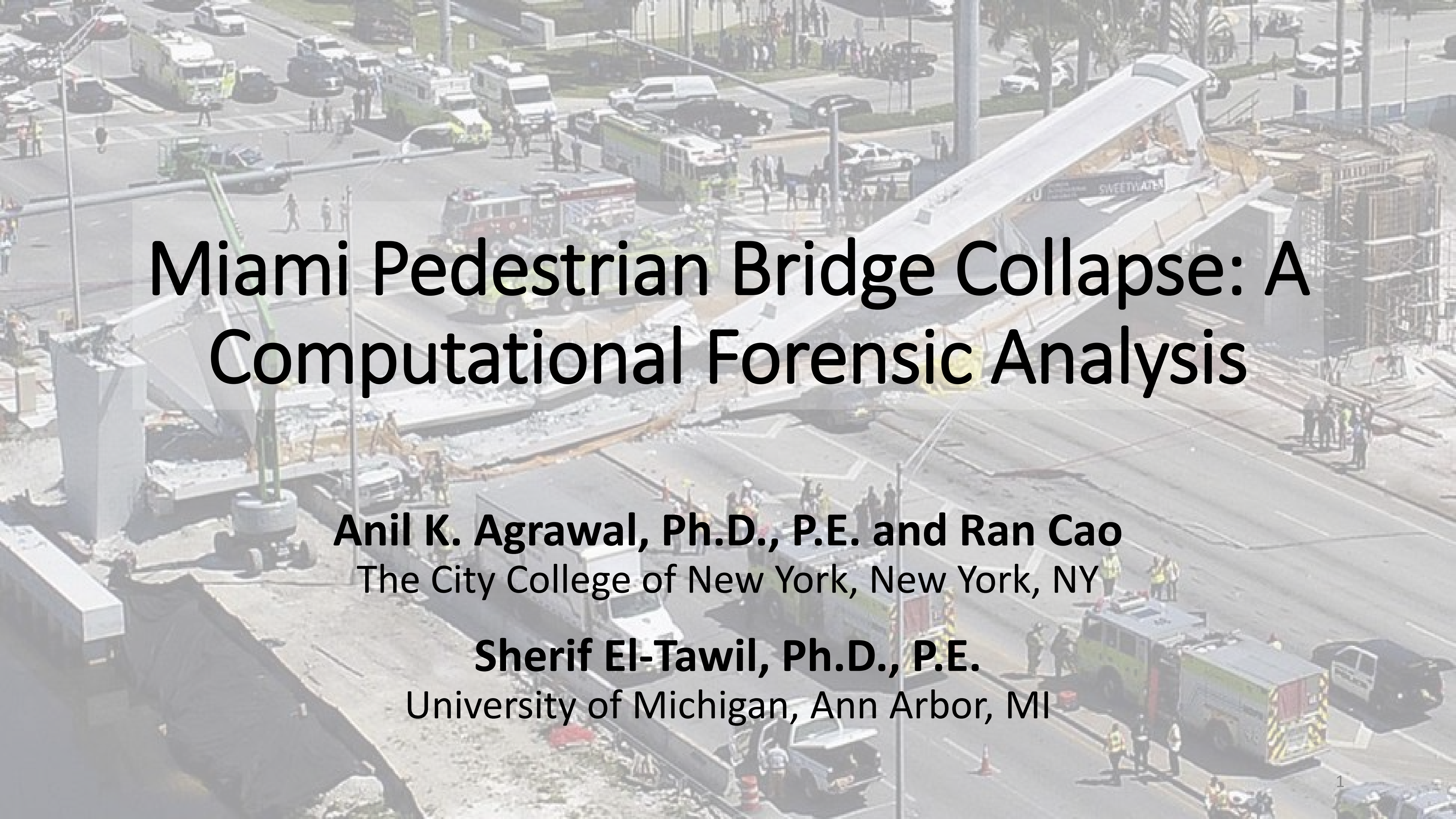


Miami Pedestrian Bridge Collapse: A Computational Forensic Analysis

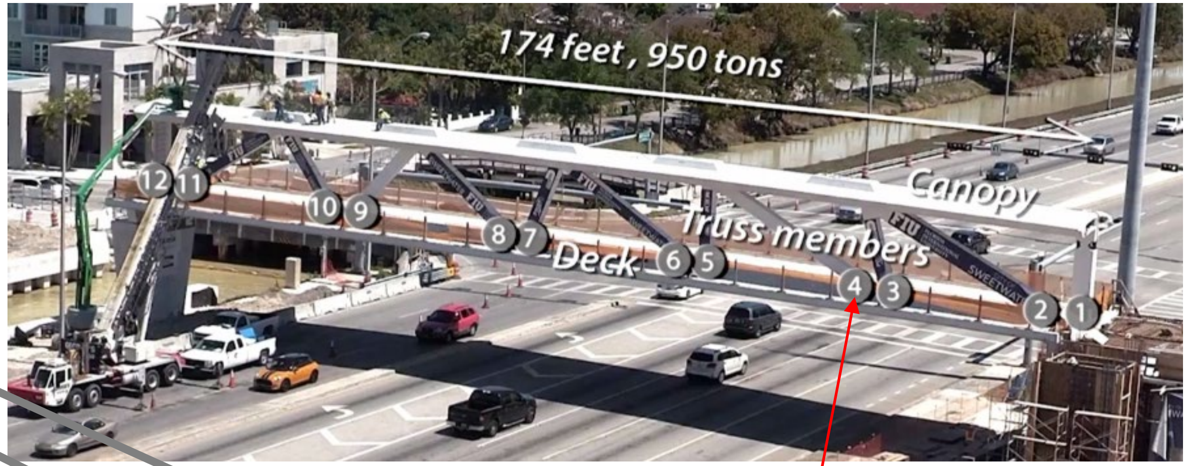
Anil K. Agrawal, Ph.D., P.E. and Ran Cao
The City College of New York, New York, NY

Sherif El-Tawil, Ph.D., P.E.
University of Michigan, Ann Arbor, MI

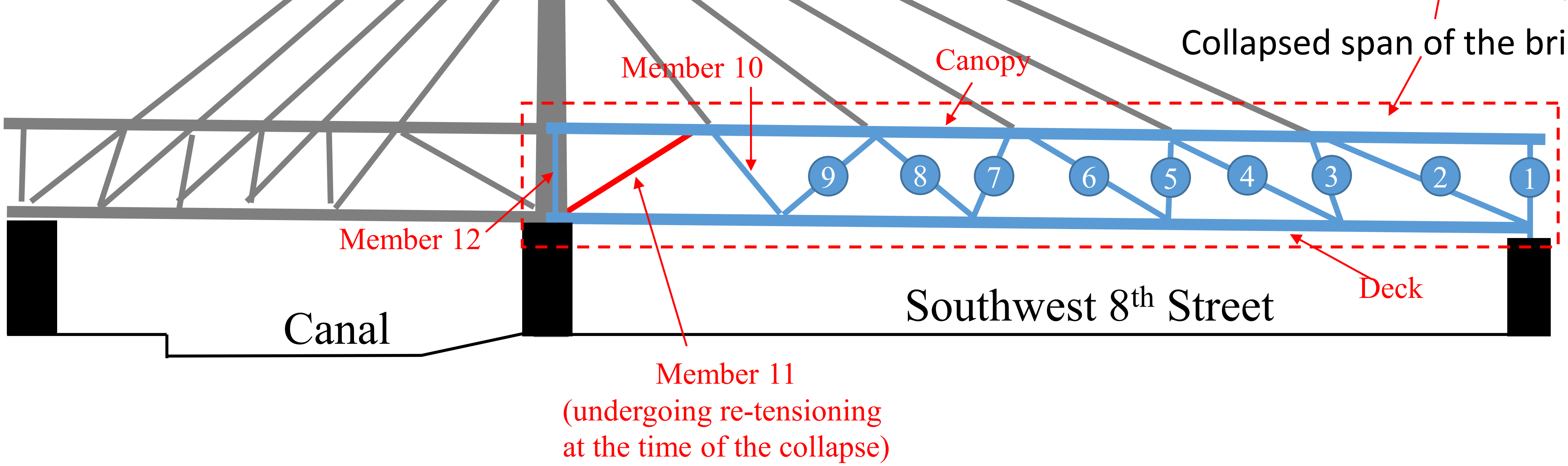
https://www.youtube.com/watch?v=p6L20I6_gzE

Introduction

- Rendering of the FIU Pedestrian Bridge



Source: NTSB

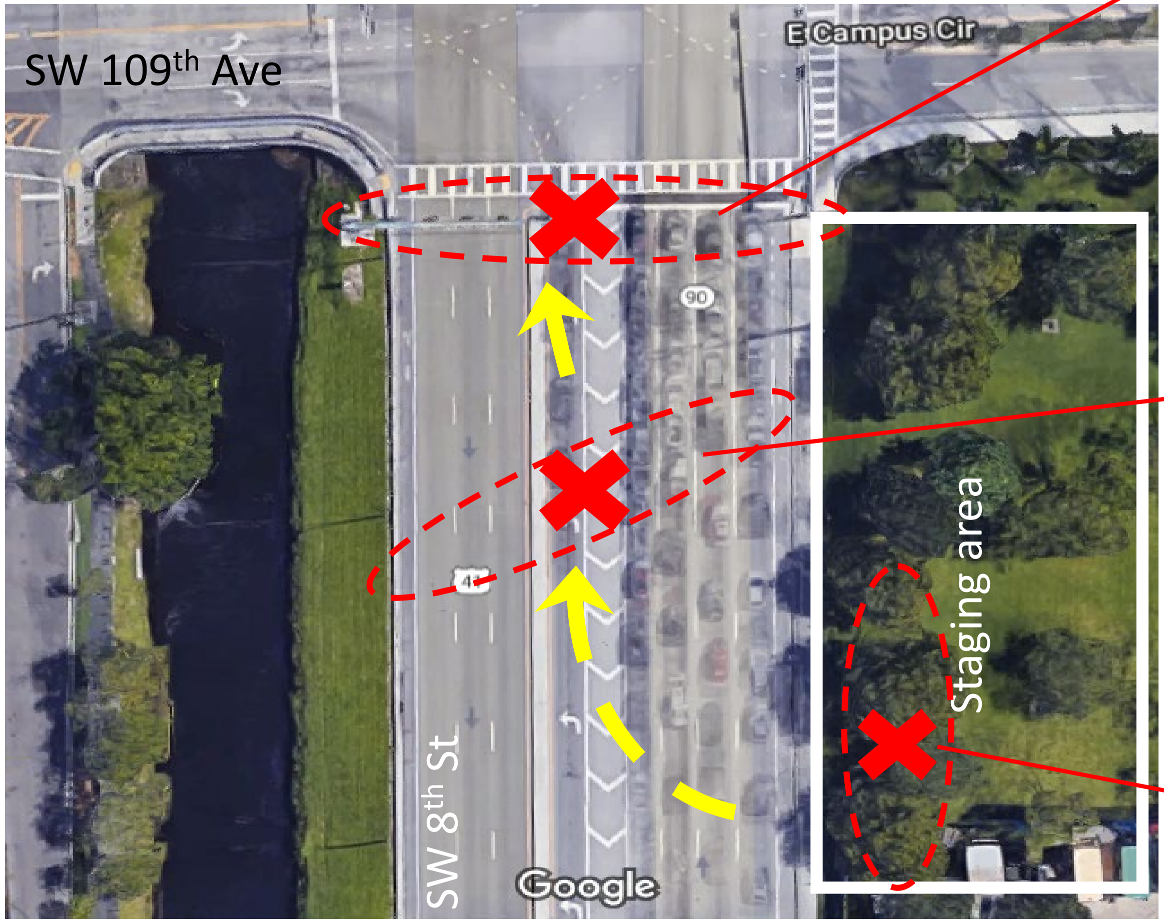


Accelerated Bridge Construction

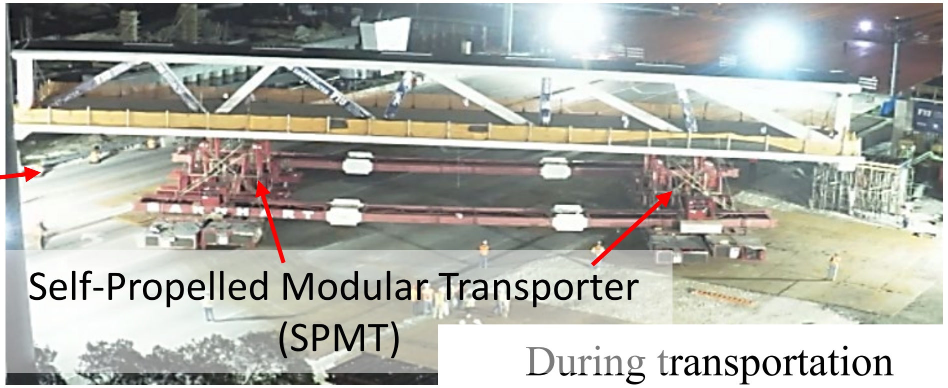
- Method that uses innovative planning, design, materials, and **construction** techniques in a safe and cost-effective manner to reduce the onsite **construction** time that occurs when building new **bridges** or replacing and rehabilitating existing **bridges**.
- Here the main objective is to reduce impact on mobility because of bridge reconstructions in urban areas.
- The bridge was being constructed through ABC to showcase the ABC method itself.

Introduction

- Transportation of the Main Span



After transportation
(before collapse)



During transportation

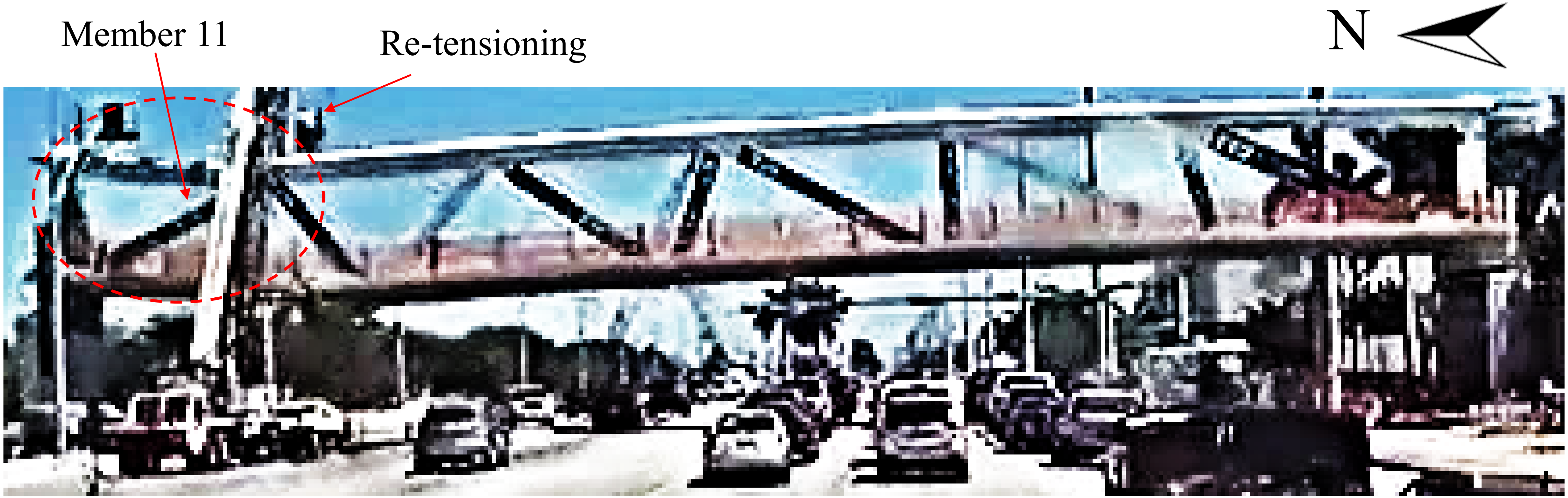
Before transportation

Introduction

Collapse Scene (March 15, 2018)

Introduction

- Failure initialized



NTSB Investigation Update – 10/22/2019

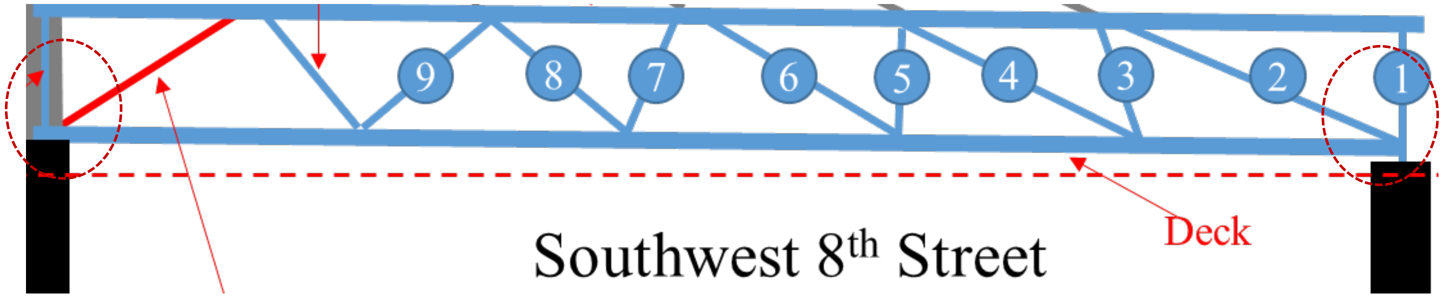
“Errors in bridge design, inadequate peer review and poor engineering judgment led to the collapse of this bridge.”

-- NTSB Chairman Robert Sumwalt

NTSB Investigation Update – 10/22/2019

➢ **Bridge design error/concerns**:

1. FIU bridge designers (1) made significant design errors in the determination of loads, leading to a severe underestimation of the demands placed on critical portions of the pedestrian bridge; and (2) significantly overestimated the capacity of the member 1/2 and 11/12 nodal regions.



NTSB Investigation Update – 10/22/2019

- **Bridge design error/concerns**:

2. The cold joint interface was not roughened.
3. Member 11/12 nodal region contained pipe sleeves, resulting in void spaces in the concrete member.
4. Re-tensioning member 11 provided additional shear force across the interface of the cold-joint, resulting in collapse of the bridge.
5. The bridge design was non-redundant because it provided only a singular load path.

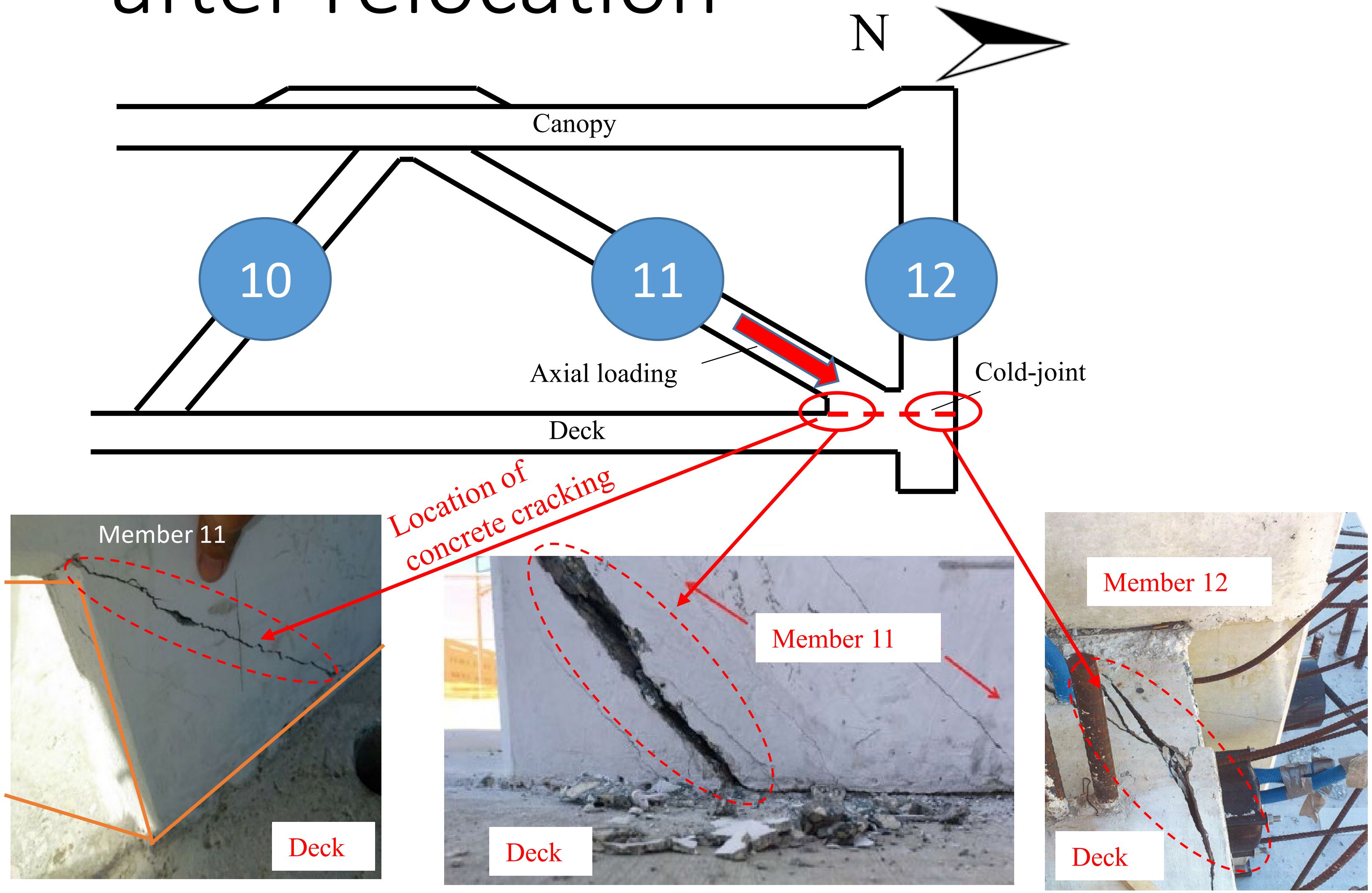
NTSB Investigation Update – 10/22/2019

- Inadequate Peer Review: the company reviewing the drawings was not qualified by the Florida Department of Transportation (inadequate experience, number of PE staff).
- To address the structural cracking, the remediation plan (re-tensioning) was not independently reviewed before being implemented.
  - Re-tensioning was not in the original plan.
  - The road underneath the bridge <u>was not </u>closed during the re-tensioning.

Possible Points of Investigation or Concerns in the Bridge Design

1. Presence of structural cracks in critical concrete members.
2. Presence of cold-joints.
3. Horizontal shear demand at the cold-joint.
4. Punching shear damage.
5. Reliance on the friction for resisting shear demand.

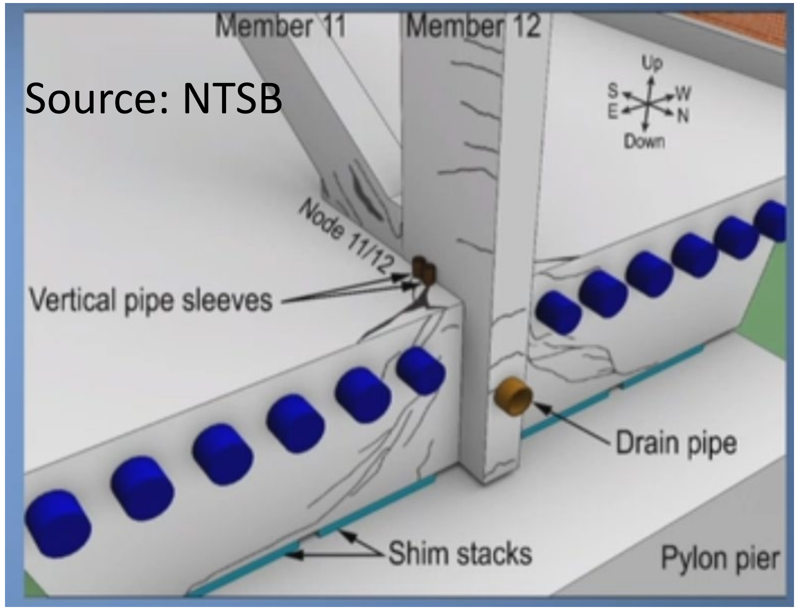
Damage mode of the cold-joint before and after relocation



Before relocation

After relocation

After relocation



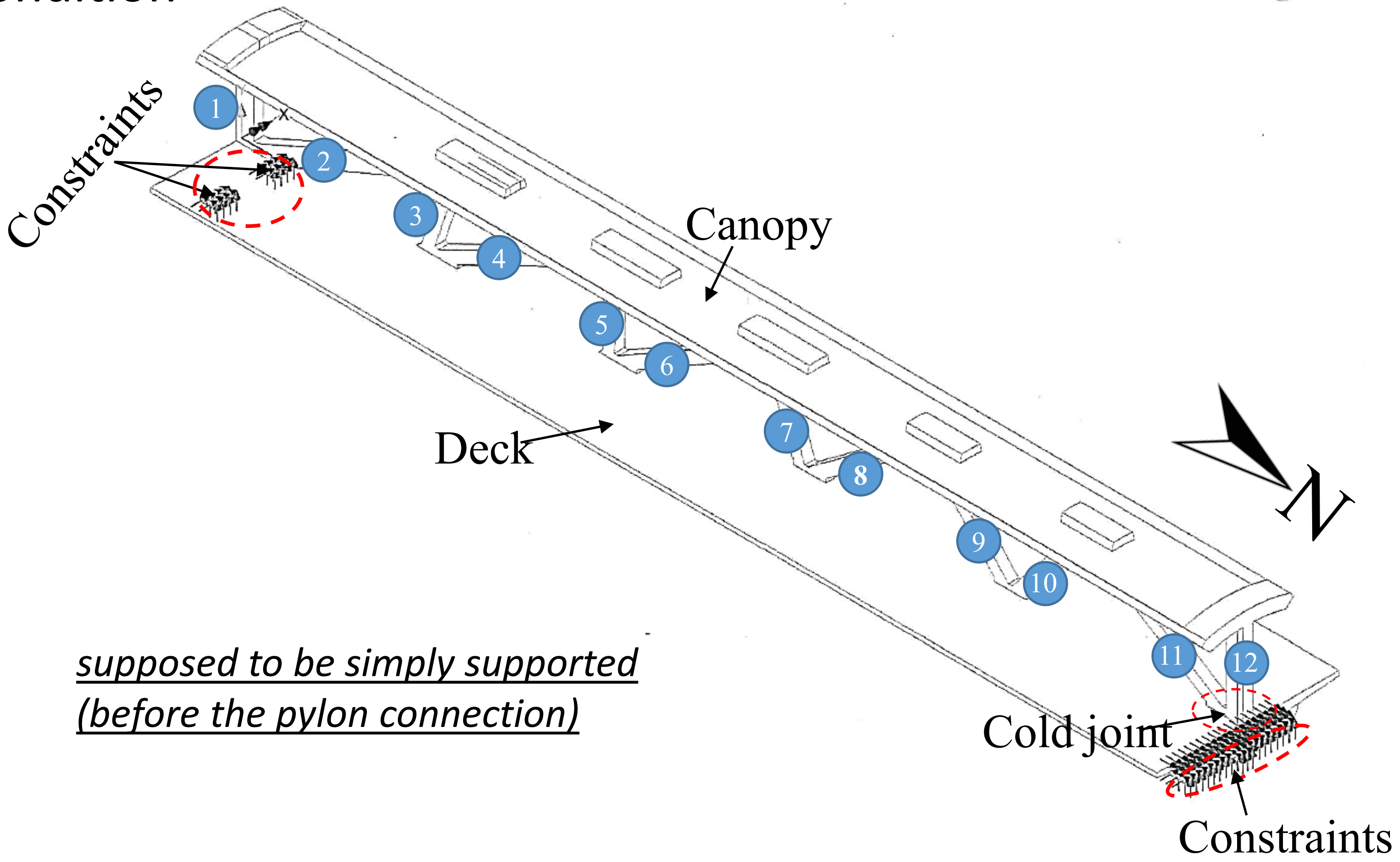
Structural cracks in the joint area were up to 0.750 inch wide, which is **40 times wider** than typically acceptable width of the crack (0.016 inch) (NTSB report)

(reprinted from NTSB report 2018)

Critical Review of Bridge Design Calculations

Superstructure Design Provided by FIU

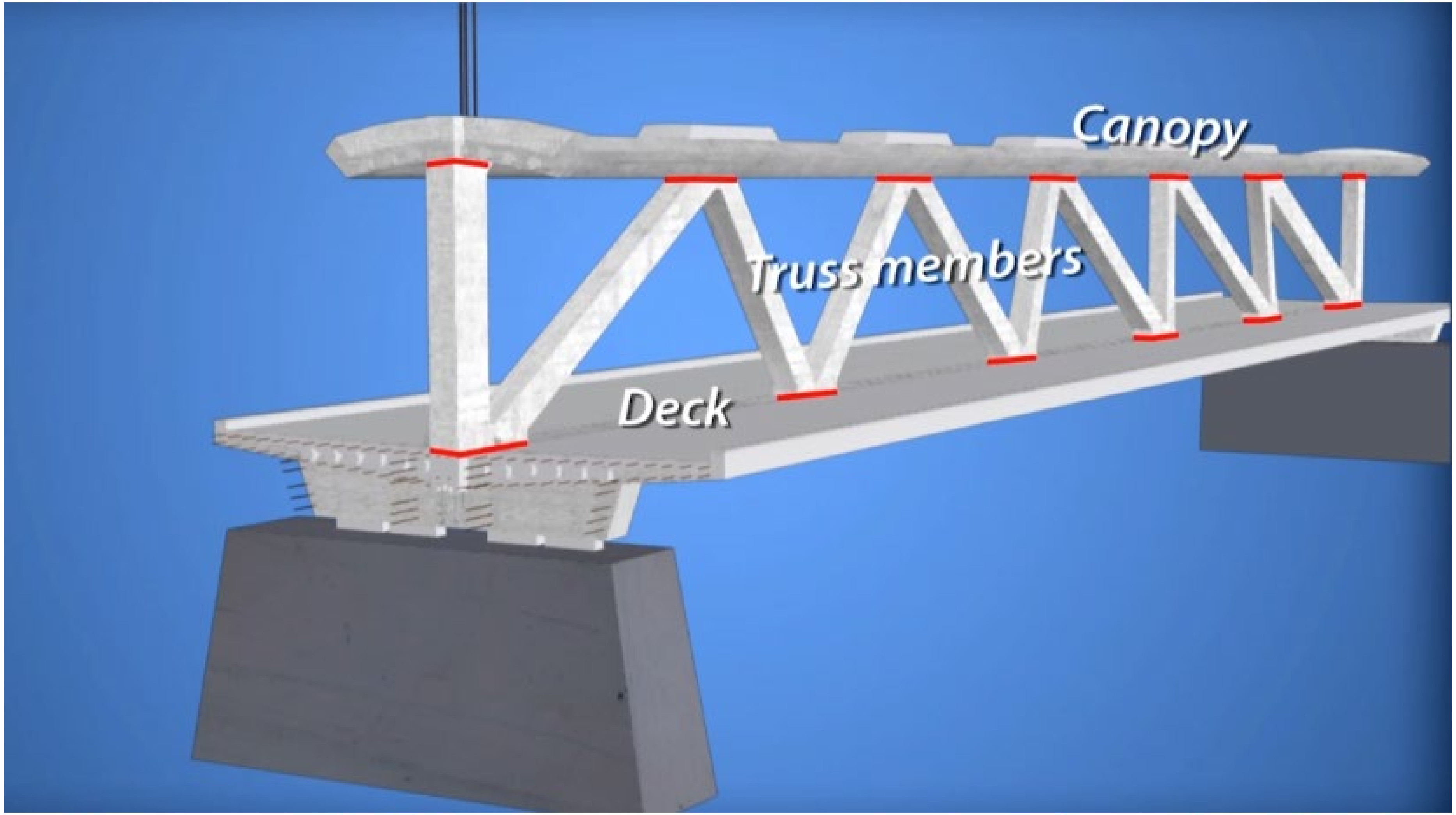
- Boundary condition



supposed to be simply supported
(before the pylon connection)

Critical Review of Bridge Design Calculations

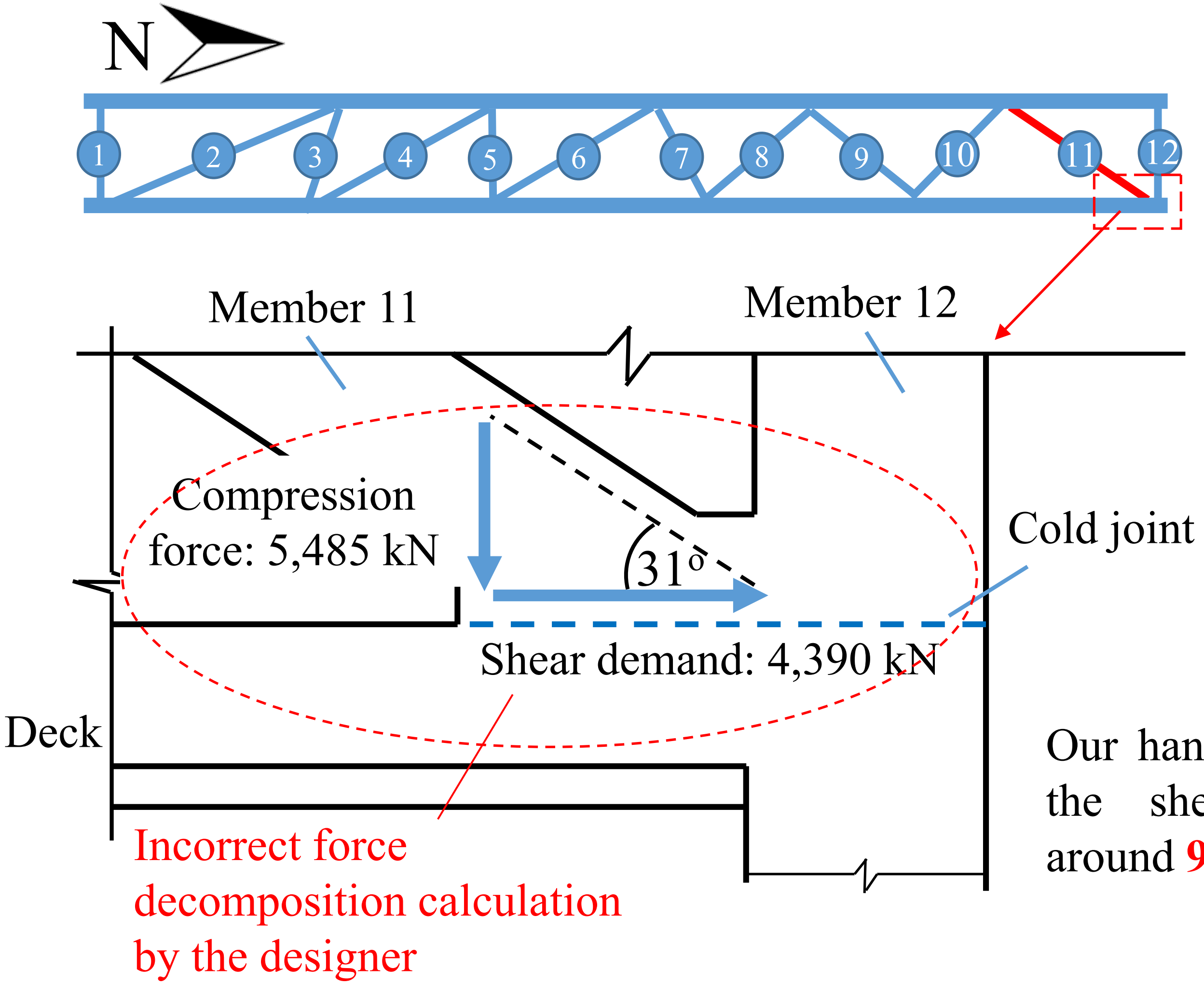
Cold-joint locations (separate concrete cast)



Source: NTSB

Critical Review of Bridge Design Calculations

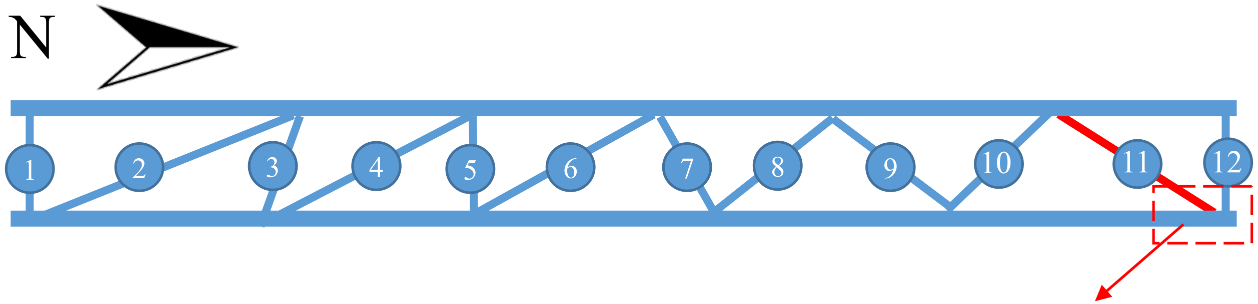
Interface Shear Design of the Northern End Zone by the Designer

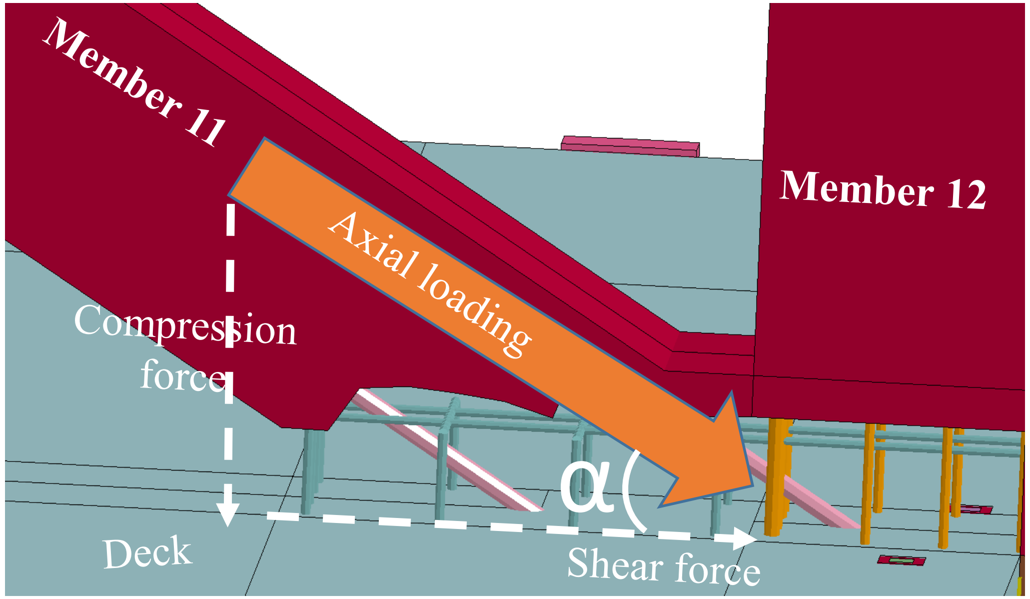


Our hand calculation showed that the shear demand should be around **9,168 kN.**

Critical Review of Bridge Design Calculations

Shear Capacity of the Construction Joint





Frictional capacity

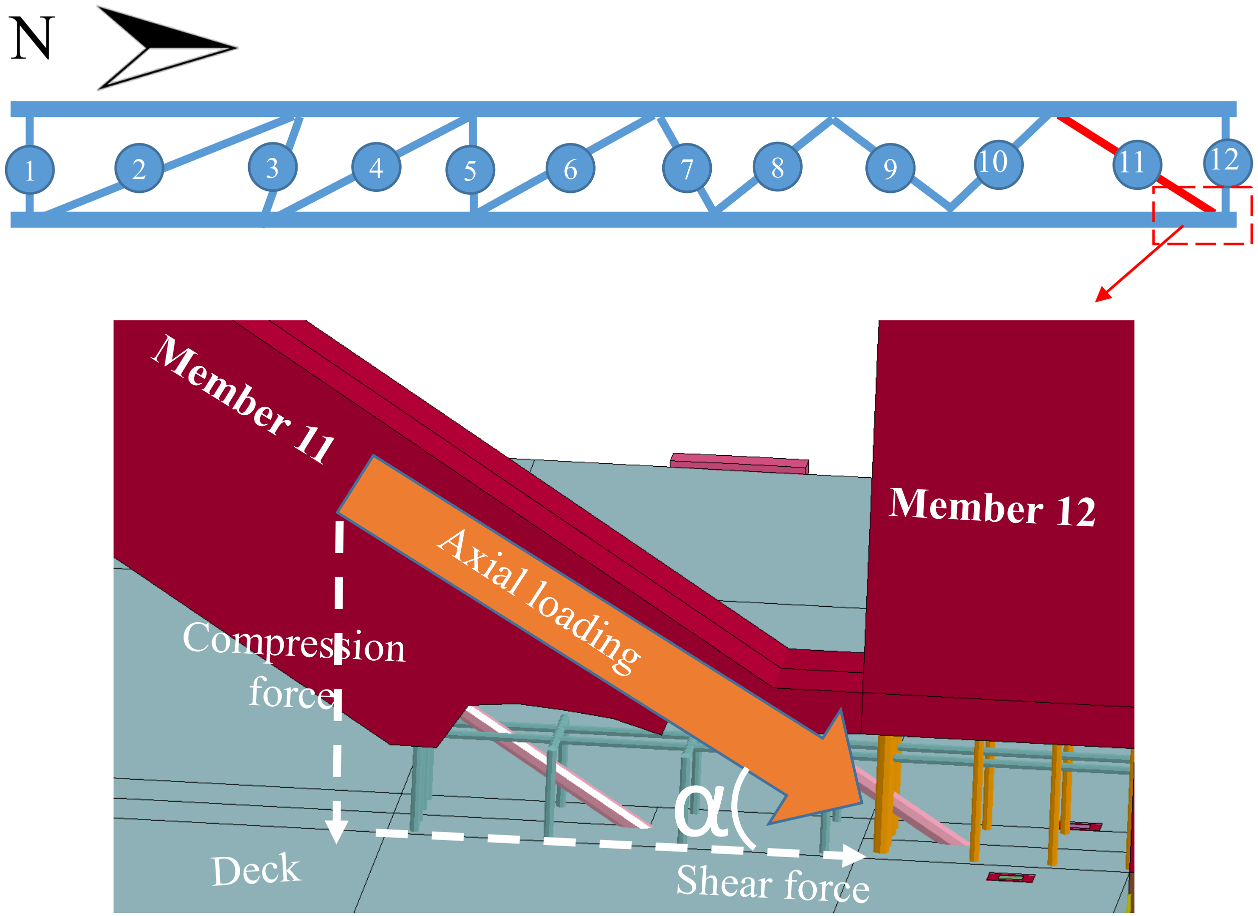
$$f = \mu(F_p + F_G)\sin\alpha$$

$$= 1.0 \times (2491 + 8205) \times \sin(31^o) = 5509\ kN$$

- F_p is the pre-stressing force in member 11,
- F_G is the axial force in member 11 due to gravity (self-weight)
- α is the inclination angle of member 11 with the horizontal surface.

Critical Review of Bridge Design Calculations

Shear Capacity of the Construction Joint



Shear strength of anchors

$$Q_n = 0.5 A_{sa}\sqrt{f'_c E_c}$$

$$= 0.5 \times 10012 \times \sqrt{58 \times 36233} = 7257\ kN$$

$$> R_g R_p A_{sa} F_u = 1.0 \times 0.75 \times 10012\ mm^2 \times 414\ Mpa$$

$$= 3109\ kN$$

$$\rightarrow Q_n = 3109\ kN$$

$$C = f + Q_n = 5509\ kN + 3109\ kN = 8618\ kN$$

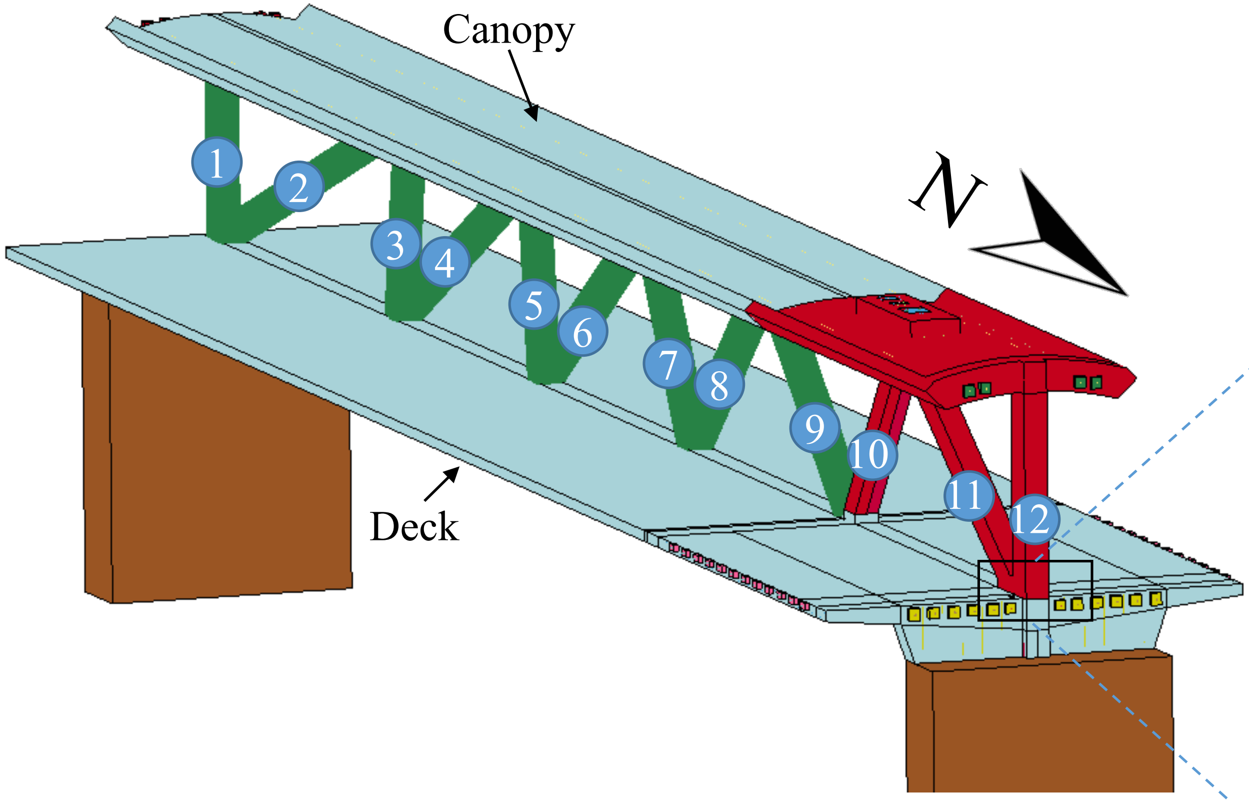
$$D = (F_G + F_P) \times \cos(\alpha) = (8205 + 2491) \times \cos(31^o)$$

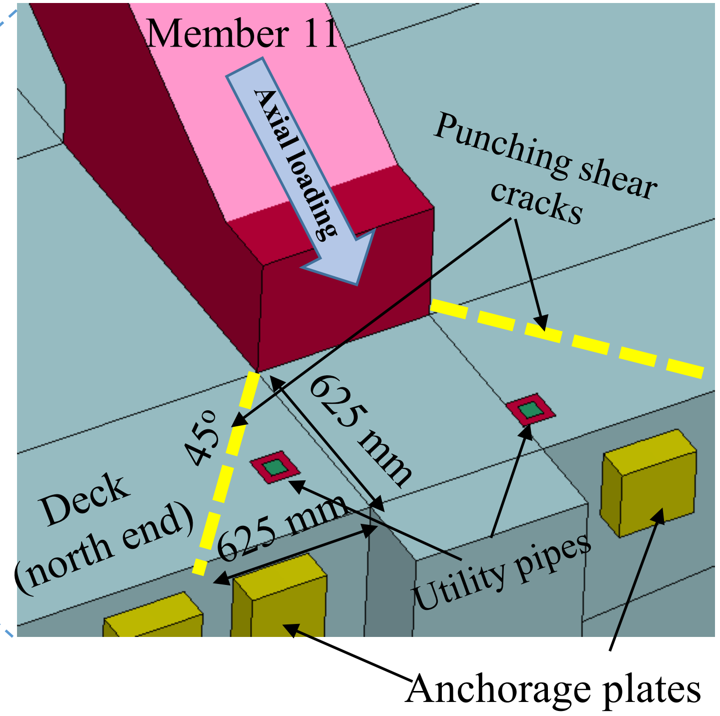
$$= 9168\ kN > C = f + Q_n = 8618\ kN$$

The shear capacity is insufficient

Critical Review of Bridge Design Calculations

Punching-Shear Capacity of the Deck

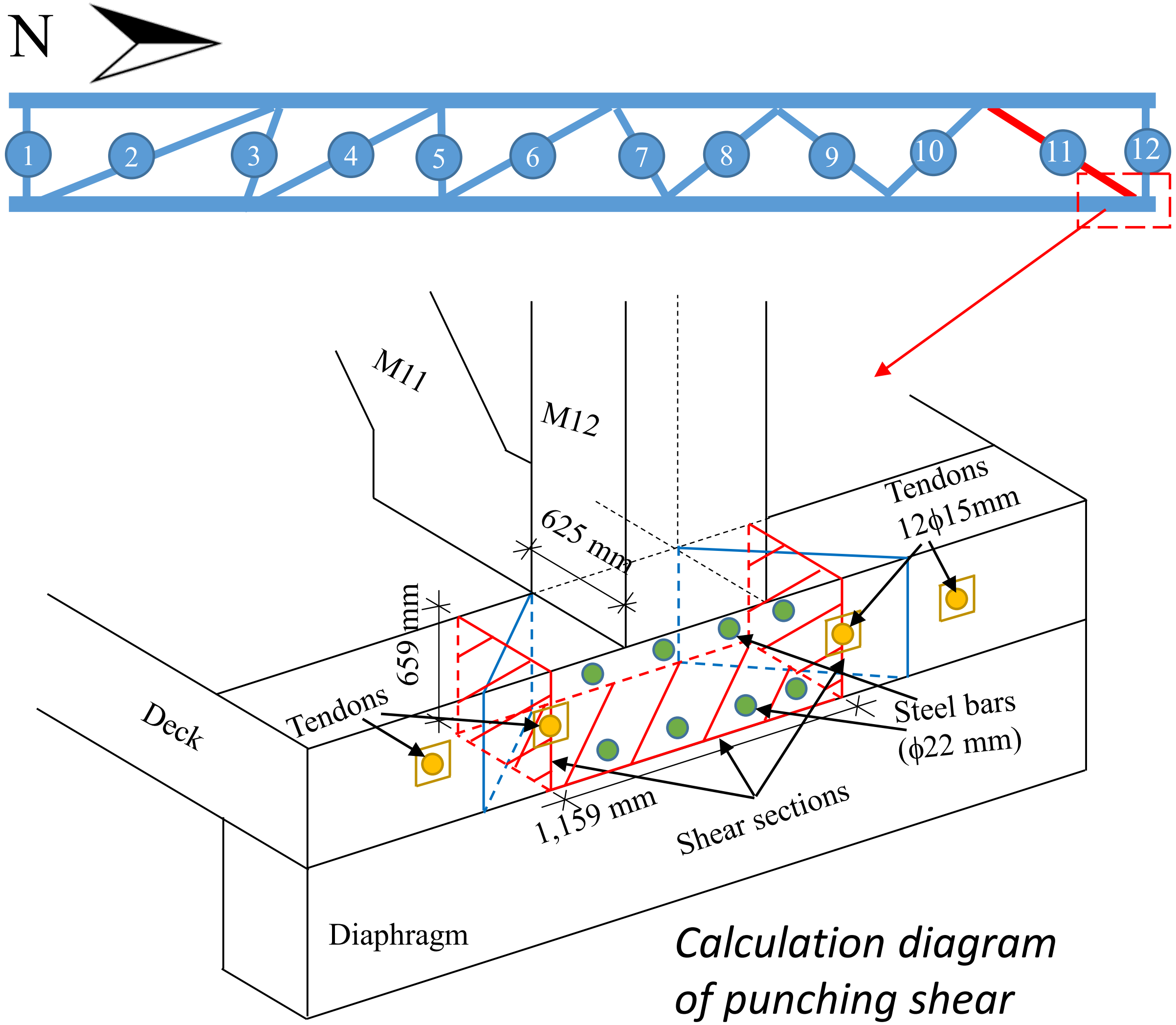




Punching shear in the cold-joint area

Critical Review of Bridge Design Calculations

Punching-Shear Capacity of the Deck



Calculation diagram of punching shear

$$V_{ps} = (1-0.18) f_{ps} A_{ps}$$ *Pre-stressing bars (18% loss)*

$$= 0.82 \times 1303\ Mpa \times 2 \times 1661\ mm^2 = 3550\ kN$$

$$V_c = 4\sqrt{f'_c} A_c$$ *Concrete*

$$= 4\sqrt{58 \times 145} \times 0.0069 \times 1548125 = 3918\ kN$$

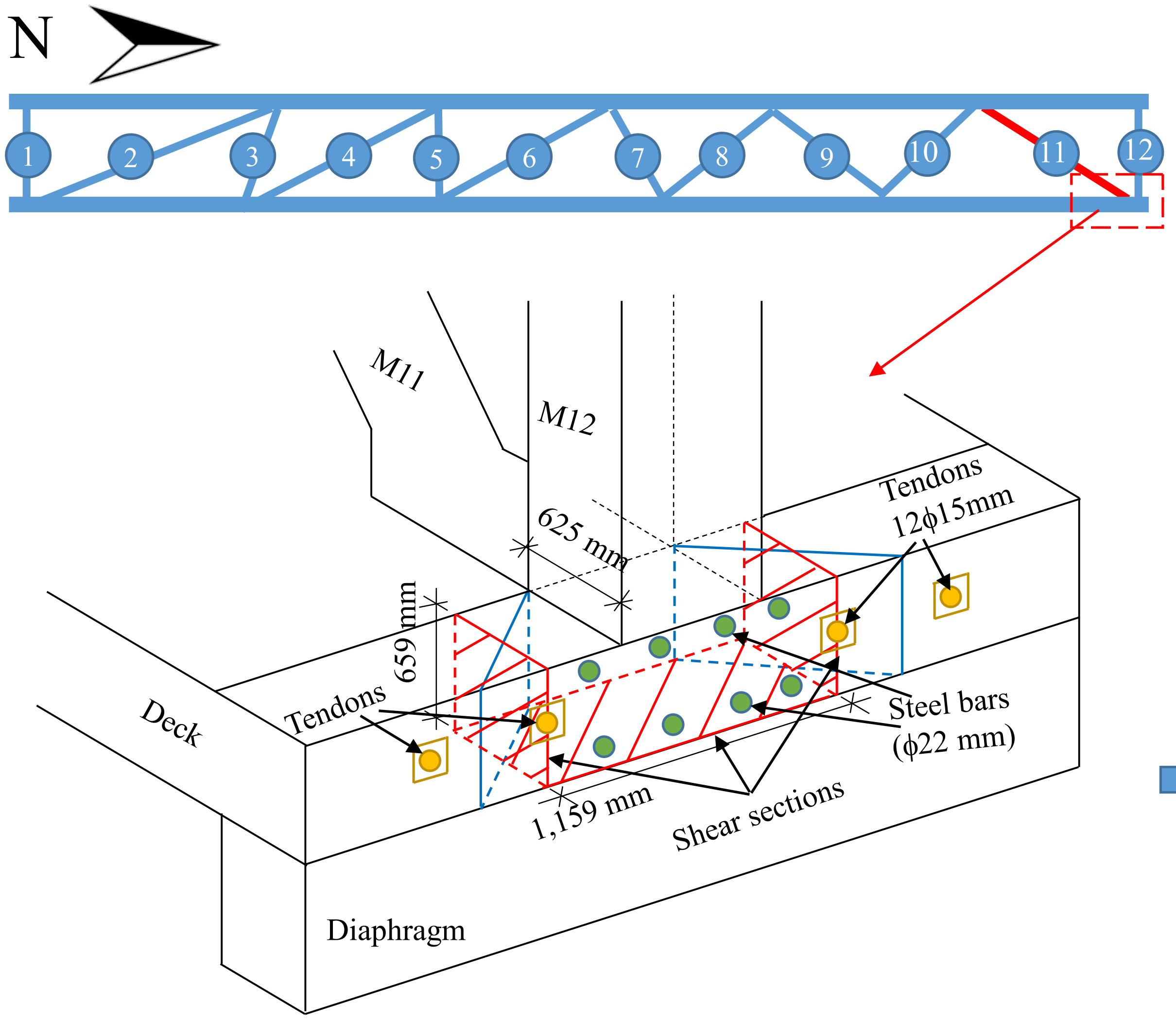
$$V_s = f_y A_s$$ *Steel bars*

$$= 414\ Mpa \times 3 \times 4 \times 2 \times 129\ mm^2 = 1282\ kN$$

$$V_{total} = V_{ps} + V_c + V_s$$ *Total capacity*

Critical Review of Bridge Design Calculations

Punching-Shear Capacity of the Deck



Case1: Punching-shear capacity, V_{total}, considering the effects of tendons:

$$V_{total} = V_c + V_{ps} + V_s = 3918 + 3550 + 1282$$
$$= 8750\ kN < D = 9168\ kN\ \text{(demand)}$$

Case2: Punching-shear capacity, V_{total}, without considering the effects of tendons:

$$V_{total} = V_c + V_s = 3918 + 1282$$
$$= 5200\ kN < D = 9168\ kN\ \text{(demand)}$$

The joint may have been deficient regarding this type of failure regardless of how much of the tendons prestressing capacity had been engaged in preventing punch out.

Critical Review of Bridge Design Calculations

Axial Loading Capacity of Member 11

$$P_n = 0.85 f'_c (A_g - A_{st} - A_{ps}) + A_{st} f_y$$

$$= 0.85 \times 58\ Mpa \times 318961\ mm^2 + 3096\ mm^2 \times 414\ Mpa$$

$$= 15725 + 1282 = 17007\ kN$$ (capacity)

Demand:

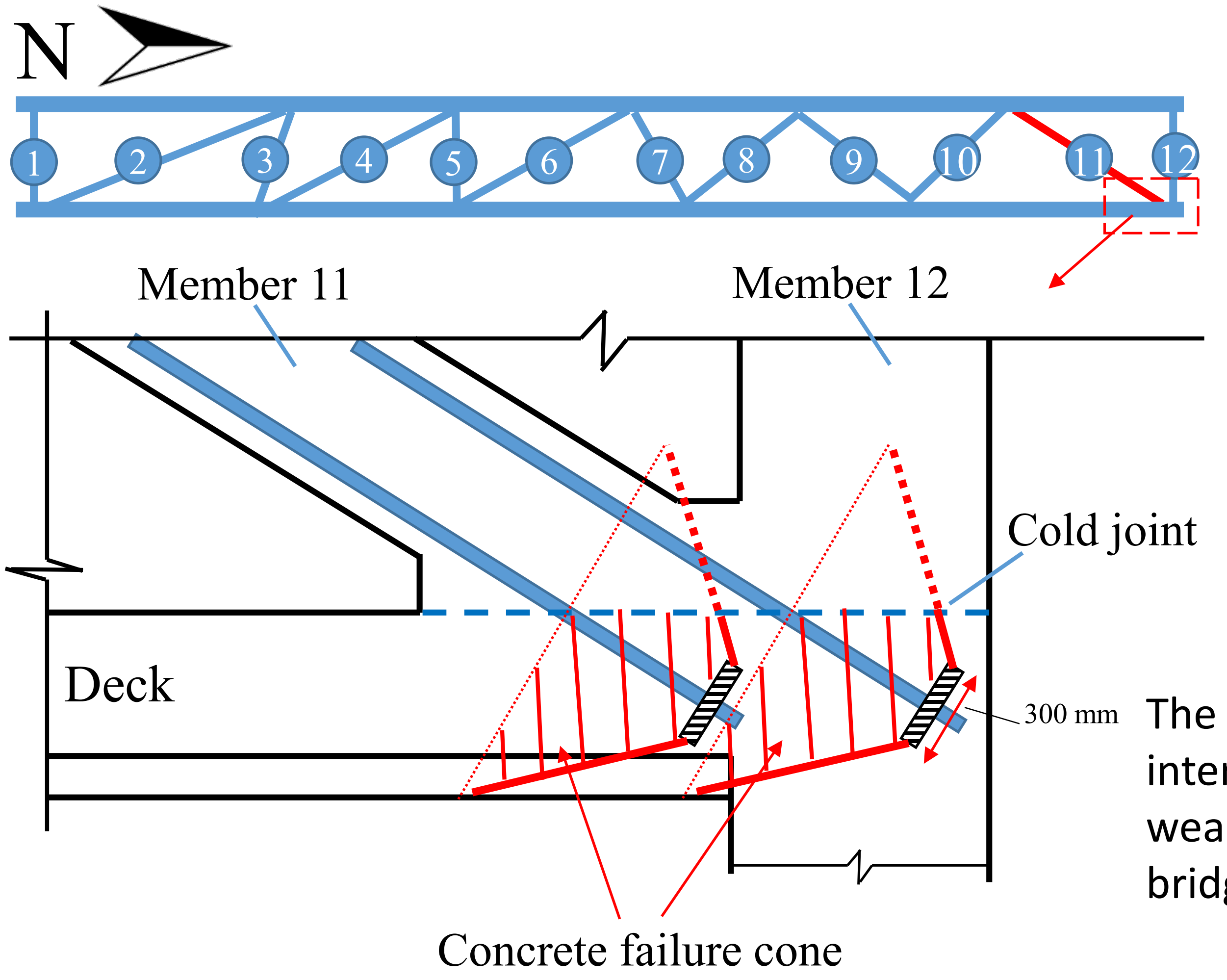
$$D_{m11} = F_G + F_P = 8205\ kN + 2491\ kN = 10696\ kN < P_n = 17007\ kN$$

$$\phi P_n = 0.8 \times P_n = 13606\ kN > D_{m11} = 10696\ kN$$ *(considering accidental eccentricities)*

- Based on the computed results, the axial capacity appears to be sufficient to resist the axial demand.
- Also, the design capacity was found to be sufficient even when considering accidental eccentricities (strength reduced by 20%) per ACI (2011), which also suggests that the design of member 11 to support axial load was not flawed.

Critical Review of Bridge Design Calculations

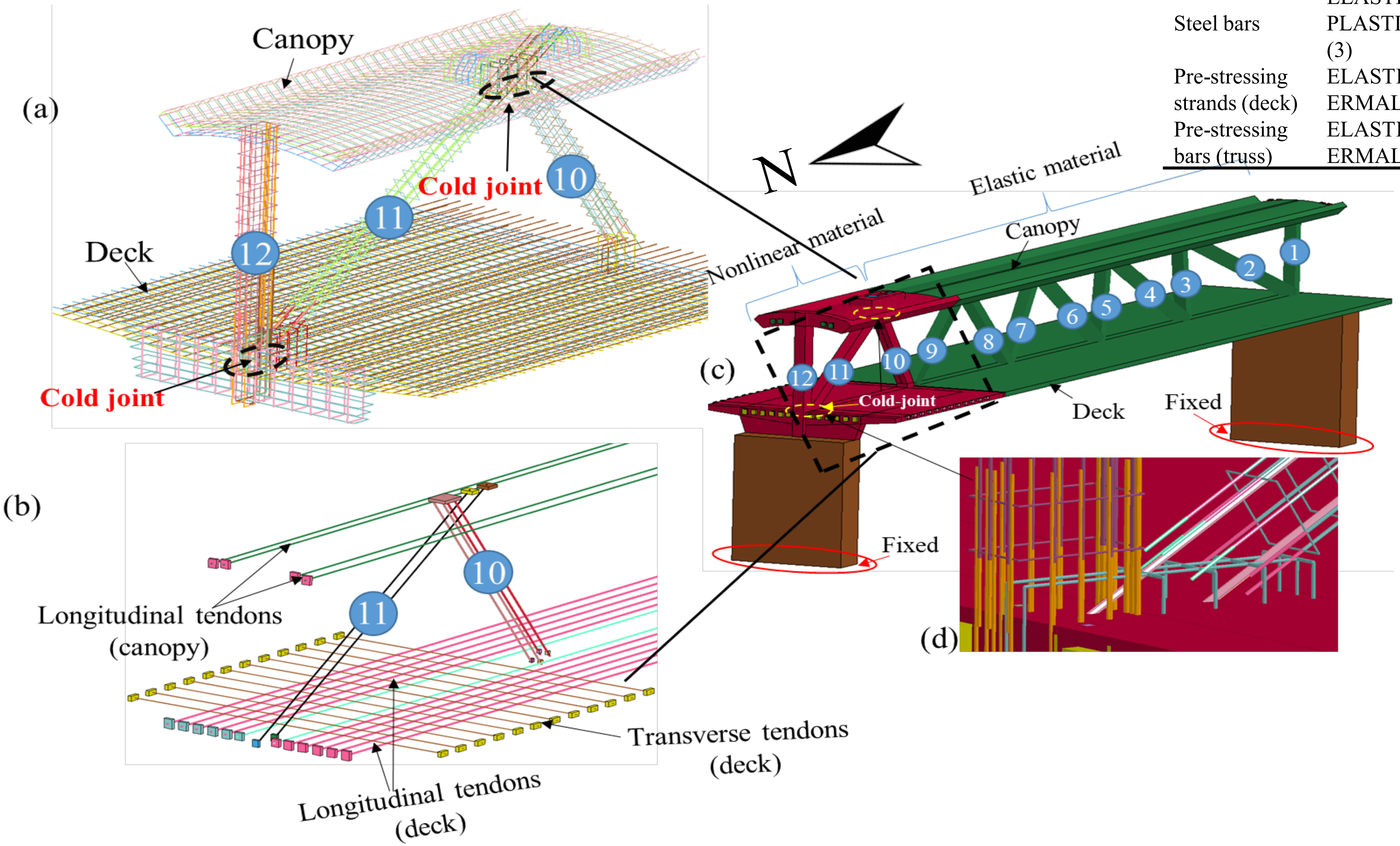
Blowout Damage from Pre-stressing Bars



The anchor plates could have caused internal damage during transportation, weakening the joint region before the bridge went into service.

Finite Element Modeling

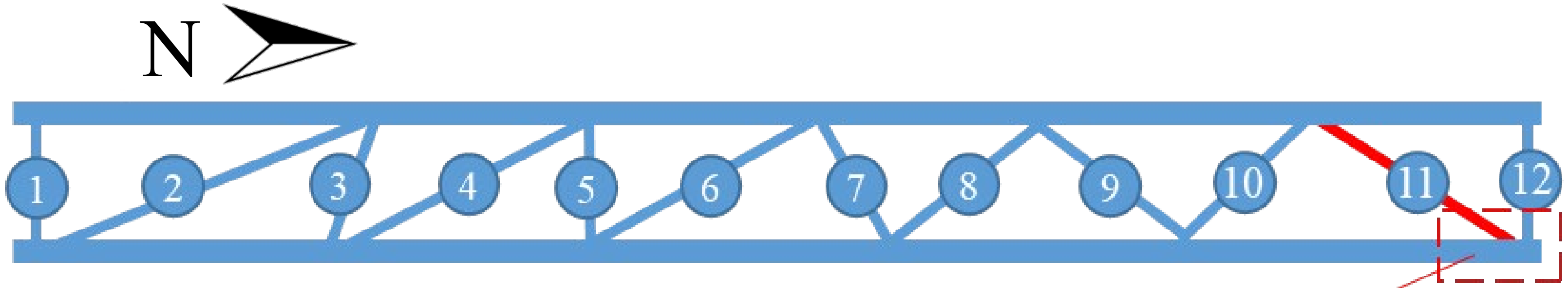
Parts	Material model	Compressive/yielding strength (Mpa)	Modulus of elasticity (Mpa)
Concrete	CSCM (159) ELASTIC (1)	58	36,232
Steel bars	PLASTIC_KINEMATIC (3)	414	200,000
Pre-stressing strands (deck)	ELASTIC_PLASTIC_THERMAL (4)	1,675	196,500
Pre-stressing bars (truss)	ELASTIC_PLASTIC_THERMAL (4)	896	200,000

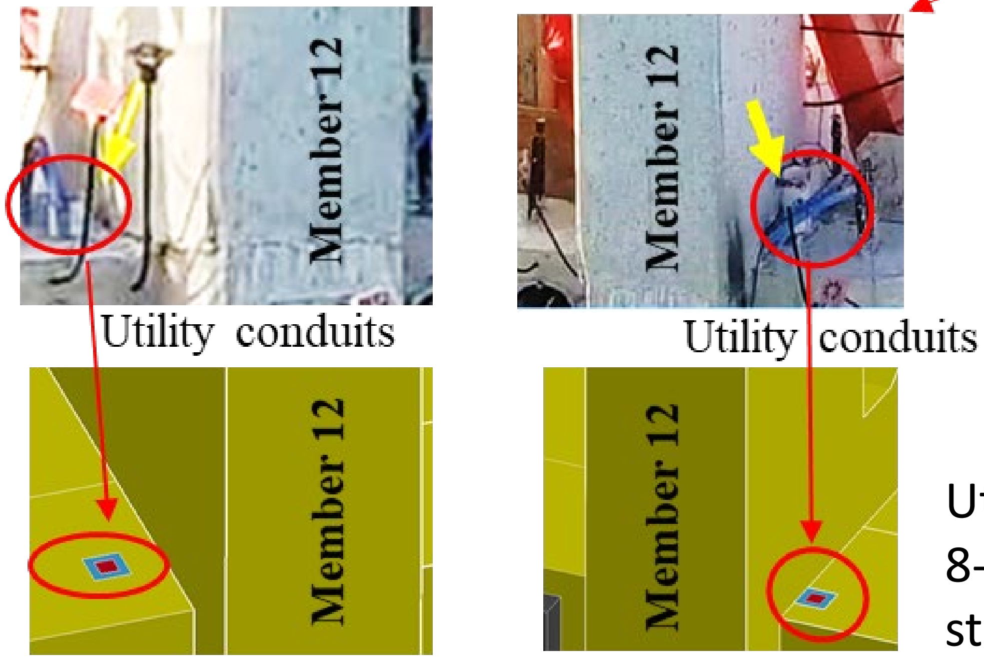


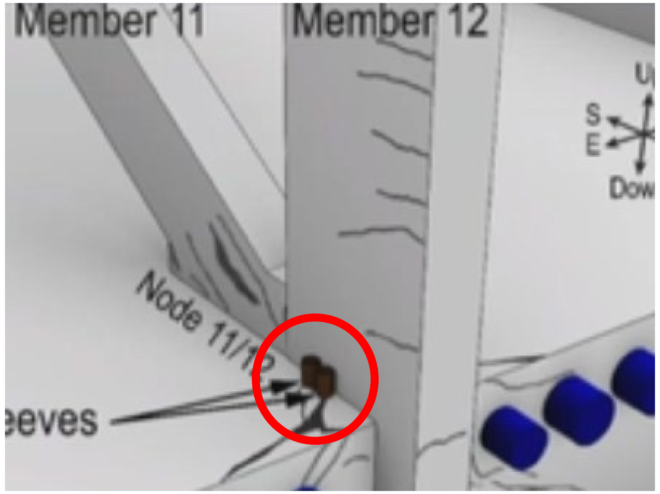
- A total of 390,000 elements.
- Mesh size: 1.5 inch.
- Time step: 2×10^{-6} s.

Finite Element Modeling

- Utility conduits modeling at the north end of the bridge



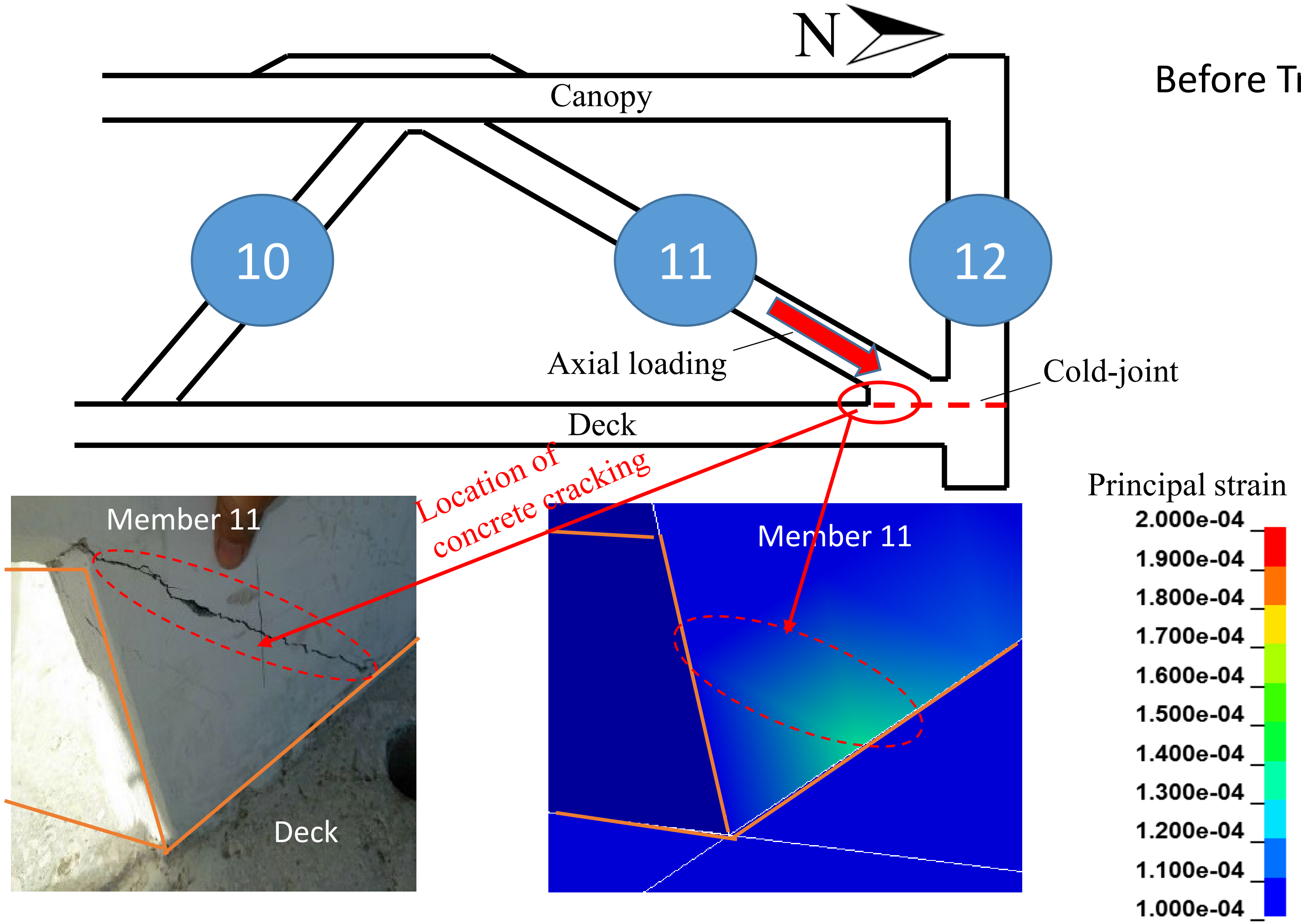




Utility conduits were modeled using 8-node solid elements with low strength concrete (20 Mpa).

Numerical Simulation Results

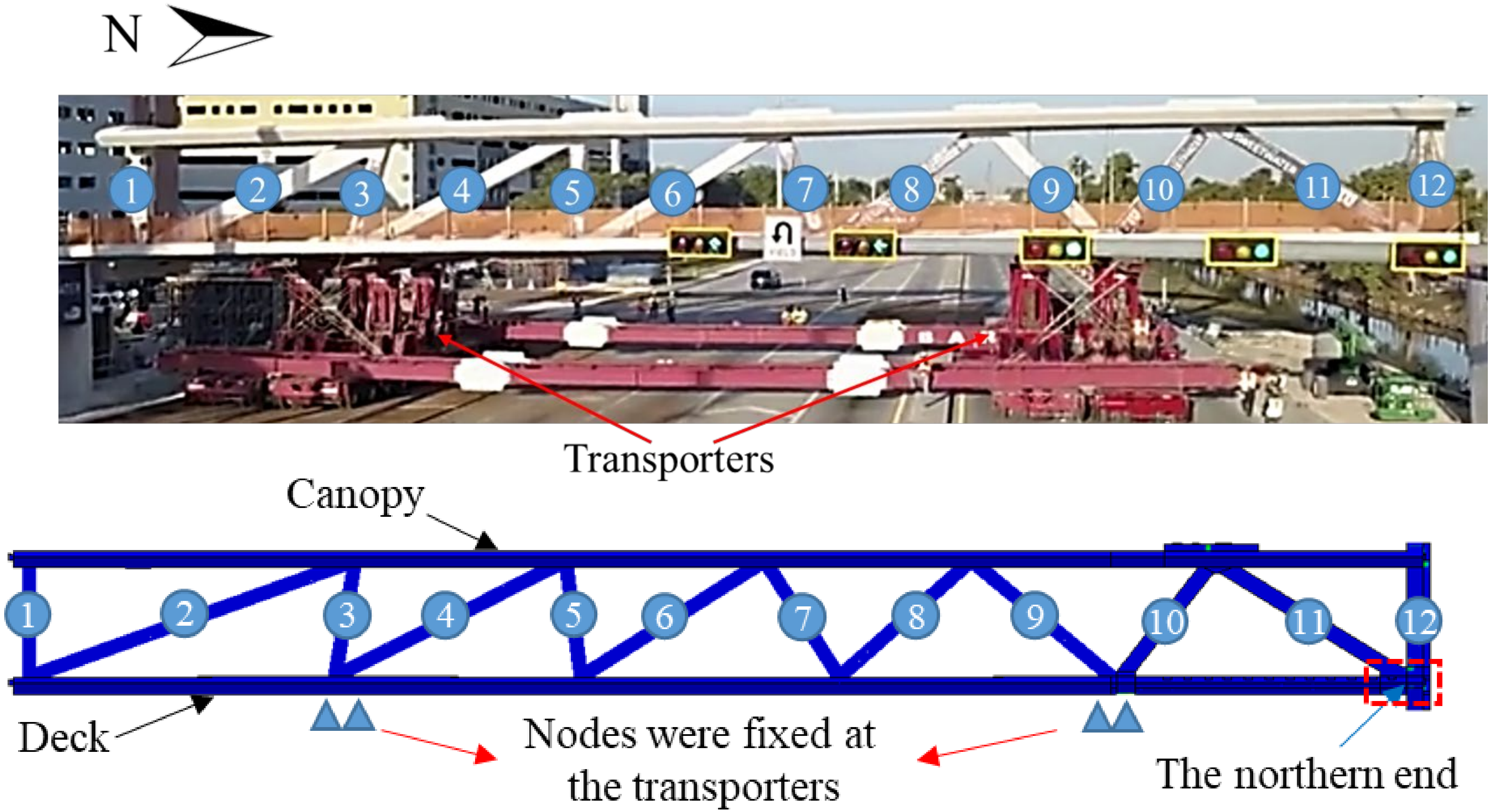
- Comparison of cracks in the joint area of member 11 after pre-stressing



Before Transportation

Numerical Simulation Results

- Transportation stage: no severe damage in the northern joint area

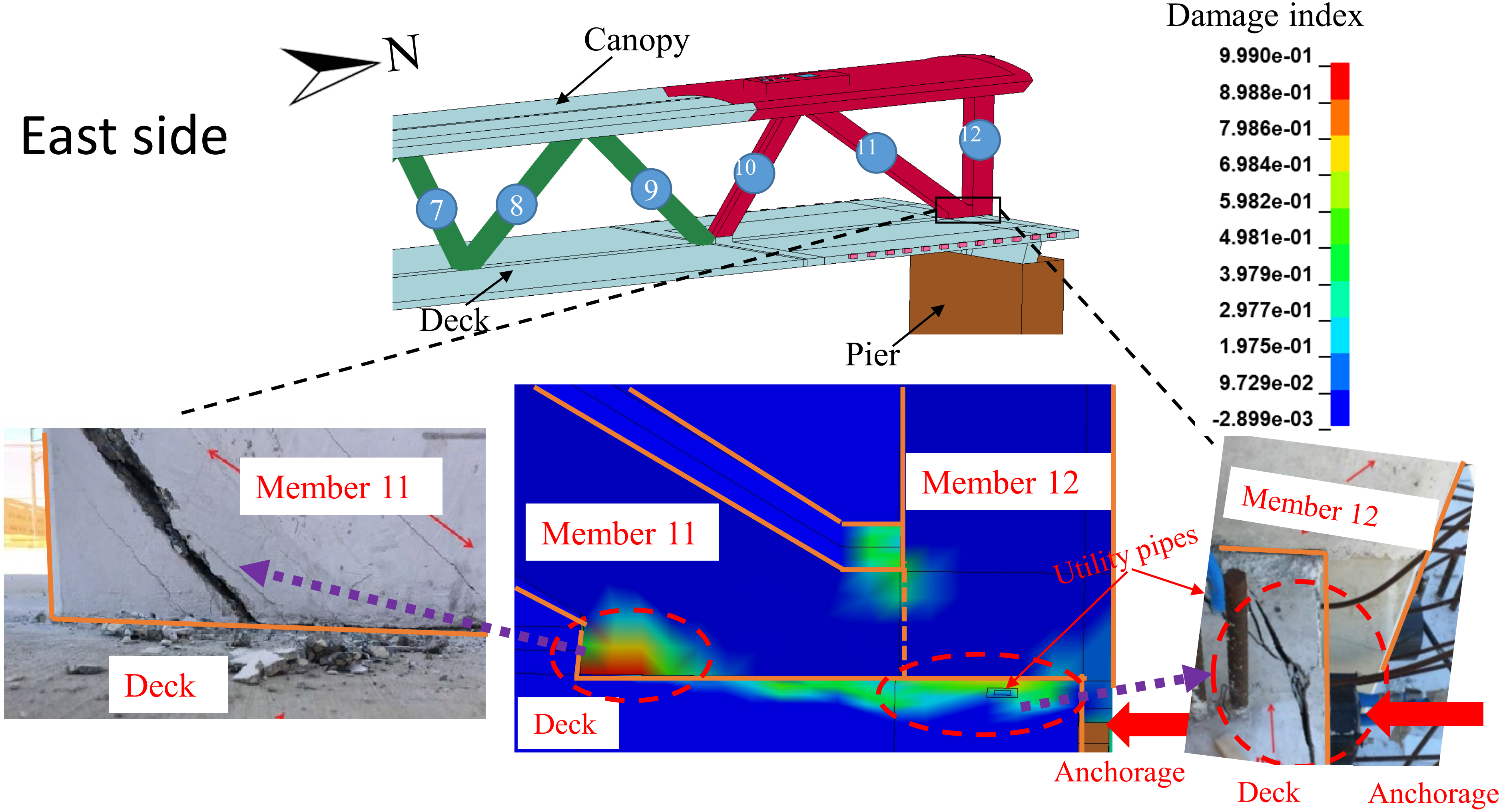


No severe damage

Numerical Simulation Results

- Comparison of cracks in the joint area of member 11 after relocation

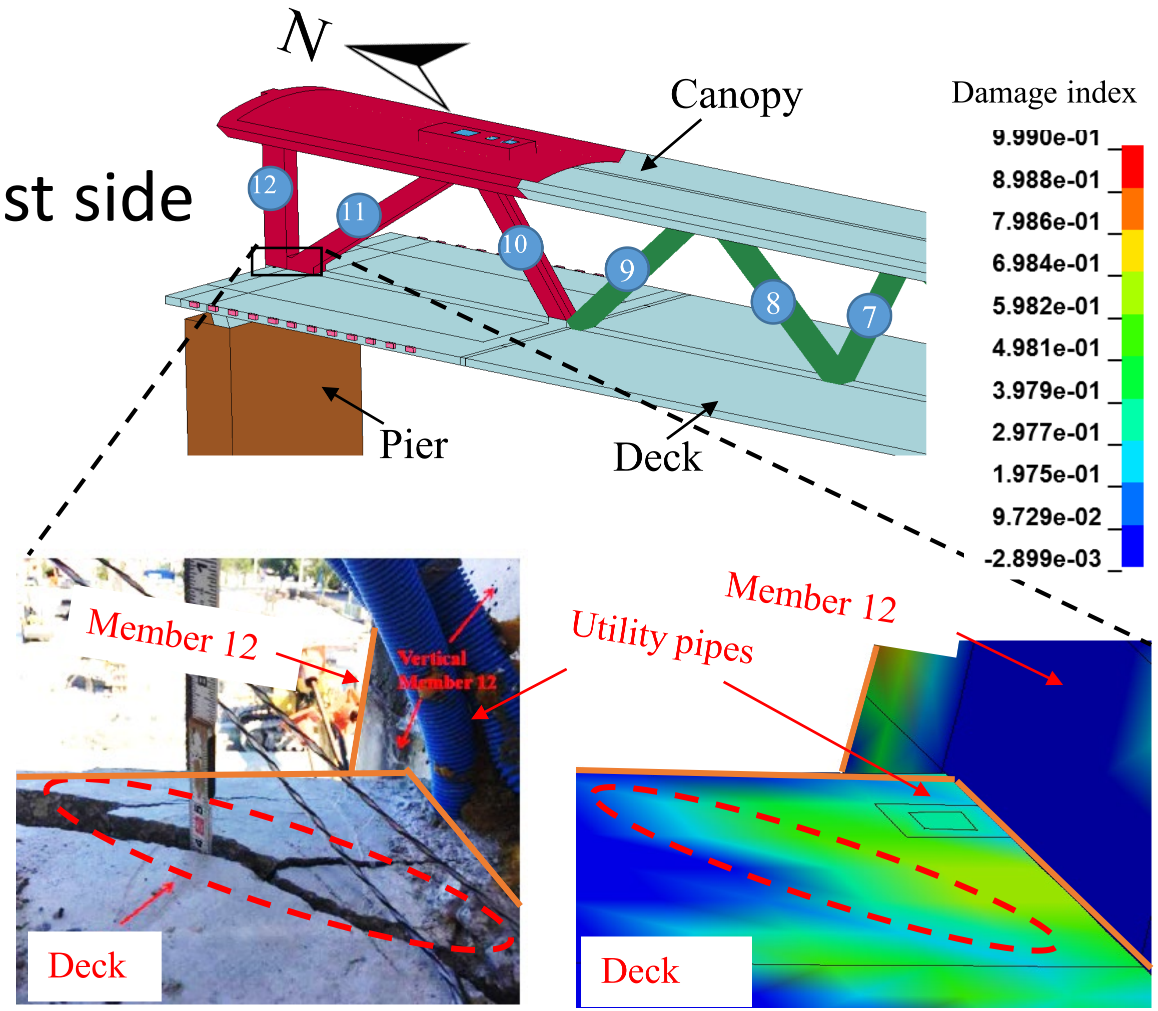
- East side



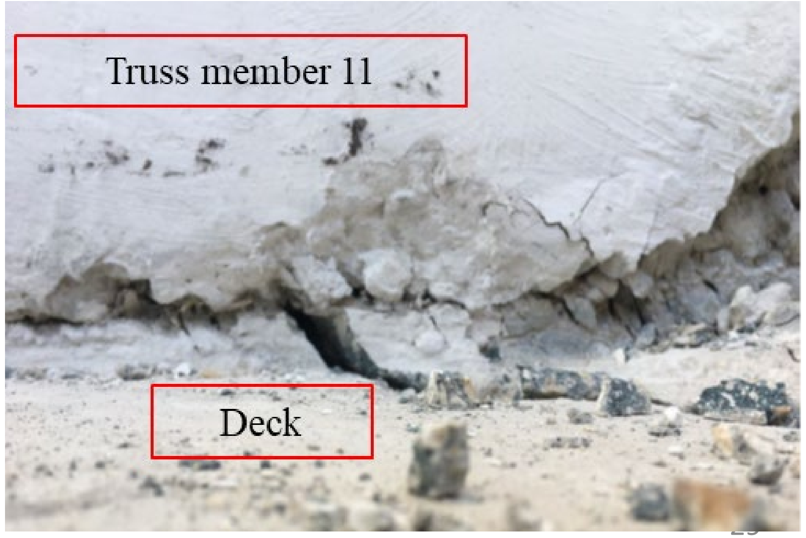
Numerical Simulation Results

- Comparison of cracks in the joint area of member 11 after relocation

- West side



Slip at the cold-joint

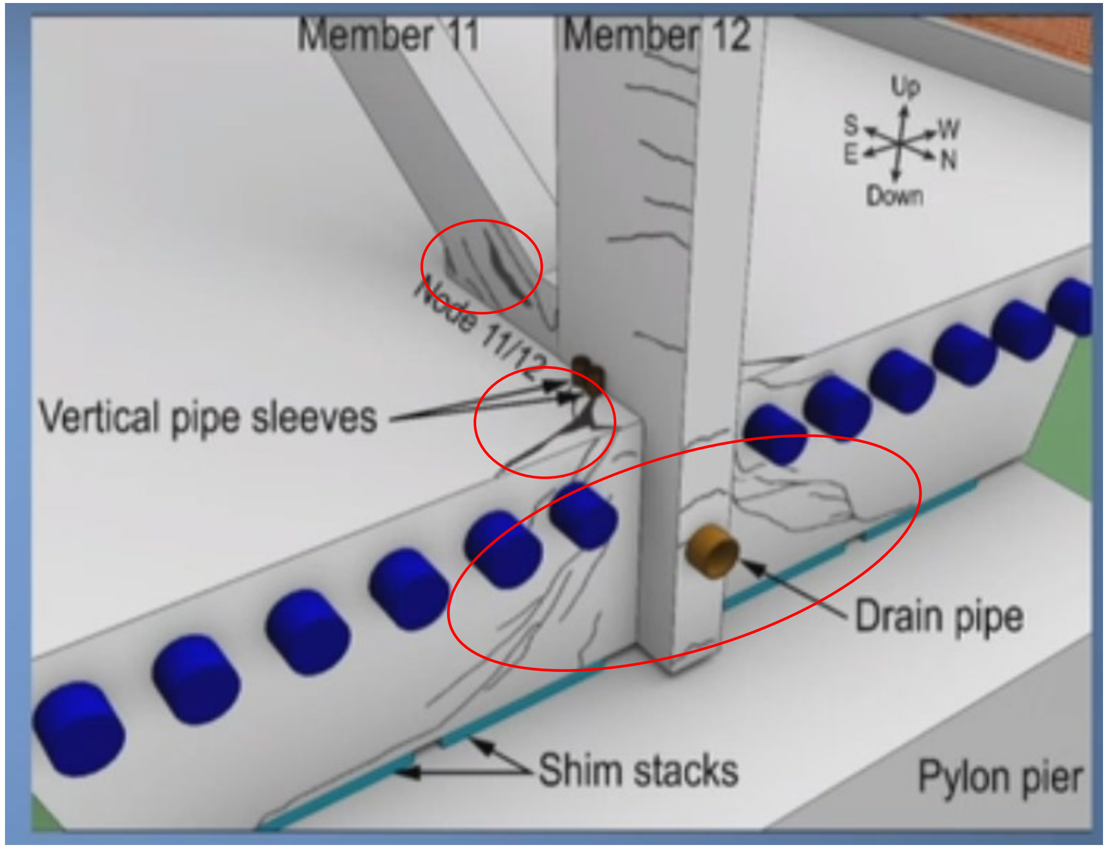


Ayub (2019)

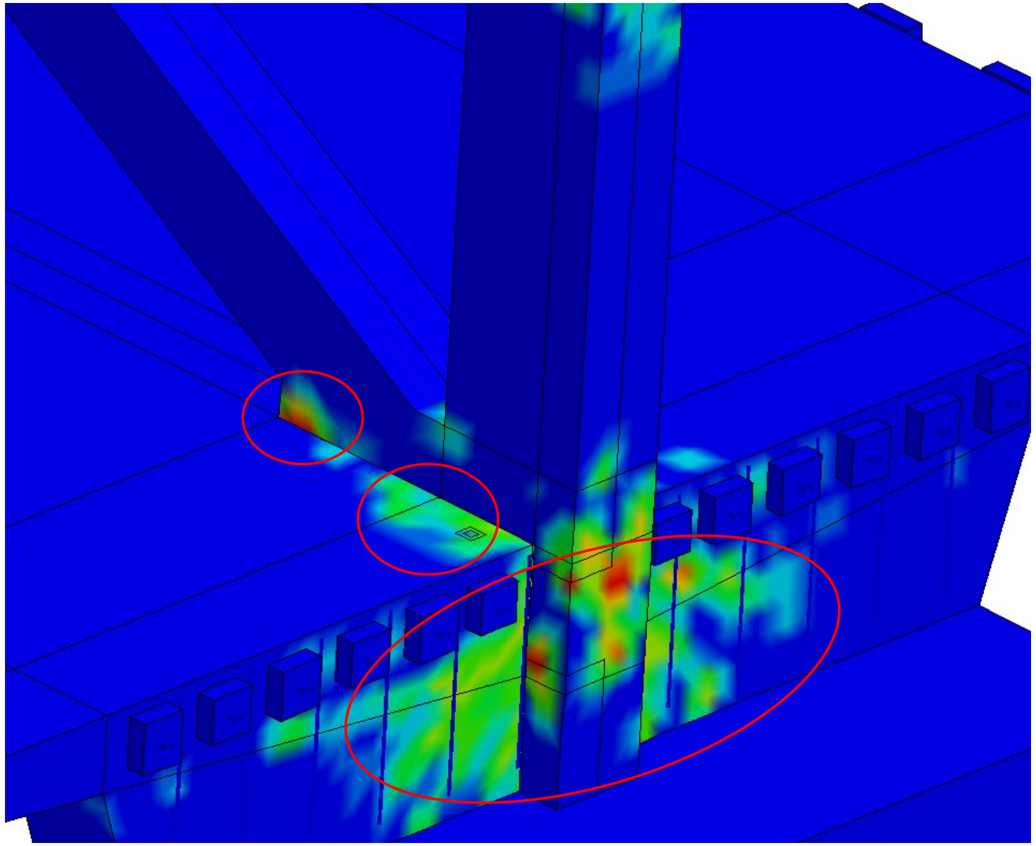
Numerical Simulation Results

- After relocation
- Before re-tensioning

Cracks were still developing before retensioning, but the bridge didn't collapse. Instead of repairing the bridge, the contractor / designer decided to retension member 11, which further damaged the already cracked joint.



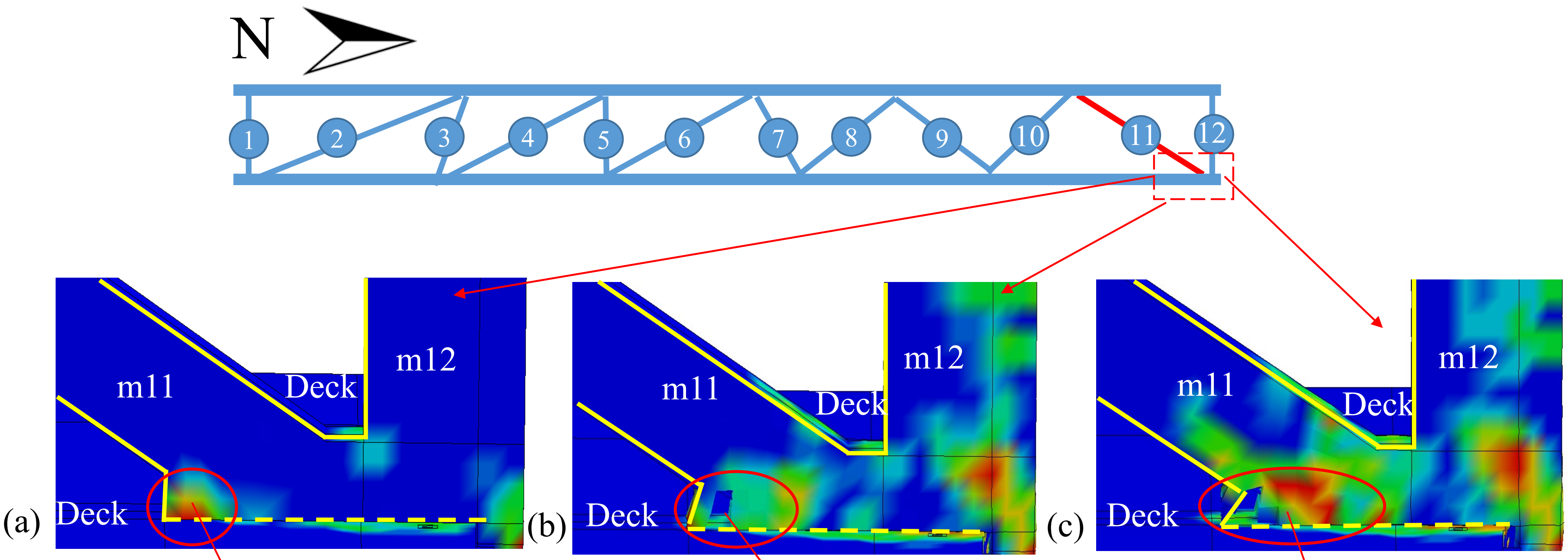
Accident (Source: NTSB)



Simulation

Numerical Simulation Results

- Re-tension tendons in m11 to maximum jacking stress of the tendons, i.e. 94% of yield strength per ACI (2011)



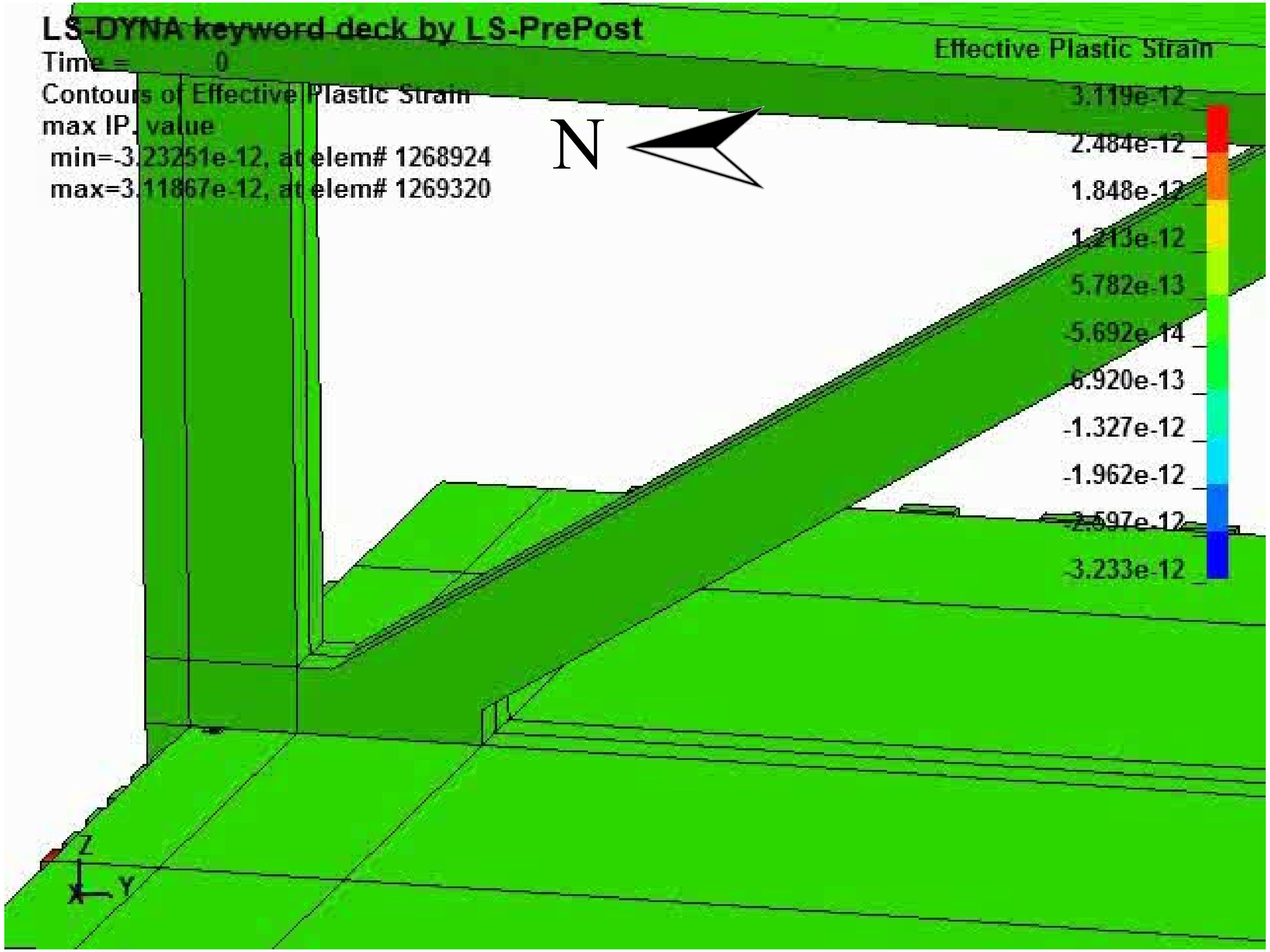
Damaged heel of member 11 (before re-tensioning)

After re-tensioning

Damage due to sliding

Concrete severely damaged by excessive sliding and mobilization of dowel action

Numerical Simulation Results

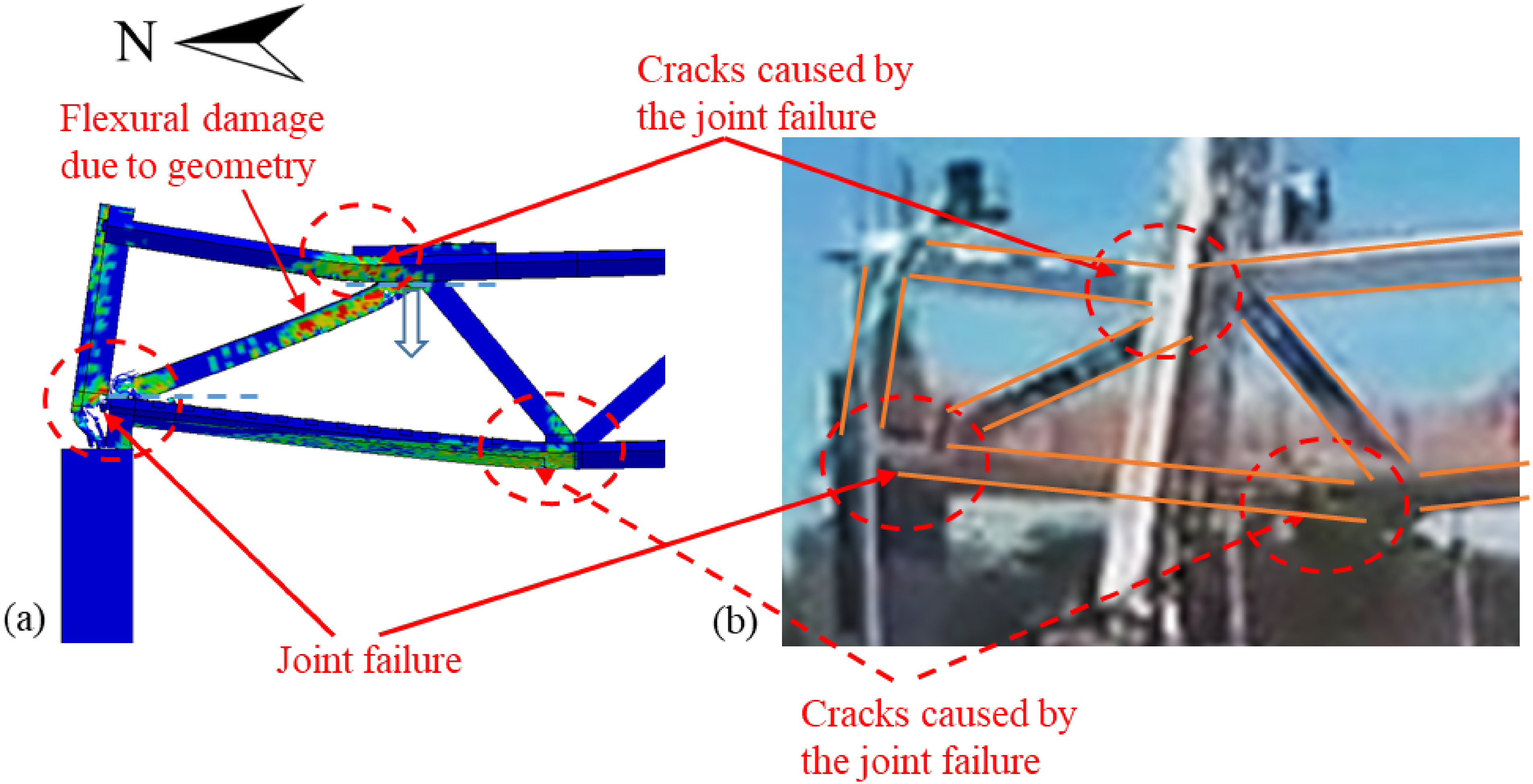


- **Failure sequence:**

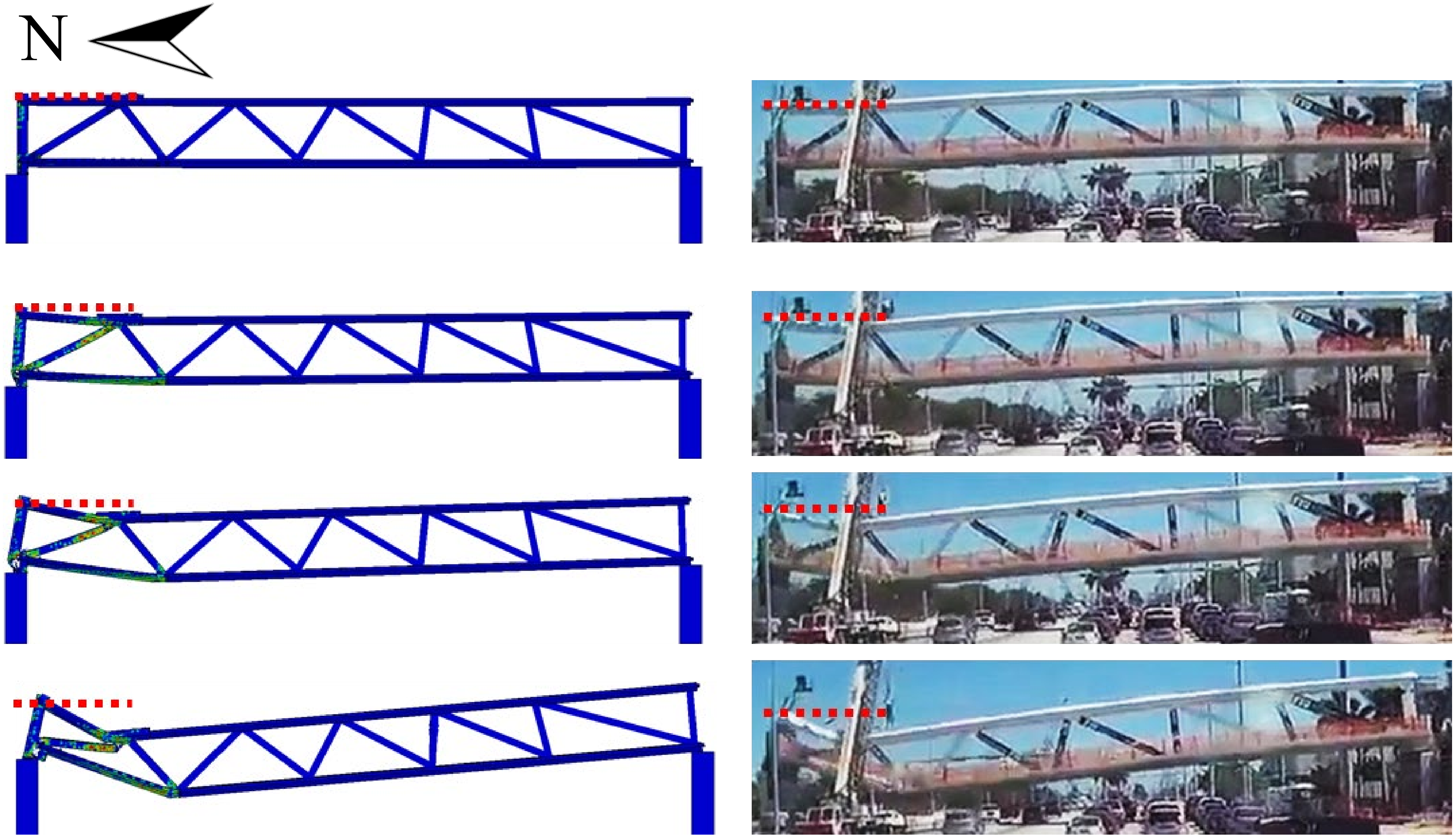
1. Damaged heel of member 11 under gravity and pre-stressing (before re-tensioning).
2. After re-tensioning, concrete in the cold joint area was severely damaged by excessive sliding and mobilization of dowel action, which also caused punching shear failure.
3. Member 11 and member 12 were pushed off the deck. The deck fell off the support, leaving behind a portion of member 11/12's joint wedged onto the pier.

Numerical Simulation Results

- Comparison of the failure mode



Numerical Simulation Results



0.00 sec

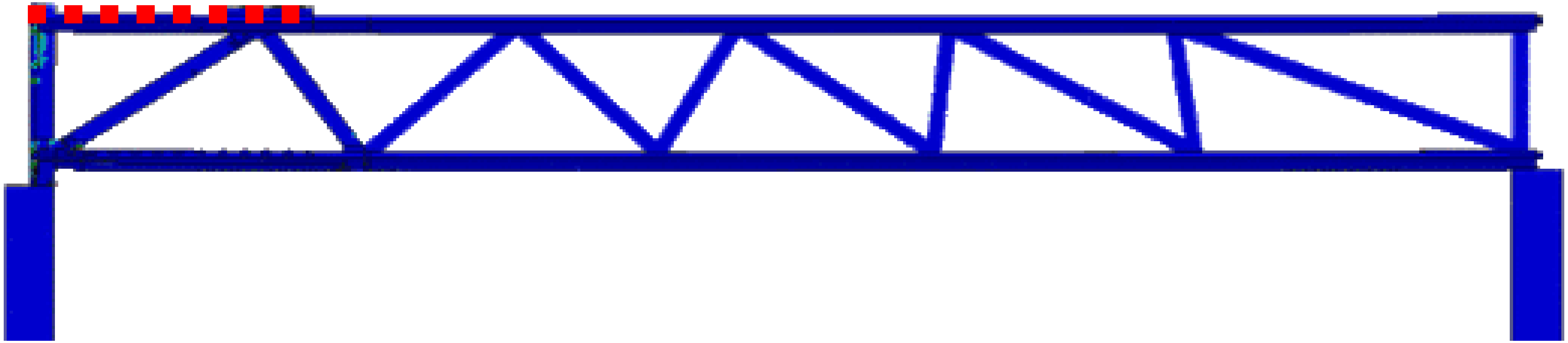
0.50 sec

1.50 sec

2.00 sec

- **Collapse sequence**

Numerical Simulation Results



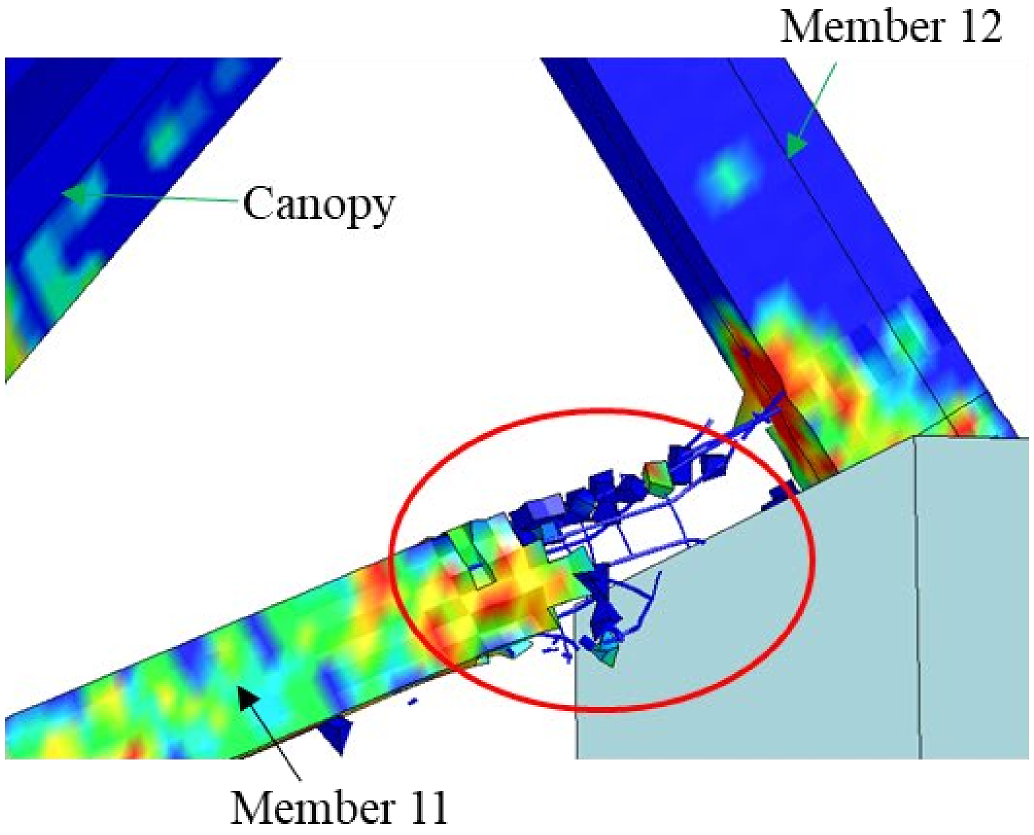
Numerical Simulation Results

Numerical Simulation Results

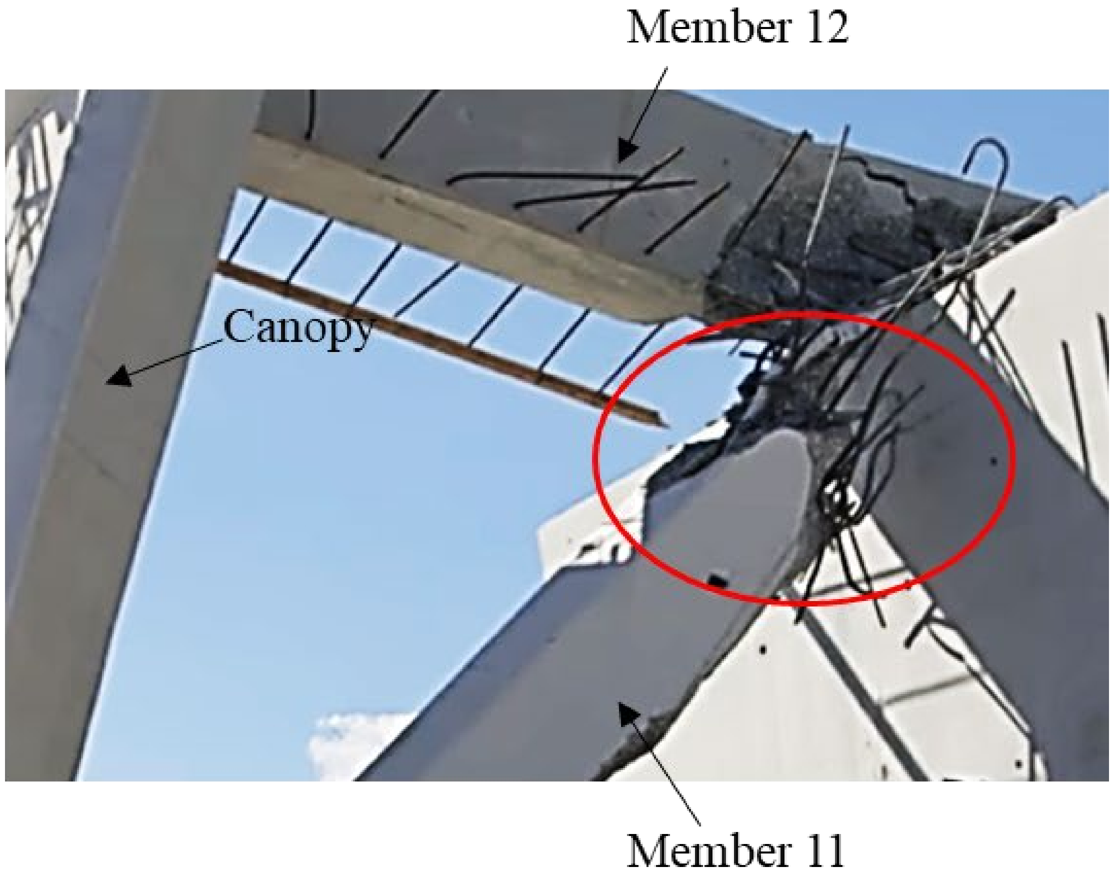
Numerical Simulation Results

- Close-up of failure of member 11

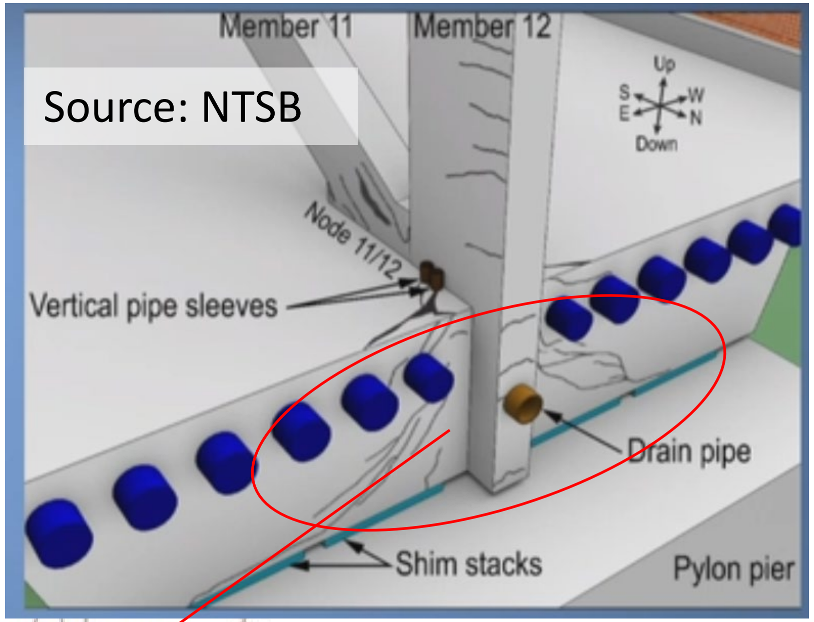
(a)



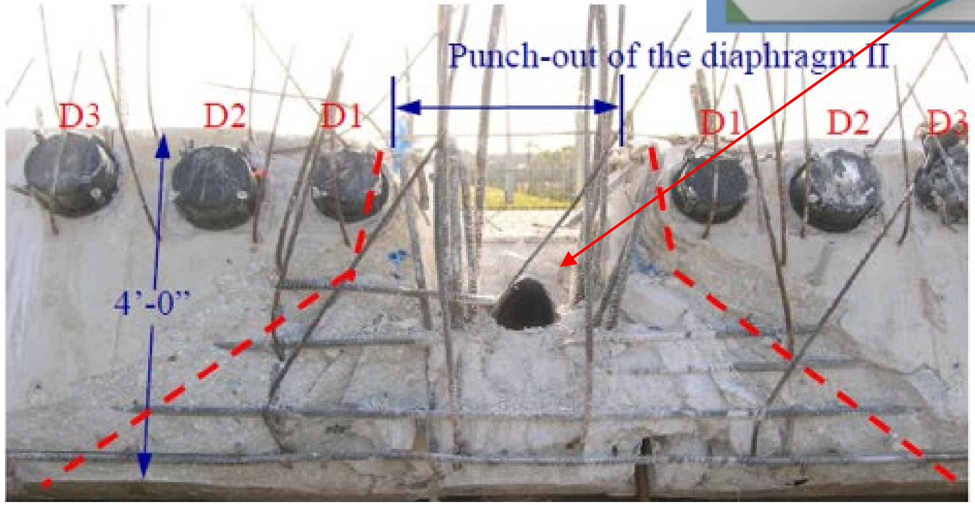
(b)



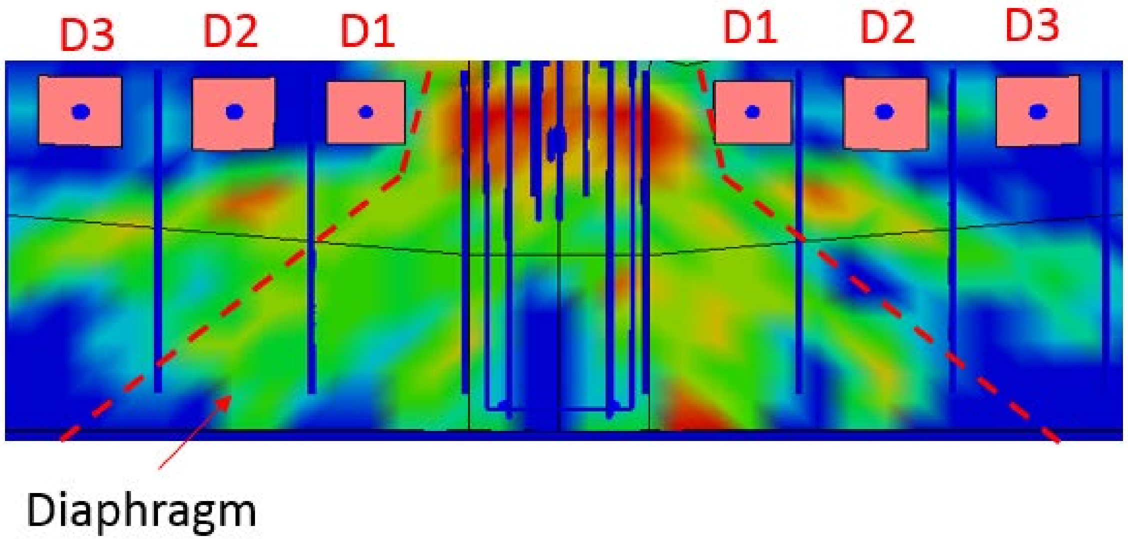
Numerical Simulation Results



- After collapse



Source: OSHA report (2019)



Parametric Study

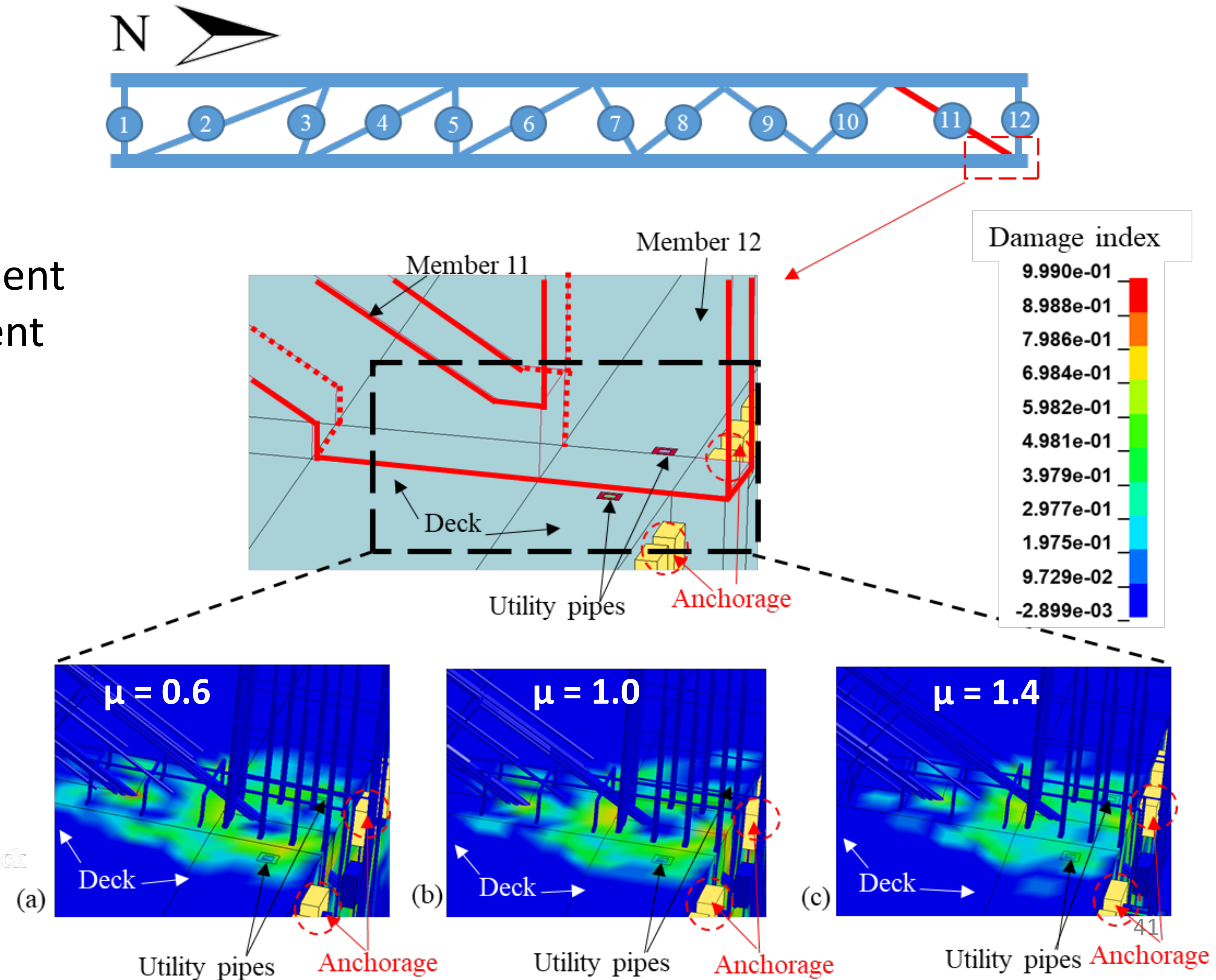
Table. Simulation matrix for parametric studies

Cases	Static coefficient of friction at the cold-joint	Pre-stress losses in the deck	Re-tensioning stress/yield strength
1	0.6	18%	N/A
2	1.0	18%	N/A
3	1.4	18%	N/A
4	1.0	28%	N/A
5	1.0	38%	N/A
6	1.0	18%	55%
7	1.0	18%	80%
8	1.0	18%	90%
9	1.0	18%	95%
10	1.0	18%	100%

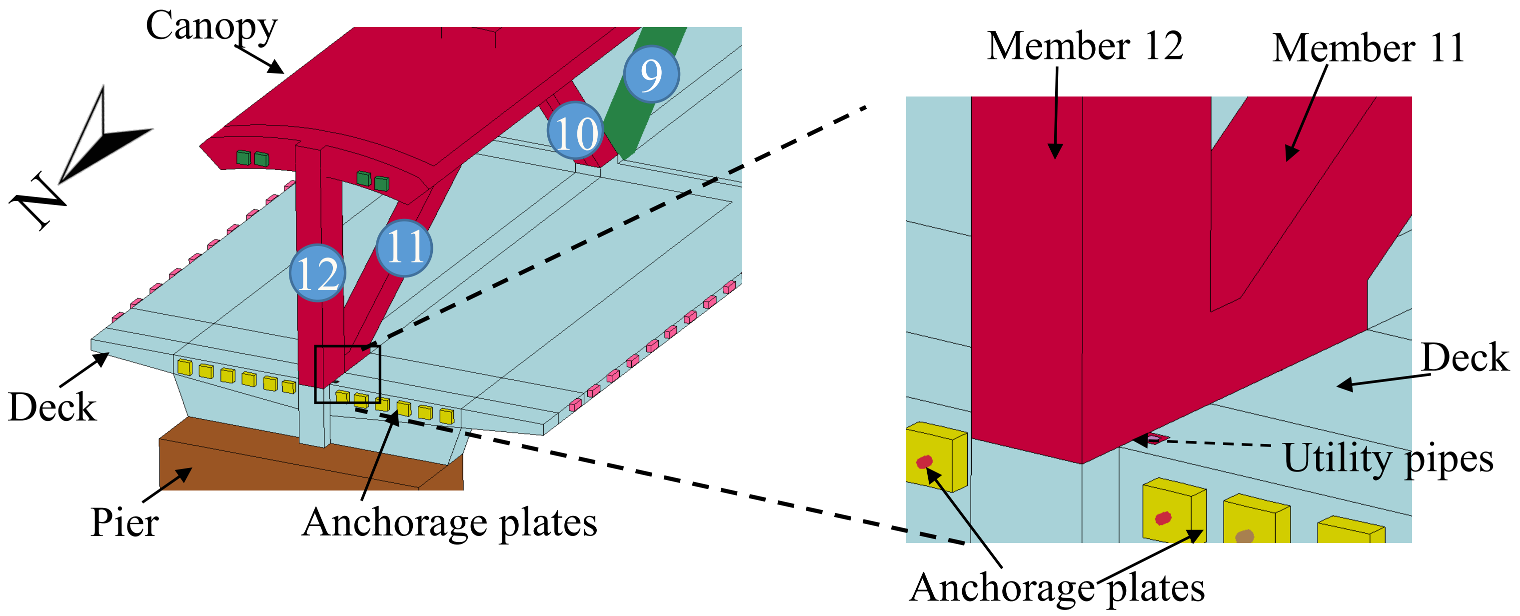
N/A-no re-tensioning is applied.

Parametric Study: Static Coefficients of Friction

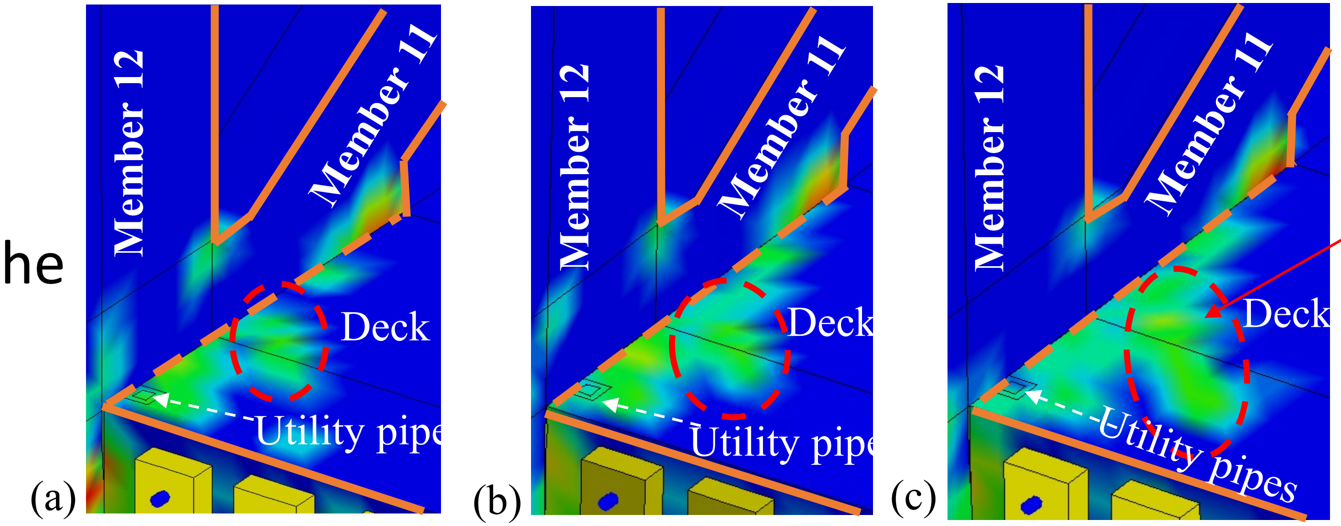
- The original design relied on friction to resist the horizontal shear forces, but simulation showed that even a high coefficient of friction of 1.4 could not prevent severe damage in the cold-joint area.
- According to OSHA and NTSB reports, the surface of the cold joints was not intentionally roughened.
  - Damage mode of the cold-joint area



Parametric Study: Prestress Losses in the Deck



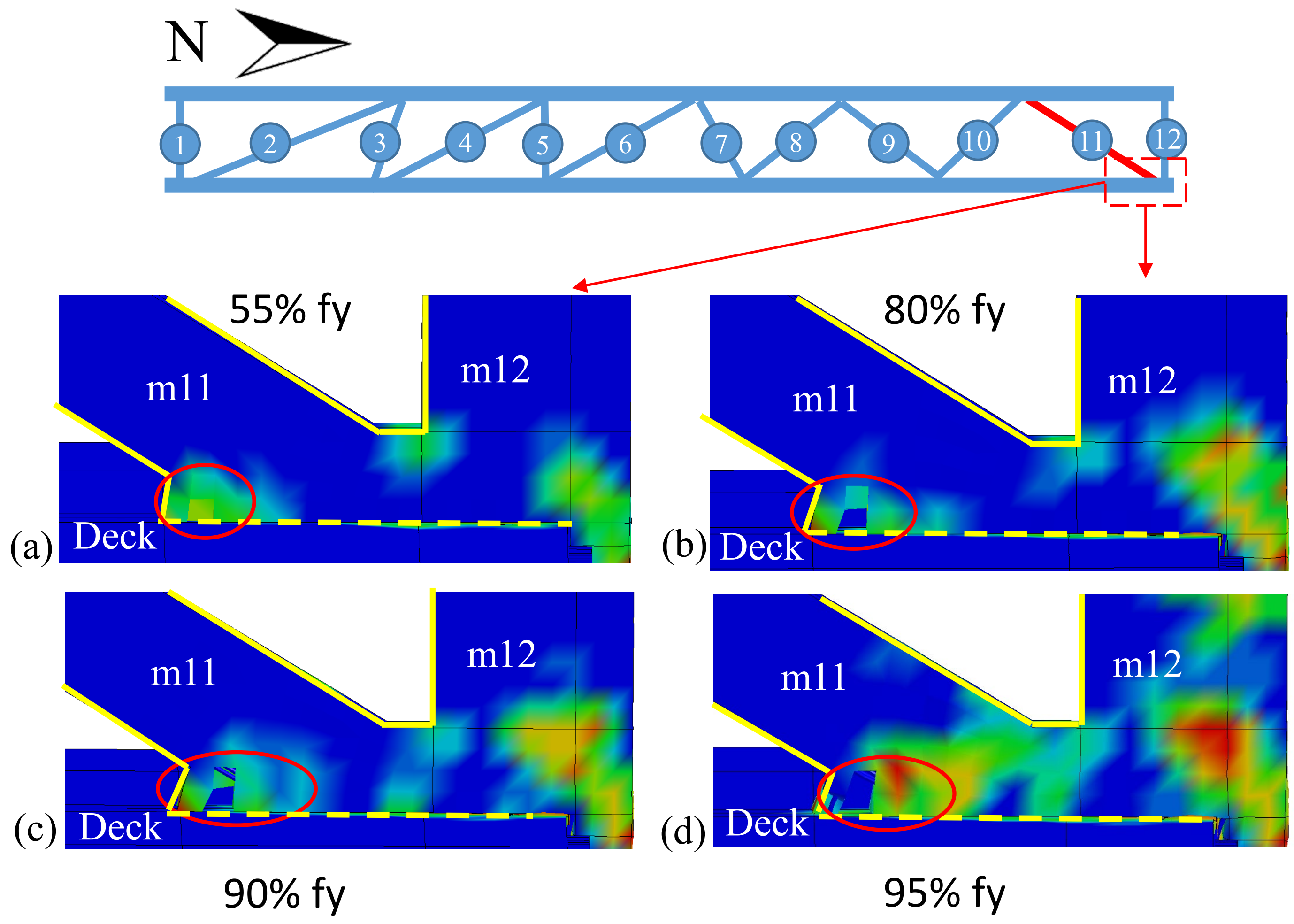
- Damage mode of the cold-joint area



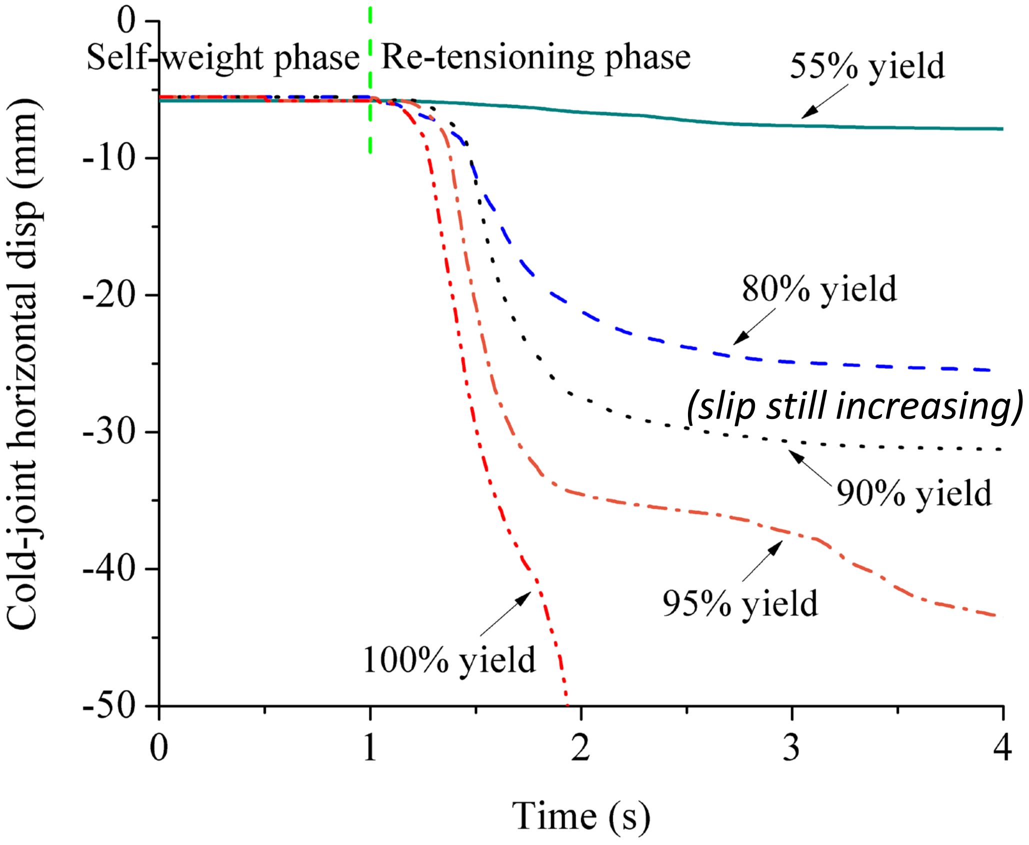
18% loss
28% loss
38% loss

the zone of punching-shear damage expanded as the pre-stressing losses increased

Parametric Study: Re-tensioning on Diagonal Member 11

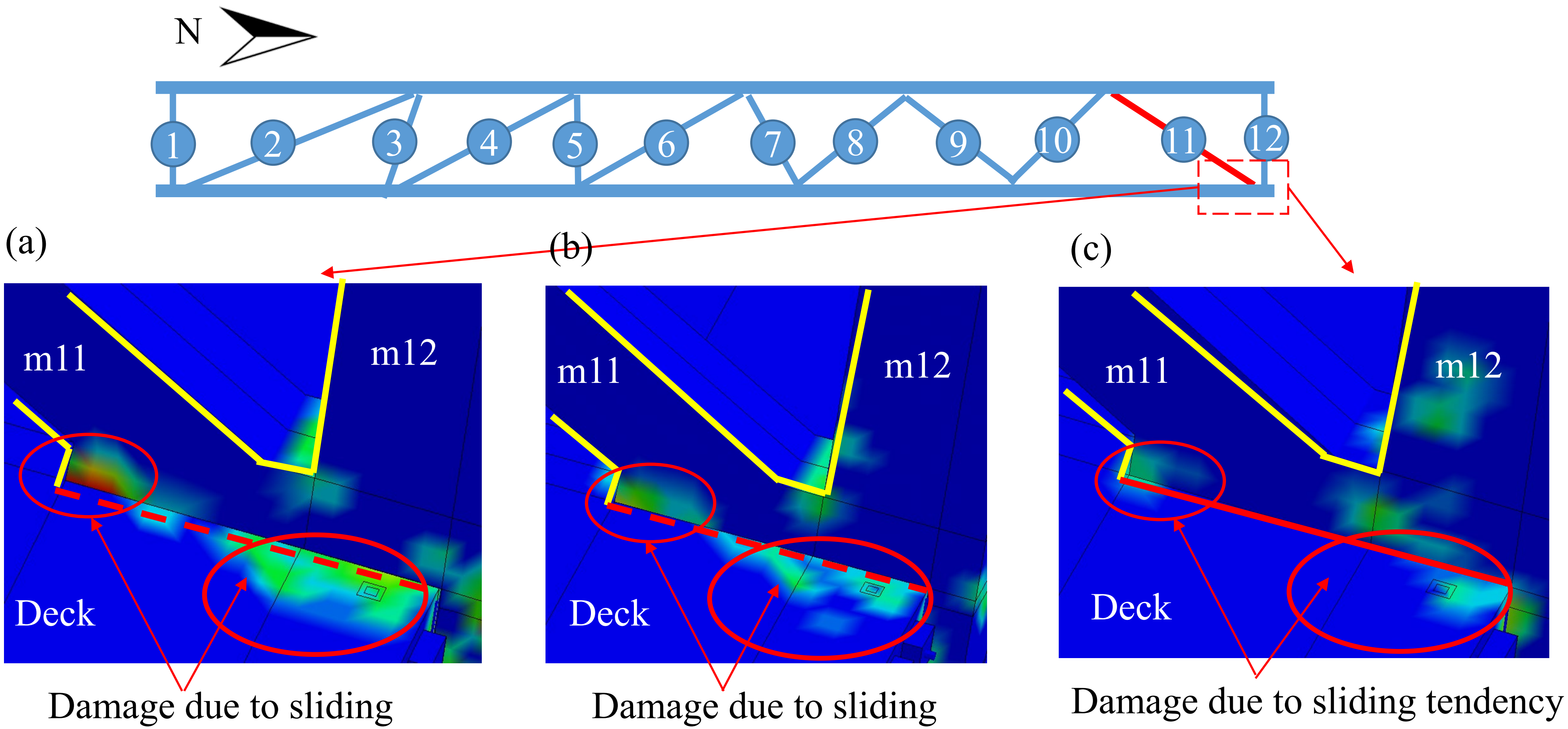


Based on OSHA report, the tendons in member 11 were re-tensioned over 80% of the yield strength, which could cause significant increase in the cold-joint slip and may further cause collapse of the bridge.



Parametric Study

- Influence of Shear Capacity of the Construction Joint



original shear connector design (cold-joint)

revised shear connector design (cold-joint)

monolithic joint design (with original reinforcement)

Conclusions and Lessons Learned

1. Simulation results show that the cold joint between members 11 and 12 and the deck could have played a significant role in the collapse of the bridge.
2. Diagonal member 11 did not suffer axial load failure, such as crushing or buckling.
3. Relying on friction at the joint between members 11, 12 and the deck is risky in a non-redundant system like that used in the bridge. Shear keys or some other explicit shear resisting mechanism placed in the cold joints would have been more reliable and helpful in meeting the horizontal shear demand in the joint.

Conclusions and Lessons Learned

4. Re-tensioning diagonal truss members should not be considered as an appropriate solution to remedy the cracks in the cold joint area.
5. Cracks in the cold joint area should be viewed as an important warning sign of impending collapse and immediate action should be taken to ensure the stability of the structure after detailed calculations or modeling.
6. The collapse of the bridge doesn’t necessarily imply that accelerated bridge construction is risky, but certainly shows the need for adequate analysis simulating construction aspects such as the presence of cold joints or utility conduits, to ensure the safety of the bridge during and after the construction.

Conclusions and Lessons Learned

7. It is possible that some of the design flaws, particularly the lack of rebar detailing in the joint region, could have been identified during this peer-review.
8. This accident reemphasizes the lesson that public safety should never be compromised simply to showcase the application of a new construction technology, even though the technology itself may have been shown to be safe in prior applications. Any construction area is by nature hazardous to the public.
9. The use of a concrete truss for aesthetic reasons unnecessarily introduced complications related to pre-stressing and cold joints in the bridge, both of which likely played key roles in the collapse of the bridge.

Lessons & Recommendations from NTSB Investigation

1. Requirement should be developed that concrete bridge structures be designed with reasonable estimates for interface shear demand, the cohesion and friction contributions to interface shear capacity, and the clamping force across the interface shear surface.
2. Bridge design specifications need to address redundancy for concrete bridge designs in the future.
3. The peer-review process of design/construction drawings should always be done rigorously.
4. The structural cracks were reported to be active and developing every day after relocation. The construction project should be suspended after noticing the cracking issue. The construction area should be closed to the public immediately.

Thank You Very Much!