}\} \tag{A.270}$$

i.e., the most unfavorable geometric factor is assumed to dominate the hit probability.

Step 5. Compute the Probability of Collapse, PC, using five failure-related proxy probabilities, each on a 0.0–1.0 scale:

- p_{111} from NBI Item 111 (pier/abutment protection), using the probability assignments in Table A.75.
- p_{59} from NBI Item 59 (superstructure condition rating), converted as

$$p_{59} = \frac{9 - \text{Rating}_{59}}{9} \tag{A.271}$$

- p_{60} from NBI Item 60 (substructure condition rating), converted as

$$p_{60} = \frac{9 - \text{Rating}_{60}}{9} \tag{A.272}$$

- p_{113} from NBI Item 113 (scour critical), using the code-to-probability mapping in Table 4.
- p_{BMS} from BMS scour evaluation, using the code-to-probability mapping in Table 5.

Then compute

$$PC = \max\left\{p_{111},\ p_{59},\ p_{60},\ p_{113},\ p_{\text{BMS}}\right\} \tag{A.273}$$

(Reference: Equation 3; Tables 3–5; Figures 5–6 in the report.)

Table A.75: NBI Item 111 Pier or abutment protection and assigned factors

Code	Description	Assigned Rating	Assigned Prob.	Assigned Prot. Factor
1	Navigation protection not required.	9.00	0.0	1.0
2	In place and functioning.	9.00	0.0	1.0
3	In place but in a deteriorated condition.	5.00	0.4	2.0
4	In place but reevaluation of design suggested.	2.00	0.8	2.0
5	None present but reevaluation suggested.	0.00	1.0	2.0
N	Not Applicable	9.00	0.0	1.0

Step 6. Compute the Protection Factor, PF, based on NBI Item 111 (pier/abutment protection). Assign

$$PF = \begin{cases} 1.0, & \text{if NBI Item 111 code is } 1, 2, \text{ or } N, \\ 2.0, & \text{if NBI Item 111 code is } 3, 4, \text{ or } 5. \end{cases} \tag{A.274}$$

(Reference: Equation 4 and Table 3 in the report.)

Step 7. Compute the Probability of Aberrancy, PA, as

$$PA = (BR)(RB)(RC)(RXC)(RD) \tag{A.275}$$

where the variables are defined as follows.

Step 7a. Set the aberrrancy base rate BR (e.g., adopt the barge value):

$$BR = 1.2 \times 10^{-4} \tag{A.276}$$

Step 7b. Compute the bridge correction factor RB based on the waterway region classification and turning angle θ:

$$RB = \begin{cases} 1.0, & \text{Straight region}, \\ 1 + \theta/90^\circ, & \text{Transition region}, \\ 1 + \theta/45^\circ, & \text{Turn/Bend region}. \end{cases} \tag{A.277}$$

(Reference: Equation 6 and Figure 4 in the report.)

Step 7c. Set current-related correction factors. If current data are unavailable, assume:

$$RC = 1.0, \qquad RXC = 1.0 \tag{A.278}$$

Step 7d. Compute the traffic density correction factor RD. First compute

$$v_t = \frac{N}{L_{\text{max}}} \tag{A.279}$$

where L_{max} is NBI Item 48 (length of maximum bridge span). Then prorate v_t to the RD range (consistent with the report's stated low-to-high range 1.0–1.6) using inventory bounds $v_{t,\text{min}}, v_{t,\text{max}}$:

$$RD = 1.0 + \left(\frac{v_t - v_{t,\text{min}}}{v_{t,\text{max}} - v_{t,\text{min}}} \right) (1.6 - 1.0) \tag{A.280}$$

(Reference: Equation 7 and Table 6 in the report.)

Step 8. Compute the Annual Frequency of Collapse (AFC) as

$$AFC = (N)(PA)(PG)(PC)(PF) \tag{A.281}$$

(Reference: Equation 1 and Table 2 in the report.)

Step 9. Optionally compute the return period of collapse due to vessel collision as

$$T_{\text{return}} = \frac{1}{AFC} \tag{A.282}$$

A.13.2.3 Oklahoma Vessel Collision Assessment (AASHTO Method II) [17]

In the Oklahoma report, vessel-collision risk is quantified using the AASHTO Method II probability-based procedure. The primary risk metric is the **Annual Frequency of Collapse (AF)** of bridge substructure elements (piers) due to vessel impact. The method combines (i) vessel traffic exposure, (ii) probability of vessel aberrancy, (iii) geometric probability of striking a pier, (iv) probability of pier collapse given impact, and (v) access/protection reduction factors. The procedure is applied for both present traffic and a future projection year to compare the two traffic conditions.

In this report, the annual frequency of pier (or substructure component) collapse due to vessel collision is modeled as

$$AF = N \cdot P_A \cdot P_G \cdot P_C \tag{A.283}$$

where the effective exposure N is reduced using access/protection factors (water depth and pier protection). Equivalently,

$$AF = N_0 \cdot P_{\text{depth}} \cdot P_{\text{prot}} \cdot P_A \cdot P_G \cdot P_C \tag{A.284}$$

Unless otherwise stated, each term is evaluated for a specific pier i, vessel group g, traffic direction d, and impact direction k (i.e., $AF \equiv AF_{i,g,d,k}$).

- AF : **Annual frequency of collapse** of the considered bridge component (e.g., pier) due to vessel collision (unit: 1/year).

- N_0 : **Raw annual vessel traffic exposure** (annual number of vessel/tow trips of a particular vessel group that pass the bridge vicinity) before applying any accessibility/protection reductions (unit: trips/year).
- P_{depth} : **Water-depth access factor** (dimensionless, $0 \leq P_{\text{depth}} \leq 1$). It represents how often the water is deep enough for the vessel to reach the pier. It is the fraction of time (or fraction of vessel trips) when the channel depth at the pier is greater than or equal to the vessel draft, so grounding will not prevent the vessel from physically getting to the pier. A simple screening definition is:

$$P_{\text{depth}} = \frac{\text{number of relevant trips (or time periods) with depth at pier } \geq \text{ vessel draft}}{\text{total number of relevant trips (or time periods) considered}}$$

  If the channel is deep enough for all relevant trips, then $P_{\text{depth}} = 1$; if it is never deep enough, then $P_{\text{depth}} = 0$.
- P_{prot} : **Pier-protection / barrier access factor** (dimensionless, $0 \leq P_{\text{prot}} \leq 1$), representing reduction in pier reachability due to physical barriers or protection systems (e.g., dolphins, fenders, guide walls, land masses, other piers). A common screening implementation is:

$$P_{\text{prot}} = \frac{W_{\text{total}} - W_{\text{blocked}}}{W_{\text{total}}}$$

  where W_{total} is the accessible channel width for the vessel group and W_{blocked} is the portion effectively blocked by protection/barriers.
- N : **Effective annual exposure** after applying accessibility/protection reductions (unit: trips/year), defined as

$$N = N_0 \cdot P_{\text{depth}} \cdot P_{\text{prot}}$$

- P_A : **Probability of aberrancy** (dimensionless, $0 \leq P_A \leq 1$), the probability that a transiting vessel becomes aberrant (off its intended navigation path) in the bridge vicinity due to human error, mechanical failure, environmental effects, etc. (evaluated for vessel group and direction).
- P_G : **Geometric probability of pier contact** (dimensionless, $0 \leq P_G \leq 1$), the conditional probability that an aberrant vessel will reach and strike the specific pier, considering channel geometry, vessel width/trajectory dispersion, pier location/skew, shoreline constraints, etc. (evaluated per pier and impact case).
- P_C : **Conditional probability of collapse given impact** (dimensionless, $0 \leq P_C \leq 1$), the probability that the impacted component collapses (or reaches the defined failure state) given that the vessel strikes the pier, based on the relationship between impact demand (force/energy) and structural resistance/capacity.

Notes.

- All P terms are dimensionless probabilities or probability-like adjustment factors; N_0 and N are annual counts (trips/year); therefore AF has units of 1/year.
- For a system-level metric (bridge/substructure), AF is typically summed over exposed piers and over vessel groups, traffic directions, and impact directions (as applicable).

System (substructure) risk aggregation. The total annual frequency of substructure collapse is obtained by summing the controlling contributions across exposed piers (and across vessel groups, traffic directions, and impact directions):

$$AF_{\text{sub}} = \sum_{i\in\mathcal{P}} \sum_{g\in\mathcal{G}} \sum_{d\in\{\uparrow,\downarrow\}} \sum_{k\in\{\parallel,\perp\}} AF_{i,g,d,k} \tag{A.285}$$

Step-by-step procedure **Step 1. Define analysis cases (present and future traffic).** Define two traffic scenarios for the risk computation: (i) a baseline (present) case representing current vessel traffic conditions and (ii) a future case representing a projected traffic level over the service life of the bridge. In this report, the baseline case uses 2021 as the present-condition evaluation year, and the future case uses 2060 as the projection year, assuming a 50% increase in vessel traffic relative to the baseline. Accordingly, the annual vessel trip counts used in the AASHTO Method II risk equation are scaled as

$$N_{2060} = 1.5\, N_{2022} \tag{A.286}$$

and the Annual Frequency of Collapse (AF) is computed for both years for comparison.

Step 2. Define evaluation vessel groups and traffic directions. Define representative vessel/tow groups and traffic directions for subsequent computations. The report updates the traffic data and concludes that there are no changes in traffic make-up that would require revising the vessel groups established in the previous study. Therefore, six evaluation barge-tow groups are used: Hopper 1–3 and Tanker 1–3. These six groups are defined primarily by tow size (number of barges per tow), as shown in Figure A.14.

- Hopper 1: 10–12 barges per tow;
- Hopper 2: 5–9 barges per tow;
- Hopper 3: 1–4 barges per tow;
- Tanker 1: 5–6 barges per tow;
- Tanker 2: 3–4 barges per tow;
- Tanker 3: 1–2 barges per tow.

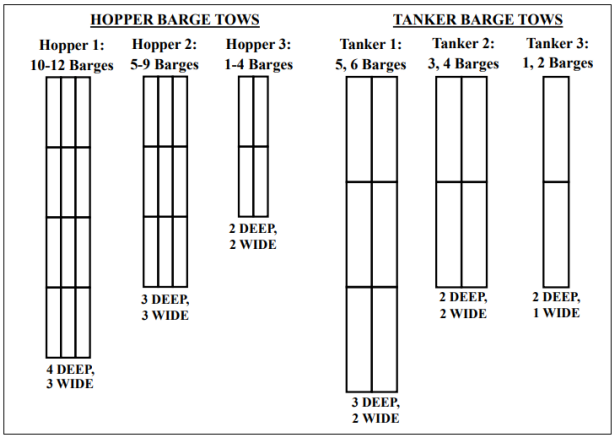


Figure A.14: Evaluation Vessel Groups
Source: [17, Figure 5.6-1]

For each group, representative physical parameters (barge length/width, loaded/light draft, capacity, towboat

length) are specified in Table A.76. These parameters are based on typical barge characteristics listed in the AASHTO Guide, and a 90-ft towboat is selected for the representative tow configuration.

Table A.76: Representative Barge Tows

Barge	Length (Feet)	Width (Feet)	Loaded Draft (Feet)	Light Draft (Feet)	Max. Capacity (tons)	Length of Towboat (Feet)
Hopper 1	4×195	3×35	9	2	20,400	90
Hopper 2	3×195	3×35	9	2	15,300	90
Hopper 3	2×195	3×35	9	2	6,800	90
Tanker 1	3×290	2×53	9	2	18,500	90
Tanker 2	2×290	2×53	9	2	14,800	90
Tanker 3	2×290	1×53	9	2	7,400	90

Source: [17, Table 5.6-1]

Step 3. Identify exposed piers and determine pier accessibility. The report first identifies the piers that are exposed to vessel access under high-water events. For the existing I-40 bridge, piers 2 through 12 are considered exposed, while the remaining piers are on land and are not likely to be struck by barges. This determination is supported by bridge elevation and riverbed (bathymetry) information, using data from the previous study and the 2020 underwater inspection report (Figure A.15).

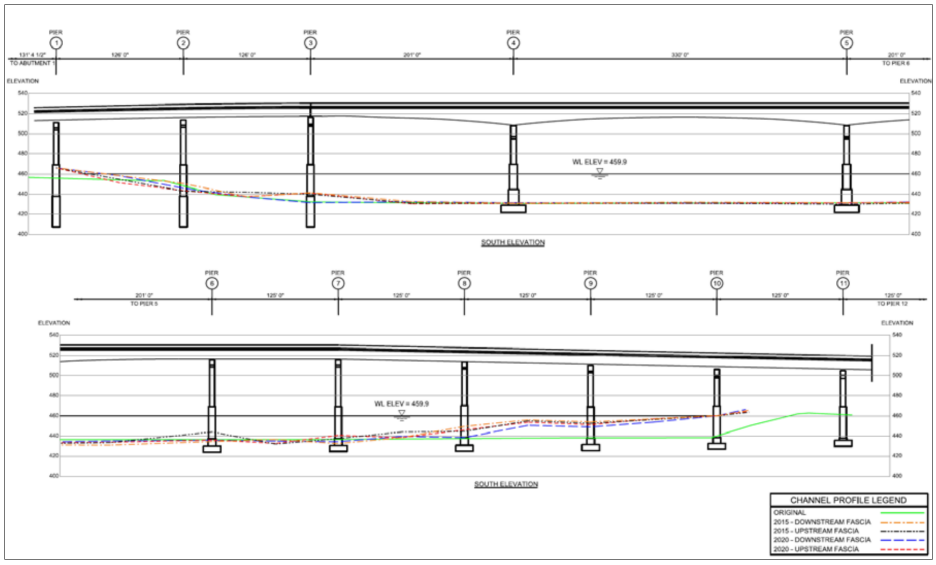


Figure A.15: Bridge Elevation and Riverbed Data
Source: [17, Figure 7.2-1]

Step 4. Compute the annual vessel transit frequency (N_0) from lock/traffic data. For each vessel group g and traffic direction d, compute the annual number of barge-tow trips $N_0(g, d)$. In this report, present-condition traffic is developed primarily from U.S. Army Corps of Engineers (USACE) lock-based statistics, using the Lock Performance Monitoring System (LPMS) as the main source (supplemented by USACE Waterborne Commerce Statistics Center (WCSC) analyses and interviews).

Workflow to obtain $N_0(g, d)$:

1. **Assemble LPMS traffic summaries for a representative year.** The report uses 2021 as a representative present-traffic year for developing barge/tow distributions at the site (based on Webbers Falls Lock statistics), and presents the resulting downbound/upbound distributions in Figure A.16 (barge type distribution) and Figure A.17 (barge tow type distribution).

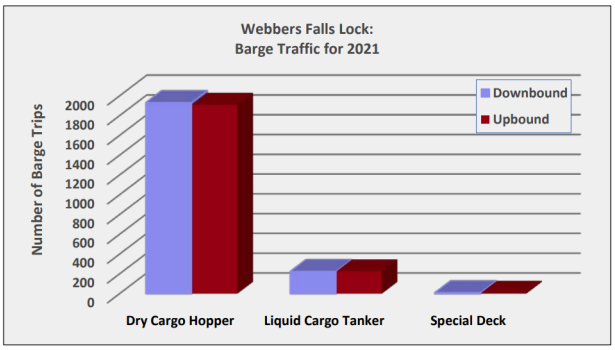


Figure A.16: Barge Type Distribution at Webbers Falls Lock in 2021
Source: [17, Figure 5.4-2]

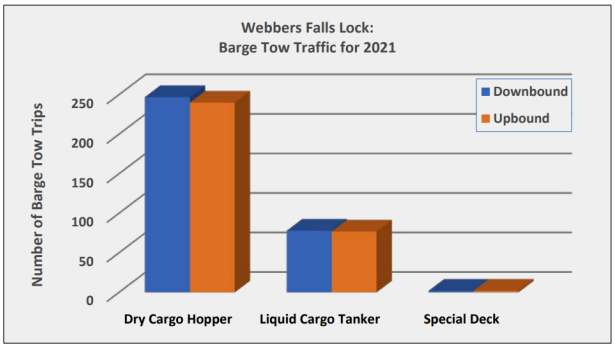


Figure A.17: Barge Tow Type Distribution at Webbers Falls Lock in 2021
Source: [17, Figure 5.4-3]

2. **Tabulate tow trips by type, tow size, and direction.** From LPMS, the tow characteristics used for grouping include: (i) number of barges by type $\times$ loading condition $\times$ direction, and (ii) number of barge tows by type $\times$ number of barges per tow $\times$ direction.
3. **Map each tow trip to an evaluation vessel group.** Assign each observed tow trip to one of the six evaluation vessel groups (Hopper 1-3 and Tanker 1-3)
4. **Aggregate trips to obtain $N_0(g,d)$.** Sum the assigned trips to obtain annual trip counts for each vessel group and direction:

$$N_0(g,d) \;=\; \sum_{\text{tow trips } t \in \mathcal{T}(d)} \mathbf{1}[t \in g] \tag{A.287}$$

where $\mathcal{T}(d)$ is the set of observed tow trips in direction d and $\mathbf{1}[\cdot]$ is the indicator function. The resulting evaluation vessel group trip distribution (including downbound/upbound and loaded/empty) is summarized in Table A.77 for the present-traffic case.

Step 5. Apply access/protection reductions to obtain effective exposure (N). For each pier i, vessel group g, traffic direction d, and impact direction k, reduce the trip count to reflect whether the vessel can

Table A.77: Evaluation Vessel Group Trip Distribution (Present Traffic)

Barge Tow	Total Trips		Loaded		Empty	
	Downbound	Upbound	Downbound	Upbound	Downbound	Upbound
Hopper 1	95	80	75	70	20	10
Hopper 2	180	175	130	150	50	25
Hopper 3	65	55	45	45	20	10
Tanker 1	15	15	10	5	5	10
Tanker 2	35	40	30	10	5	30
Tanker 3	75	70	70	55	5	15

Source: [17, Table 5.6-3]

physically reach the pier:

$$N(i,g,d,k) = N_0(g,d) \cdot P_{\text{depth}}(i,g,d,k) \cdot P_{\text{prot}}(i,g,d,k) \tag{A.288}$$

where,

- P_{depth} accounts for vessel draft versus available water depth (water-depth access factor),
- P_{prot} accounts for pier protection systems or other barriers that limit vessel access (pier-protection access factor).

Table A.78: Screening-level access/protection factors (P_{depth}, P_{prot}) inferred from the 2022 I-40 Bridge vessel-collision risk assessment report.

Pier	P_{depth}	Rationale (water-depth access)	P_{prot}	Rationale (pier-protection / barrier access)
1	0	Not listed among high-water "piers exposed to vessel access" (2-12); treated as not reachable in screening.	1.00	Not evaluated because $P_{\text{depth}} = 0$ (no vessel access).
2	1	In the high-water exposed set (2-12); navigation channel is maintained to a 9-ft draft (screening assumes depth is not the limiting constraint for exposed piers).	1.00	No explicit independent barrier geometry reported for this pier; treat as unblocked in screening.
3	1	Same as Pier 2 (exposed in high water).	1.00	Same as Pier 2.
4	1	Same as Pier 2 (exposed in high water).	0.73	Two 40-ft-diameter upstream sheet-pile dolphins protect Piers 4 and 5. Screening approximation: $P_{\text{prot}} \approx 1 - \frac{W_{\text{blocked}}}{W_{\text{channel}}} = 1 - \frac{2\times 40}{300} = 0.73$, using $W_{\text{channel}} \approx 300$ ft and $W_{\text{blocked}} \approx 2 \times 40$ ft.
5	1	Same as Pier 2 (exposed in high water).	0.73	Same as Pier 4.
6	1	Same as Pier 2 (exposed in high water).	1.00	The screening assigns 1.00 because the report identifies selected 12-ft drilled-shaft protection without a per-pier blockage width or coverage value.
7	1	Same as Pier 2 (exposed in high water).	1.00	Same as Pier 6.
8	1	Same as Pier 2 (exposed in high water).	1.00	Same as Pier 6.
9	1	Same as Pier 2 (exposed in high water).	1.00	Same as Pier 6.
10	1	Same as Pier 2 (exposed in high water).	1.00	Same as Pier 6.
11	1	Same as Pier 2 (exposed in high water).	1.00	Same as Pier 6.
12	1	Same as Pier 2 (exposed in high water).	1.00	Same as Pier 6.
13	0	Pier 13 exists, but it is not included in the high-water exposed set (2–12); treated as not reachable in screening.	1.00	Not evaluated because $P_{\text{depth}} = 0$ (no vessel access).

Step 6. Compute probability of aberrancy (P_A). Compute the probability that a vessel becomes aberrant

Table A.79: Inferred access/protection factors (P_{depth}, P_{prot})

Pier	P_{depth}	$P_{\text{prot}}^{\text{down}}$	$P_{\text{prot}}^{\text{up}}$
1	0	1.00	1.00
2	1	1.00	1.00
3	1	0.9†	0.9†
4	1	0.73	1.00
5	1	0.73	1.00
6	1	0.9†	0.9†
7	1	0.9†	0.9†
8	1	0.9†	0.9†
9	1	0.9†	0.9†
10	1	0.9†	1.00
11	1	1.00	1.00
12	1	1.00	1.00
13	0	1.00	1.00

Note: P_{depth} is treated as a screening "reachability gate" under high-water conditions: piers 2–12 are considered exposed to vessel access ($P_{\text{depth}} = 1$), while other piers are assigned $P_{\text{depth}} = 0$. P_{prot} is shown separately for downbound and upbound traffic. For Piers 4–5 in the downbound direction, $P_{\text{prot}}^{\text{down}} = 0.73$ is a screening approximation based on two 40-ft-diameter upstream sheet-pile dolphins (Figure A.20) and a 300-ft-wide navigation channel: $P_{\text{prot}}^{\text{down}} \approx 1 - (2 \times 40)/300 = 0.73$. Entries marked with † have selected 12-ft drilled-shaft protection shown in Figure A.18 and Figure A.19; the screening assigns $P_{\text{prot}} = 0.9$ where the available geometry supports a protection credit without a quantified blockage width.

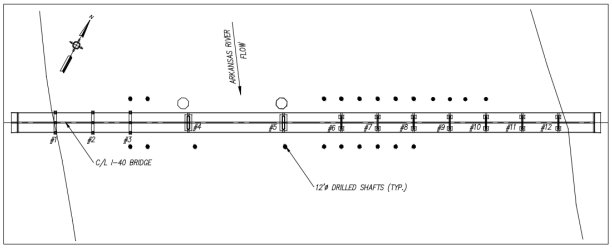


Figure A.18: Bridge Plan View Showing Additional 12-Foot Diameter Drilled Shafts Protection
Source: [17, Figure 2.1-4]



Figure 2.4-2: View of the 12-Foot Drilled Shaft Protection on the Downstream Side of the Bridge

Figure A.19: View of the 12-Foot Drilled Shaft Protection on the Downstream Side of the Bridge
Source: [17, Figure 2.4-2]



Figure A.20: Sheet Pile Dolphins Upstream of Piers 4 and 5
Source: [17, Figure 2.4-1]

(strays off course) in the vicinity of the bridge. The report notes two AASHTO-recommended approaches:

(6a) **AASHTO formula method.** Compute the probability of aberrancy using the AASHTO-recommended multiplicative model:

$$P_A = (BR)\,(R_B)\,(R_C)\,(R_{XC})\,(R_D) \tag{A.289}$$

where BR is the base aberrancy rate for the vessel type and R_B, R_C, R_{XC}, R_D are correction factors for bridge location/geometry, current, cross-currents, and traffic density, respectively.

Base aberrancy rate (BR). For barge tows, the report adopts the AASHTO-recommended base value

$$BR = 1.2 \times 10^{-4} \tag{A.290}$$

The following correction-factor formulas apply the commonly used AASHTO/LRFD implementation for R_B, R_C, R_{XC}, and R_D:

$$R_C = 1 + \frac{V_C}{10}, \tag{A.291}$$

$$R_{XC} = 1 + V_{XC}, \tag{A.292}$$

$$R_B = \begin{cases} 1.0, & \text{Straight region} \\ 1 + \theta/90^\circ, & \text{Transition region} \\ 1 + \theta/45^\circ, & \text{Turn/Bend region} \end{cases} \tag{A.293}$$

$$R_D = \begin{cases} 1.0, & \text{low traffic density (rarely meet/pass)} \\ 1.3, & \text{average (occasionally)} \\ 1.6, & \text{high (routinely)} \end{cases} \tag{A.294}$$

where V_C is the current component (knots) parallel to the transit/aberrant path, V_{XC} is the cross-current component (knots), and θ is the channel bend/turn angle.

Site inputs documented in the I-40 report. The report documents (i) an upstream bend of 31° about 1,375 ft upstream and a downstream bend of 24° about 3,240 ft downstream, (ii) representative current

magnitudes including a yearly mean of 1.5 knots (downstream direction) and an assumed $V_C = 2.5$ knots as a representative current component parallel to an aberrant path, (iii) no observed cross currents, so $V_{XC} = 0$ knots, and (iv) low vessel traffic density (rarely meet/pass), so $R_D = 1.0$.

Directional assignment of bend angles. For downstream (downbound) traffic, the controlling approach-side bend is taken as the upstream bend $\theta_{\text{down}} = 31^\circ$. For upstream (upbound) traffic, the controlling approach-side bend is taken as the downstream bend $\theta_{\text{up}} = 24^\circ$.

Compute R_C, R_{XC}, and R_D. Using $V_C = 2.5$ knots and $V_{XC} = 0$ knots,

$$R_C = 1 + \frac{2.5}{10} = 1.25, \qquad R_{XC} = 1 + 0 = 1.0, \qquad R_D = 1.0 \tag{A.295}$$

Compute R_B and P_A The bridge vicinity is evaluated under both the Transition-region and Turn/Bend-region classifications.

Case A: Transition region.

$$R_B^{\text{down}} = 1 + \frac{31^\circ}{90^\circ} = 1.344, \tag{A.296}$$

$$R_B^{\text{up}} = 1 + \frac{24^\circ}{90^\circ} = 1.267 \tag{A.297}$$

Then

$$P_A^{\text{down}} = (1.2 \times 10^{-4})(1.344)(1.25)(1.0)(1.0) = 2.02 \times 10^{-4}, \tag{A.298}$$

$$P_A^{\text{up}} = (1.2 \times 10^{-4})(1.267)(1.25)(1.0)(1.0) = 1.90 \times 10^{-4} \tag{A.299}$$

Case B: Turn/Bend region.

$$R_B^{\text{down}} = 1 + \frac{31^\circ}{45^\circ} = 1.689, \tag{A.300}$$

$$R_B^{\text{up}} = 1 + \frac{24^\circ}{45^\circ} = 1.533 \tag{A.301}$$

Then

$$P_A^{\text{down}} = (1.2 \times 10^{-4})(1.689)(1.25)(1.0)(1.0) = 2.53 \times 10^{-4}, \tag{A.302}$$

$$P_A^{\text{up}} = (1.2 \times 10^{-4})(1.533)(1.25)(1.0)(1.0) = 2.30 \times 10^{-4} \tag{A.303}$$

Remark. These AASHTO-formula estimates vary with the region classification (Transition vs. Turn/Bend) and the selected value of V_C (1.5 vs. 2.5 knots). The report adopts a site-specific, historical-incident-based P_A for downstream and upstream traffic; these values are lower than the AASHTO-formula estimates.

(6b) **Historical-incident method (site/waterway data).** Estimate a historical base aberrancy rate BR from incident records on the study waterway, then apply the same AASHTO-style local correction factors to obtain P_A.

Step 6b-1. Define the historical reporting window and split the waterway. Use the historical incident reporting period 1991-2001, so

$$N_{\text{years}} = 11$$

Analyze incidents separately for the Oklahoma and Arkansas portions of the navigation system, with

$$L_{\text{waterway}} = \begin{cases} 136.2 \text{ miles}, & \text{Oklahoma portion}, \\ 308.6 \text{ miles}, & \text{Arkansas portion}. \end{cases}$$

Define the bridge "vicinity" influence length as

$$L_{\text{vicinity}} = 2.0 \text{ miles}$$

Step 6b-2. Obtain annual vessel-trip volume T from LPMS lock statistics. Use Lock Performance Monitoring System (LPMS) Monthly Summary Statistics and consider only barge tows. Compute average annual tow counts T by averaging LPMS tow data from representative locks (1995 and 1999):

$$T = \begin{cases} T_{\text{OK}} = 784 \text{ barge tows/year}, \\ T_{\text{AR}} = 1225 \text{ barge tows/year}. \end{cases}$$

Step 6b-3. Set geometric adjustment factors $F_{PG,a}$. For incident type a, use the following geometric adjustment factors:

$$F_{PG,a} = \begin{cases} 0.30, & a = \text{allision}, \\ 0.20, & a = \text{collision (vessel–vessel)}, \\ 1.00, & a = \text{loss of vessel control}, \\ 0.80, & a = \text{grounding (from the Kristiansen model)}. \end{cases}$$

Step 6b-4. Compile incident counts $N_{\text{inc},a}$ by type. Let $N_{\text{inc},a}$ be the number of recorded incidents of type a in the reporting window (for each waterway portion). The incident counts used are:

Accident type a	$\boldsymbol{N_{\text{inc},a}}$ (OK)	$\boldsymbol{N_{\text{inc},a}}$ (AR)
Allisions	2	4
Groundings	8	55
Collisions	1	1
Loss of vessel control	1	19

Step 6b-5. Compute BR_a and sum to obtain BR. For each accident type a, compute the contribution:

$$BR_a = \left(\frac{N_{\text{inc},a} \, F_{PG,a}}{N_{\text{years}} \, L_{\text{waterway}} \, T} \right) \, L_{\text{vicinity}} \tag{A.304}$$

Then:

$$BR = \sum_{a} BR_a \tag{A.305}$$

The report tabulates the resulting contributions and totals as:

Oklahoma portion	$N_{\text{inc},a}$	$F_{PG,a}$	$\boldsymbol{BR_a}$
Allisions	2	0.30	1.0×10^{-5}
Groundings	8	0.80	1.7×10^{-5}
Collisions	1	0.20	8.5×10^{-6}
Loss of vessel control	1	1.00	1.7×10^{-6}
$\sum_a BR_a$			$\mathbf{3.8 \times 10^{-5}}$

Arkansas portion	$N_{\text{inc},a}$	$F_{PG,a}$	$\boldsymbol{BR_a}$
Allisions	4	0.30	5.8×10^{-6}
Groundings	55	0.80	3.4×10^{-5}
Collisions	1	0.20	2.4×10^{-6}
Loss of vessel control	19	1.00	9.1×10^{-6}
$\sum_a BR_a$			$\mathbf{5.1 \times 10^{-5}}$

Step 6b-6. Apply local correction factors to obtain P_A. Using the same AASHTO-style correction factors as in the formula method,

$$P_A = BR\,(R_B)\,(R_C)\,(R_{XC})\,(R_D) \tag{A.306}$$

with the report's local influencing conditions (bends/skew, current, cross-current, and traffic density). Using these local influencing factors, the report computes the barge-tow P_A as:

$$P_A = 8.0 \times 10^{-5} \text{ (downstream traffic)}, \qquad P_A = 5.0 \times 10^{-5} \text{ (upstream traffic)}$$

Step 7. Compute geometric probability (P_G).

The report defines the geometric probability P_G as the conditional probability that an aberrant vessel will hit a specific pier and computes P_G from a normal distribution for the vessel's lateral location across the waterway. One standard deviation equals the length overall of the evaluation vessel group. Pier size, pier skew, tow width, shoreline constraints, vessel category, and traffic direction determine the calculation. The case study presents the resulting annual frequencies of collapse AF; the reconstruction below calculates the intermediate P_G values by pier and vessel group.

Let y denote the cross-channel (lateral) position of the aberrant tow's trackline at the bridge crossing. Model y as normally distributed:

$$y \sim \mathcal{N}(\mu_{g,d},\, \sigma_g^2) \tag{A.307}$$

where,

$\mu_{g,d}$ is the mean (nominal) lateral track position for vessel group g and traffic direction d (upstream/downstream);
σ_g is the standard deviation of the aberrant lateral position distribution for group g.

The report assumes:

$$\sigma_g \approx \text{LOA}_g \tag{A.308}$$

i.e., one standard deviation is taken as the length overall of the evaluation vessel group.

Zone of impact for a specific pier. For pier i (and for a specified traffic direction d and vessel group g), define the "zone of impact" $\mathcal{Z}_{i,g,d}$ on the lateral axis as the set of lateral positions y for which an aberrant tow would contact the pier. The report states that this zone depends on:

- pier size (projected width in the cross-channel direction),

- pier skew relative to the channel,
- tow width (wider tows increase contact likelihood),
- shoreline constraints (which may block the tow from reaching some piers).

In a screening implementation, $\mathcal{Z}_{i,g,d}$ is often represented by a simple interval $[y_{L,i,g,d},\, y_{R,i,g,d}]$, where y_L and y_R are the left/right lateral bounds of the effective impact zone (after accounting for pier width, tow width, skew, and any blocked region due to shoreline).

Vessel-group inputs (definitions of variables). Using Table A.76, define a vessel (tow) group g by a representative barge-tow configuration. The report parameterizes each group using the number of barges in the tow (in the longitudinal and transverse directions) and standard barge/towboat dimensions. For a given group g:

- g : evaluation vessel (tow) group index (e.g., Hopper 1-3, Tanker 1-3).
- LOA_g : length overall of the representative tow for group g (ft), i.e., the total longitudinal length of the tow measured from the front of the leading barge to the back of the towboat.
- $n_{\mathrm{deep},g}$: number of barges in-line ("deep") in the tow for group g (dimensionless), i.e., how many barges are arranged front-to-back.
- L_{barge} : length of one barge (ft). The report uses $L_{\mathrm{barge}} = 195$ ft for hopper barges and $L_{\mathrm{barge}} = 290$ ft for tanker barges.
- L_{towboat} : length of the towboat (ft). The report uses a representative towboat length $L_{\mathrm{towboat}} = 90$ ft.
- W_g : overall tow width for group g (ft), i.e., the transverse width of the barge arrangement.
- $n_{\mathrm{wide},g}$: number of barges abreast ("wide") in the tow for group g (dimensionless), i.e., how many barges are arranged side-by-side.
- B_{barge} : width (beam) of one barge (ft). The report uses $B_{\mathrm{barge}} = 35$ ft for hopper barges and $B_{\mathrm{barge}} = 53$ ft for tanker barges.

With these definitions, the report computes the representative tow dimensions as:

$$\mathrm{LOA}_g = \left(n_{\mathrm{deep},g} \cdot L_{\mathrm{barge}}\right) + L_{\mathrm{towboat}}, \qquad W_g = n_{\mathrm{wide},g} \cdot B_{\mathrm{barge}} \tag{A.309}$$

Geometric-probability computation (normal CDF form). With $\sigma_g := \mathrm{LOA}_g$ and with the illustrative impact-zone assumption $[y_L, y_R] = [\mu - W_g/2, \mu + W_g/2]$, we obtain:

$$P_G(g) = \Phi\left(\frac{(\mu + W_g/2) - \mu}{\sigma_g}\right) - \Phi\left(\frac{(\mu - W_g/2) - \mu}{\sigma_g}\right) \tag{A.310}$$

$$= \Phi\left(\frac{W_g}{2\sigma_g}\right) - \Phi\left(-\frac{W_g}{2\sigma_g}\right) = 2\Phi\left(\frac{W_g}{2\sigma_g}\right) - 1 \tag{A.311}$$

Numerical results for all evaluation vessel groups (same inference)

Step 8. Determine vessel operating speeds and compute collision forces (P). Select representative vessel operating speeds near the bridge (the report discusses a plausible speed range for the site). Compute collision (impact) loads for each vessel group and loading condition using the AASHTO Guide. Evaluate both:

- Longitudinal (parallel-to-channel) impacts: use 100% of the computed impact force.
- Transverse (perpendicular-to-channel) impacts: use 50% of the computed impact force.

Table A.80: Inferred geometric probabilities $P_G(g)$

Group g	$\sigma_g = \text{LOA}_g$ (ft)	W_g (ft)	$z = \frac{W_g}{2\sigma_g}$	$P_G(g) = 2\Phi(z) - 1$
Hopper 1	$4(195) + 90 = 870$	$3(35) = 105$	0.06034	0.0481
Hopper 2	$3(195) + 90 = 675$	$3(35) = 105$	0.07778	0.0620
Hopper 3	$2(195) + 90 = 480$	$3(35) = 105$	0.10938	0.0871
Tanker 1	$3(290) + 90 = 960$	$2(53) = 106$	0.05521	0.0440
Tanker 2	$2(290) + 90 = 670$	$2(53) = 106$	0.07910	0.0631
Tanker 3	$2(290) + 90 = 670$	$1(53) = 53$	0.03955	0.0315

Step 8. Determine vessel operating speeds and compute collision forces (P). Select representative vessel operating speeds near the bridge, then compute the corresponding equivalent (static) collision forces for each evaluation vessel group using the AASHTO vessel-collision guide procedure.

Step 8a. Select operating speeds V_d and V_u. The report notes that the previous study used an operating speed of $V_d = 12.5$ ft/s for downstream traffic and $V_u = 9.5$ ft/s for upstream traffic, but the current study suggests that a lower vessel speed in the range 8-11 ft/s is representative of typical transit speeds in the bridge vicinity under normal environmental circumstances. As additional context for screening choices: (i) the 2002 I-40 allision involved an estimated tow speed of about 6.7 mph ≈ 9.8 ft/s, and (ii) for high-water drifting conditions the report later references $V \approx 6$ ft/s as a representative speed for minimum-impact-load checks.

Step 8b. Compute head-on (longitudinal) collision force $P^{\parallel}$ for each vessel group. For each vessel group g, compute the equivalent head-on collision force $P_g^{\parallel}(V)$ (units: kips) using the AASHTO guide method. The report presents the resulting head-on collision loads as a function of speed (Figure A.21). The equivalent head-on collision force is represented as

$$P_g^{\parallel}(V) = \mathcal{F}_{\text{AASHTO}}(g, \text{loading}, V) \tag{A.312}$$

where $\mathcal{F}_{\text{AASHTO}}(\cdot)$ denotes the AASHTO guide collision-load model used by the report.

Step 8c. Compute transverse collision force $P^{\perp}$. The report evaluates both impact directions:

$$P_g^{\perp}(V) = 0.5\, P_g^{\parallel}(V) \tag{A.313}$$

i.e., for impacts perpendicular to the navigation-channel centerline, the applied impact force is taken as 50% of the head-on (parallel-to-channel) force.

Step 9. Determine substructure capacities (H) at the design water elevation. Because vessel collision is treated as an extreme event, evaluate pier capacities using survival-oriented ultimate limit-state criteria. Determine nominal global pier capacities in longitudinal and transverse directions, with the collision load applied at the mean high water level (or the 2% flowline) for overall capacity checks (and distributed appropriately for localized checks, as applicable).

Step 9. Determine substructure capacities (H) at the design water elevation. Because vessel collision is treated as an extreme event, the report evaluates pier capacities using AASHTO ultimate-state (survival-oriented) criteria, with a resistance factor of 1.0. Rather than re-computing capacities from first principles within this document, the report adopts the global pier-capacity results from the previous study, defined as

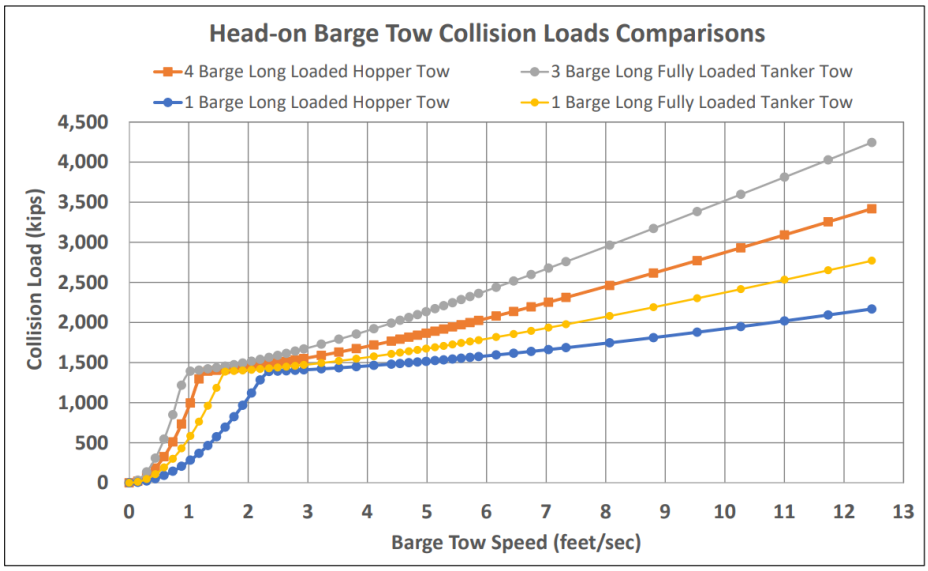


Figure A.21: Barge Collision Loads as a Function of Vessel Speed
Source: [17, Figure 7.5-1]

the controlling maximum concentrated lateral force that can be applied at the **2% flowline** (design water elevation) in each direction. Capacities are reported for two foundation-stiffness bounds ("Lower level" and "Upper level") and for two loading directions:

- Parallel to channel (longitudinal direction), and
- Perpendicular to channel (transverse direction).

The report notes that the previous-study annual frequency results are based on the upper-limit substructure capacities. Accordingly, for reproducing the report's primary computation path, take $H_{i,L}$ and $H_{i,T}$ for pier i from the Upper level columns of Table A.81.

Table A.81: Nominal Substructure Capacities (kips)

Pier	Parallel to Channel		Perpendicular to Channel	
	Lower Level	Upper Level	Lower Level	Upper Level
2	2,685	3,825	1,880	2,670
3	2,280	3,400	1,420	1,530
4	8,175	10,410	1,670	1,710
5	8,125	10,430	1,670	1,700
6	765	1,000	655	670
7	857	1,100	690	720
8	825	1,080	690	700
9	880	1,095	720	740
10	805	1,060	675	680
11	949	1,345	760	785
12	965	1,375	770	770

Source: [17, Table 7.6-1]

Step 10. Compute probability of collapse (P_C). Compute the probability of collapse of a struck substruc-

ture element as a function of vessel characteristics and structural resistance. The report determines P_C based on the ratio of structural capacity H to the static impact force P:

$$P_C = f\left(\frac{H}{P}\right) \tag{A.314}$$

where $f(\cdot)$ follows the Method II collapse-probability model, including vessel size and type, speed, and impact direction. The report presents the resulting annual frequencies of collapse AF: 8.0×10^{-5} for the current case and 1.3×10^{-4} for the 2060 future-traffic case.

Step 11. Compute annual frequency of collapse (AF) for each pier and aggregate. For each exposed pier i, vessel group g, traffic direction d, and impact direction k, compute:

$$AF_{i,g,d,k} = N(i,g,d,k) \cdot P_A(g,d) \cdot P_G(i,g,d,k) \cdot P_C(i,g,d,k) \tag{A.315}$$

and aggregate to the total substructure annual frequency via (A.285). Report results as: (i) per-pier AF contributions and (ii) total AF_{sub} for each traffic year.

Step 12. Compare against bridge classification criteria (acceptability check). Assign a vessel-collision classification ("typical" vs "critical/essential") based on owner-defined criteria (e.g., emergency response needs, defense network importance, detour availability, AADT). Compare the computed AF_{sub} to the acceptable annual frequency threshold for the classification:

$$AF_{\text{sub}} \leq 10^{-3} \text{ (typical)}, \qquad AF_{\text{sub}} \leq 10^{-4} \text{ (critical/essential)} \tag{A.316}$$

Step 13. Iterate mitigation/protection scenarios if needed. If AF_{sub} exceeds the acceptable criterion (or to reduce risk further), modify protection assumptions and/or capacity targets (e.g., adjust minimum impact load assumptions, change pier protection reliance, increase pier capacities) and repeat Steps 5-12 until the acceptability criterion is satisfied.

APPENDIX B. Value of Research

The Value of Research (VoR) for Project 0-7239 / 26-010 is based on the statewide bridge-risk ranking procedures, the GIS Tool, the bridge review and response selection procedures, and the recommended language for TxDOT manuals. The assessment follows the benefit-area and economic-calculation framework in the TxDOT University Handbook. The primary economic value is an infrastructure-preservation benefit represented by one avoided replacement-equivalent expenditure during the five-year value period. The planning scenario assumes that risk-informed inspection, maintenance, protection, and mitigation allow TxDOT to address a high-risk structure before conditions require major replacement work. Staff time saved during recurring district-level bridge-risk prioritization reviews is included as a separate secondary economic benefit.

B.1 Selected Benefit Areas and Definitions

Table B.1 identifies the benefit areas used in this VoR assessment and defines their relationship to the project results.

B.2 Economic Value Inputs and Calculations

The economic assessment quantifies two benefits: infrastructure preservation and staff productivity. Infrastructure preservation is the primary benefit. For the VoR planning scenario, the infrastructure-preservation benefit is set equal to one representative bridge replacement over the five-year value period. The representative bridge is 300 ft long and 40 ft wide, with a deck area of 12,000 ft^2. Applying the TxDOT FY 2025 statewide low-bid average of $135.29 per square foot for new and replaced bridges of all types gives a representative replacement value of $1,623,480 [462]. For the annual VoR calculation, this replacement-equivalent value is allocated evenly across the five-year value period, resulting in an annual infrastructure-preservation benefit of $324,696.

The secondary benefit is staff productivity during recurring district-level risk-prioritization reviews. TxDOT's statewide organization comprises 25 districts. The calculation uses two planning reviews per district per year, representing a midyear review and an annual review. Use of the project procedures and GIS Tool is estimated to save eight staff hours per review by reducing manual data assembly, filtering, comparison, and screening-list preparation.

The loaded labor rate is $65 per hour. The rate is based on the midpoint of the TxDOT 2025 salary range of $69,572–$124,600 per year for a Bridge Inspection Quality Assurance Engineer II, III, or IV. The salary midpoint is $97,086 per year, equivalent to $46.68 per hour using 2,080 work hours per year. Applying a planning factor of 1.40 for salary-related benefits and overhead gives $65.35 per hour, which is rounded to $65 per hour for the VoR calculation.

The five-year expected value duration represents five annual application cycles of the current analytical files, procedures, and GIS Tool before a major update. A 5 percent annual discount rate is used, consistent with the typical rate identified in the TxDOT University Handbook for VoR calculations.

The replacement value of the representative bridge is calculated as

Table B.1: Selected Value of Research benefit areas and project-specific definitions.

Selected Benefit Area	Value Type	Recipient	Project-Specific Definition
Reduced Construction, Operations, and Maintenance Cost	Economic	TxDOT Bridge Division and districts	Quantified as one avoided replacement-equivalent construction expenditure during the five-year value period.
Improved Productivity and Work Efficiency	Economic	TxDOT Bridge Division and districts	Reduces staff time required to assemble risk-related bridge information, compare bridges across the included risk rankings, prepare screening lists, and identify structures for bridge-level review.
Management and Policy	Qualitative	TxDOT Bridge Division and program managers	Provides a statewide procedure for organizing screening information, documenting review decisions, connecting ranking results to follow-up actions, and supporting incorporation of the procedures into TxDOT manuals.
Level of Knowledge	Qualitative	TxDOT engineers, inspectors, and asset-management staff	Consolidates bridge-risk data sources, analytical procedures, statewide findings, historical cases, and mitigation practices into a technical resource supported by bridge-level information in the GIS Tool.
Safety	Qualitative	TxDOT and bridge users	Supports prioritization of bridge-level review using the included risk rankings and the multi-hazard recurrence ranking.
Infrastructure Condition	Qualitative	TxDOT Bridge Division and districts	Supports prioritization of inspection, engineering review, monitoring, maintenance, and mitigation planning based on bridge-level screening results and current conditions.
Engineering Design Improvement	Qualitative	TxDOT bridge designers and reviewers	Provides hazard-specific review procedures, decision trees, response options, and recommended manual language that can inform design, rehabilitation, protection, and related engineering decisions.

$$V_b = L_b W_b U \tag{B.1}$$

where V_b is the representative bridge replacement value, L_b is the representative bridge length, W_b is the representative bridge width, and U is the TxDOT statewide bridge replacement unit cost. The calculation uses $L_b = 300$ ft, $W_b = 40$ ft, and $U = \$135.29$ per square foot:

$$V_b = (300)(40)(\$135.29) = \$1{,}623{,}480 \tag{B.2}$$

The annualized infrastructure-preservation value, E_i, divides the representative bridge value, V_b, by the five-year value duration, n:

$$E_i = \frac{V_b}{n} = \frac{\$1{,}623{,}480}{5} = \$324{,}696 \text{ per year} \tag{B.3}$$

The expected annual staff-productivity value, E_s, uses $D = 25$ TxDOT districts, $C = 2$ review cycles per district per year, $H = 8$ staff hours saved per cycle, and a loaded labor rate of $R = \$65$ per hour:

$$E_s = DCHR = 25(2)(8)(\$65) = \$26{,}000 \text{ per year} \tag{B.4}$$

The combined expected annual value, E, is the sum of the annual infrastructure-preservation value, E_i, and staff-productivity value, E_s:

$$E = E_i + E_s = \$324{,}696 + \$26{,}000 = \$350{,}696 \text{ per year} \tag{B.5}$$

Total Savings before discounting, S, equal the combined annual value, E, multiplied by the five-year duration, n, less the total project budget, $B = \$64{,}961.25$:

$$S = En - B = \$350{,}696(5) - \$64{,}961.25 = \$1{,}688{,}518.75 \tag{B.6}$$

For present-value calculations, the five annual benefits are treated as end-of-year benefits. Their discounted present value is

$$PV_B = E\left[\frac{1-(1+r)^{-n}}{r}\right] \tag{B.7}$$

where PV_B is the discounted present value of benefits, $E = \$350{,}696$ is the combined expected annual value, $r = 0.05$ is the annual discount rate, and $n = 5$ years is the expected value duration. Substitution gives $PV_B = \$1{,}518{,}330.15$. Net present value, NPV, subtracts the project budget, B, from the discounted benefits:

$$NPV = PV_B - B = \$1{,}518{,}330.15 - \$64{,}961.25 = \$1{,}453{,}368.90 \tag{B.8}$$

The simple payback period, P, divides the project budget, B, by the combined expected annual value, E:

$$P = \frac{B}{E} = \frac{\$64{,}961.25}{\$350{,}696} = 0.185 \text{ years} \tag{B.9}$$

The result is approximately 2.22 months. The Cost-Benefit Ratio, BCR, is net present value, NPV, divided by the project budget, B:

$$BCR = \frac{NPV}{B} = \frac{\$1{,}453{,}368.90}{\$64{,}961.25} = 22.37 \tag{B.10}$$

Table B.2: Project information and Value of Research economic inputs and results.

Field	Value
Project Number	0-7239 / 26-010
Project Name	Synthesis: Assess Hazard and Vulnerability Prioritization for Texas Bridges and Tunnels
Performing Agencies	University of Houston and The University of Texas at Austin Center for Transportation Research
Total Project Budget	$64,961.25
Project Duration	1.0 year
Representative Bridge Dimensions	300 ft long by 40 ft wide
TxDOT FY 2025 Bridge Replacement Unit Cost	$135.29 per square foot
Representative Bridge Replacement Value	$1,623,480
Expected Annual Infrastructure-Preservation Value	$324,696 per year
Expected Annual Staff-Productivity Value	$26,000 per year
Combined Expected Annual Value	$350,696 per year
Expected Value Duration	5 years
Discount Rate	5 percent
Total Savings	$1,688,518.75
Present Value of Benefits	$1,518,330.15
Net Present Value	$1,453,368.90
Payback Period	0.185 years (2.22 months)
Cost-Benefit Ratio	22.37

Under this planning scenario, the five-year infrastructure-preservation and staff-productivity benefits have a discounted present value of $1,518,330.15 and exceed the project budget by $1,453,368.90 on a net-present-value basis. Infrastructure preservation contributes $1,623,480 of the $1,753,480 undiscounted five-year gross benefit, while staff productivity contributes $130,000. The infrastructure component therefore ac-

counts for approximately 92.6 percent of the calculated gross benefit. The simple payback period is approximately 2.22 months of recurring statewide use.

B.3 Qualitative Value

The Management and Policy benefit is provided by a statewide framework for organizing bridge-risk screening information and connecting screening results to bridge-level review and follow-up action. Statewide and district staff can use the included risk rankings and recurrence ranking to establish prioritized review lists, examine the bridge information contributing to each result, apply the corresponding review procedures, and document the resulting disposition or action. The recommended TxDOT manual language provides a mechanism for incorporating these procedures into agency practice.

The Level of Knowledge benefit results from consolidating the data sources, analytical procedures, statewide findings, historical cases, and mitigation practices used in the project. The GIS Tool provides bridge-level access to ranking information, supporting attributes, district and statewide views, and relevant GIS layers. These products provide TxDOT staff with a common technical resource for interpreting the screening results and conducting subsequent bridge review.

The Safety benefit is associated with prioritizing engineering attention using the included risk rankings and the multi-hazard recurrence ranking. These screening products establish an ordered basis for prioritizing bridge-level review.

The Infrastructure Condition benefit is associated with using the screening results to focus inspection, condition review, monitoring, maintenance, and mitigation planning. Bridge-level review allows staff to confirm current conditions and determine whether the factors contributing to a ranking indicate a need for data correction, additional evaluation, maintenance, or another response.

The Engineering Design Improvement benefit is provided through the risk-specific review procedures, decision trees, response options, and recommended TxDOT manual language. These products organize the project findings into a sequence of bridge-level checks and potential responses that can inform design, rehabilitation, protection, and related engineering decisions.

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