DESIGNING TWIN TUB GIRDER BRIDGES TO BE NON-FRACTURE CRITICAL

Twin Tub Task Group (T3)
Texas Steel Quality Council
Texas Steel Quality Council

- Texas Steel Quality Council (TSQC)
  - Established 1995
  - Joint owner-industry forum
  - Meets annually

- Twin Tub Task Group
  - Established in September 2016
  - 17 Members from TSQC – TxDOT designers, inspectors, consultants, Fabricators, Researchers
  - National Steel Bridge Alliance
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BACKGROUND
Background

- Twin Tub Girders - All two girder bridges are defined as Fracture Critical (FC).
- June 2012 FHWA Memorandum, “Clarification of Requirements for Fracture Critical Members”:
  - Introduced the new concept of System Redundancy Member (SRM)
  - New member classification: a member that requires fabrication according to the AWS FCP, but need not be considered a FCM for in-service inspection.
  - SRMs should be designated on the design plans
TWIN TUB TASK
GROUP GOAL
Twin Tub Task Group Goal

- Develop LRFD-based design specifications which will allow twin tub girder bridges to be designed as SRM’s.
- Scope includes loads, analysis, design, detailing, materials, fabrication and construction components
- Results can be incorporated into either AASHTO LRFD Bridge Design Specifications, AASHTO Guide Specification or adopt requirements as part of TxDOT Bridge Design Manual
- Receive FHWA concurrence with the final recommendations & deliverable
PRECEDENTS
Precedents

General Services Administration (GSA) – Buildings

- Alternate Path Analysis & Design Guidelines for Progressive Collapse Resistance (October 2013)
- Progressive Collapse: defined as an extent of damage or collapse that is disproportionate to the magnitude of the initiating event
- Extreme Event
Precedents

General Services Administration (GSA) – Buildings

- “...these Guidelines aim to reduce the potential for progressive collapse by bridging over the loss of a structural element, limiting the extent of damage to a localized area (Alternate Path) and providing a redundant and balanced structural system along the height of the building.”
General Services Administration (GSA) – Buildings

“The intent of these requirements is to distribute progressive collapse resistance up the height of the building without explicitly requiring column/wall removal scenarios at each level.”
Precedents

General Services Administration (GSA) – Buildings

- Allows for 3 levels of analysis procedures
  - Linear Static (LSP)
  - Nonlinear Static (NSP)
  - Nonlinear Dynamic (NDP)

- Dependent on facility risk assessment
Precedents

FHWA Guidelines for Cable-Stayed and Arch Bridges

- Extreme Event
- Redundancy Required (Section B):
  - “Long span bridges shall be designed such that controlled or sudden loss of a stay cable, individual hanger, or other structural element, will not cause collapse of the structure”
  - “Tie girders...shall be designed for redundancy...may be accomplished through internal redundancy, external redundancy, or some combination of the two.”
FHWA Guidelines for Cable-Stayed and Arch Bridges

- **Load Combinations**
  - 8.1.3.2 Loss of Structural Element
    \[1.25DC + 1.5DW + 1.3 \text{(LL + IM)}\]
  - 8.1.3.3 Loss of Stay Cable or Suspender
    \[1.1DC + 1.35DW + 0.75(\text{LL} \ast + \text{IM}) + 1.1(\text{cable loss or suspender dynamic forces})\]
    * Full LL placed in their actual striped lanes
  - 8.5 Loss of Structural Elements
    1.5 minimum dynamic impact factor, regardless of analysis
    Minimum cable loss force = 1.5 x static force in cable from DL + LL
AASHTO LRFD for Cable-Stayed and Arch Bridges

- Analysis requirements
  - AASHTO 4.6.3.7
  - Cable stay bridges shall be investigated for the loss of any one cable stay.
  - Not prescriptive on analysis “means and methods”
Pennsylvania DOT Design Manual (April 2015)

- Section 3.4.1 Load Factors and Combinations
  - Extreme Event
  - Part of the Manual since 1996
  - Both are uncalibrated load combinations
  - Intended to force consideration of the safety of damaged structures

- EXTREME EVENT III - Load combination relating to the failure of one element of a component without the failure of the component.

- EXTREME EVENT IV - Load combination relating to the failure of one component without the collapse of the structure.

System Redundancy
Pennsylvania DOT Design Manual (April 2015)

Section 3.4.1 Load Factors and Combinations

| Load Combination | DC | DD | DW | EH | EV | ES | PS | CR | SH | LL | IM | CE | BR | PL | LS | WA | WS | WL | FR | TU | TG | SE |
|------------------|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|----|
| EXTREME EVENT III | $\gamma_p$ | $\gamma'_LL$ | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |
| EXTREME EVENT IV  | $\gamma'_p$ | $\gamma'_LL$ | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - | - |

Table 3.4.1-3 – Load Factor for Live Load for Extreme III and IV, $\gamma'_LL$

<table>
<thead>
<tr>
<th>Case</th>
<th>III $\gamma'_LL$</th>
<th>IV $\gamma'_LL$</th>
</tr>
</thead>
<tbody>
<tr>
<td>PHL-93 Loading – all applicable lanes</td>
<td>1.30</td>
<td>1.15</td>
</tr>
<tr>
<td>Permit load in governing lane with PHL-93 in other applicable lanes</td>
<td>1.10</td>
<td>1.05</td>
</tr>
</tbody>
</table>

Table 3.4.1-4 – Load Factors for Permanent Loads for Extreme Event IV, $\gamma'_p$

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Load Factor</th>
<th>Maximum</th>
<th>Minimum</th>
</tr>
</thead>
<tbody>
<tr>
<td>DC: Component and Attachments</td>
<td>1.05</td>
<td>0.95</td>
<td></td>
</tr>
<tr>
<td>DW: Wearing Surfaces and Utilities</td>
<td>1.05</td>
<td>0.90</td>
<td></td>
</tr>
</tbody>
</table>
Pennsylvania DOT Design Manual (April 2015)

- **Section 1.3.4 Redundancy [Two-girder Bridges]**
  - Two girders can only be used if designed for redundancy – alternate load paths
  - Approval of analysis is required (from Chief Bridge Engineer and FHWA)
  - 3D Analysis requirement - linear analysis

A two-girder bridge shall be used, only if (1) fracture-critical members are eliminated by developing alternative load paths and (2) approval of the 3-D analysis methodology is obtained from the Chief Bridge Engineer and FHWA Office of Bridge Technology. The designer should evaluate the ability of secondary members to transfer loads and prevent collapse through the use of suitable computer analysis. Each girder of a two-girder bridge shall meet the requirements above for a stitched built-up component.

A two-girder bridge has the potential for fracture-critical members, i.e., a failure of one girder appears to cause collapse of the entire structure. There are certain situations, however, in which use of two girders may provide a significant economy.

For both two- and three-girder systems, computer-aided designs have been completed where secondary members are designed to transfer load around a failed portion of a girder, thereby preventing a collapse of the bridge.
Pennsylvania DOT Design Manual (April 2015)

- FHWA has already approved a newly designed bridge to contain SRM’s using an Extreme Event as the load case to analyze redundancy.
- Members are designated on the plans as SRM’s and fabricated according to AWS FCP.
RESEARCH BASIS
Research Basis

Modeling and Response of Fracture Critical Steel Box-Girder Bridges  TxDOT Project 0-5498 (2010)

- University of Texas at Austin
- Characterize the redundancy that exists in twin tub girder bridges
- Developed guidelines for modeling behavior in the event of a fracture of the tension flange
- Full-scale test with fractured bottom flange and web of outside girder
Research Basis

Modeling and Response of Fracture Critical Steel Box-Girder Bridges TxDOT Project 0-5498 (2010)

- Full scale test
- Simulated Worst Case
- Simple Span
- 2 tub girders
- No external intermediate diaphragms
- Horizontally curved
- Simulated HS20 over fracture
Research Basis

1st Test – 5498 Project

- Linear shape-charge explosive
- Rapidly cut bottom flange
- Combination of analytical and full scale testing
- Equivalent of an HS-20 truck positioned directly above the fracture at the most severe location
- The bridge deflected less than 1 inch
Research Basis

2nd Test – 5498 Project

- Exterior Girder - full-depth fracture
- Induced and applied loads suddenly released through the use of explosive acting on a temporary support system
- Bridge performed extremely well
- Fractured girder deflected only 7 inches
- Even in its damaged state, bridge could support traffic and did not collapse
Research Basis

3rd and Final Test – 5498 Project

- Conducted under statically applied loads
- Demonstrated that the bridge tested in the study was able to carry 363,000 lbs
- More than 5 times greater than a legal truck load
Research Basis

TxDOT Project 0-5498 (2010)

- Recommended, or allowed for, 3 levels of analysis
  - Simplified method
  - Yield line method
  - 3D FEA
- Clearly demonstrated system redundancy for a span that was not specifically designed to have system redundancy
- Highlighted the importance of the deck and shear connectors and their contribution to system performance
- Barrier railing was also observed to contribute structurally
STRENGTH VS. EXTREME EVENT LIMIT STATE
**Strength vs. Extreme Event Limit States**

**STRENGTH**
- Event is expected in life of structure (75 years)
- Reliability Index = 3.5

**EXTREME EVENT**
- Return period of event far exceeds expected life of bridge
- Not calibrated for a reliability Index in AASHTO LRFD
- There has been some research in this area, but not in LRFD
Strength vs. Extreme Event Limit States

Steel Bridge Design Handbook: Limit States
Author: Dennis Mertz

- Section 6.0, Extreme Event Limit States
  - “[Extreme Event] limit states represent loads or events of such great magnitude that to design for the levels of reliability or failure rates of the strength limit states would be economically prohibitive.”
PROPOSED AGENDA
ITEM FOR DESIGN
Proposed Agenda Item for Design

1. Design bridge as normally done
2. Design bridge for member failure under Extreme Event, $\phi = 1.0$
1. Design bridge as normally done for the following limit states:

- **Strength**
  - Use Redundancy Factor, $\eta_R = 1.05$

- **Service**

- **Fatigue & Fracture**
  - Infinite Fatigue Life
2. Design bridge for member failure under Extreme Event

- $\phi$, Resistance Factor = 1.0, for Extreme Event
- Modified load factor for LL under Extreme Event II (1.0 for LL)
- Modified load factor for DC (1.1 for DC)
2. Design bridge for member failure under Extreme Event

- Multiple presence does not apply – otherwise 1.2 for one lane
- LL placed in striped traffic lane(s); none in striped shoulder
- LL set to HL-93 (truck or tandem + lane)
- LL is notional – meant to capture an envelope
- IM zeroed out for fracture event
2. Design bridge for member failure under Extreme Event

- Allows for Simplified Strength check analysis
  - Conservative
  - Surviving girder carries full dead and live load
- Refined method, but with exceptions
- Points the designer to check simple spans and end spans of continuous units
 Proposed Agenda Item for Design

2. Design bridge for member failure under Extreme Event

- Dynamic Increase Factor (DIF) set to 1.2 minimum
  - Commentary on strain rate
  - Strain rate can compensate for DIF, but probably only up to 10%

- Uses same strength equations as Strength Limit State
  - Flexure
  - Shear/Torsion

NEW SECTION 6.11.12
2. Design bridge for member failure under Extreme Event

- Deck/stud connectors treated differently
  - Refer to Research Project 0-5498
  - AASHTO Article 6.16 (Provisions for seismic design - interaction equation)
  - Deck performance likely the key item in this design process
2. Design bridge for member failure under Extreme Event

NEW SECTION 6.11.12

- No checks required for Deformations or Deflections
  - Precedent set with cable-stayed/arch
  - Not included in the definition of Fracture Critical
2. Design bridge for member failure under Extreme Event

- **Railing** - use a structurally continuous rail
- **HPS** - encourage use
- **Drain Holes** - bottom flange (Cat D) omit 20ft either side of max moment location
- **Shear Studs** – require to extend above bottom mat of reinf
- **Deck Design** – Do not allow the use of empirical deck
2. Design bridge for member failure under Extreme Event

- **Deck Construction** – do not allow the use of precast concrete panels (sub deck panels)

- **Diaphragms** - Require 2 “full-depth” diaphragms bracketing max moment location if slab is not designed to support fractured girder

- **Details** - Restrict details within 20ft of maximum moment location to Cat C‘; this does not include secondary members, e.g. internal cross-frames
2. Design bridge for member failure under Extreme Event

**Substructure**

- Strength and stability must be adequate to support the superstructure during fracture and post-fracture
- Specify the use of anchor bolts or shear keys as required to keep superstructure on the substructure
- Do not require bearings to meet rotation and compression requirements due to fracture
2. Design bridge for member failure under Extreme Event

   - Plans – Identify tension flanges and webs as System Redundant Members (SRMs) as defined in the FHWA June 2012 Memo
Proposed Agenda Item for Design

2. Design bridge for member failure under Extreme Event

- Fabrication – fabricate SRMs according to the Fracture Control Plan (FCP – AWS D1.5)
  - Base Metal Requirements
  - Welding Processes and Procedures
  - Certification, Qualification, Inspection
  - Straightening, Curving and Cambering
2. Design bridge for member failure under Extreme Event

- **Construction stage** – evaluate NCRs critically (commentary)
- Stud connector penetration above bottom mat should be verified prior to deck placement, etc.
## Proposed Agenda Item for Design

### Load Factors

<table>
<thead>
<tr>
<th></th>
<th>PennDOT Extreme Event IV</th>
<th>TSQC Extreme Event II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Redundancy Factor, $n_R$</td>
<td>1.0</td>
<td>1.00</td>
</tr>
<tr>
<td>DC: Dead Load</td>
<td>1.05 (0.95 min)</td>
<td>1.10 (0.9 min)</td>
</tr>
<tr>
<td>DW: Dead Load</td>
<td>1.05 (0.90 min)</td>
<td>1.5</td>
</tr>
<tr>
<td>Live Load</td>
<td>1.15</td>
<td>1.0</td>
</tr>
<tr>
<td>Dynamic Increase Factor</td>
<td>None</td>
<td>1.2 (min); 1.3 (max)</td>
</tr>
</tbody>
</table>

### Dynamic Increase Factor

- None
- 1.2 (min); 1.3 (max)

### Comparison

<table>
<thead>
<tr>
<th></th>
<th>PennDOT Extreme Event IV</th>
<th>TSQC Extreme Event II</th>
</tr>
</thead>
<tbody>
<tr>
<td>Application of Live Load</td>
<td>Placed in design lanes</td>
<td>Placed in striped lanes</td>
</tr>
<tr>
<td>Level of Analysis</td>
<td>Linear 3D analysis</td>
<td>Allows for simplified conservative analysis and Linear 3D analysis</td>
</tr>
<tr>
<td>LL</td>
<td>PHL-93</td>
<td>HL-93</td>
</tr>
<tr>
<td>Impact</td>
<td>IM = 33%</td>
<td>IM = 0</td>
</tr>
<tr>
<td>Deformations/Deflections</td>
<td>No checks required – objective of analysis is survival of the bridge</td>
<td>No checks required</td>
</tr>
<tr>
<td>Calibration</td>
<td>Uncalibrated – intended to force consideration of damaged structures</td>
<td>Uncalibrated</td>
</tr>
</tbody>
</table>
Proposed Agenda Item for Design

Where we are:

- Minor revisions are needed in order to update the proposed Agenda Item

- Fracture Critical Workshop (NSBA) on May 1st
  - Load factors were discussed – more work is slated to be done
  - Dynamic Increase Factor (DIF) – discussions were in the range of 1.2 to 1.3

- Take two examples and analyze them with the TSQC approach to see if it controls the design (if so, how much)
EXPECTED EFFECT ON BRIDGES
Expected Effect on Bridges

Inspections

- Decrease number of Fracture Critical Inspections
  - Hands on inspections will not be required
  - Increase safety – most inspections are done at night
  - Decrease disruption to traffic

- 2 year Routine Inspections will continue
FUTURE APPLICATIONS
Future Applications

- Other members currently identified as Fracture Critical
  - Steel box Straddle bents
Acknowledgements

- John Holt, HDR
- Greg Turco, TxDOT Bridge Division
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- Texas Steel Quality Council
- Texas Twin Tub Task Group Members
- National Steel Bridge Alliance
- Tom Macioce, Pennsylvania DOT
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