Experimental Evaluation of Overheight Impact Damage and Repair Effectiveness in Prestressed Concrete Bridge Girders

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Between 2013 and 2018, more than 78,000 collisions between motor vehicles and fixed bridge objects were reported in the United States, with overheight truck impacts identified as the predominant accident type.[1] While many overheight collisions result in minor damage or cosmetic repairs, a smaller subset leads to severe structural damage, extended closures, and, in some cases, loss of load-carrying capacity that requires girder replacement.

Overheight truck impacts impose a substantial financial burden even when bridges remain standing. A Washington State Department of Transportation (WSDOT) assessment reported repair costs exceeding $14,000 per linear foot of prestressed girder (2025 dollars).[2] For typical multigirder highway bridges, total repair expenditures can rival or exceed the costs of full replacement once traffic control, emergency contracting, and user delays are considered. In many cases, bridge owners are compelled to rapidly make repair-versus-replacement decisions with only limited information regarding the residual capacity and long-term performance of the damaged structure.

Despite the frequency, cost, and operational consequences of overheight vehicle impacts, the experimental literature addressing prestressed concrete bridge girders under impact loading is sparse. Most existing impact research focuses on small-span reinforced concrete beams tested under drop-weight or pendulum-type impactors. While those studies provide insights into local shear mechanisms and contact-force histories, their applicability to full-scale prestressed girders is limited due to differences in span length, prestressing-induced force states, composite deck interaction, restraint conditions, and system-level load redistribution. More importantly, small-scale vertical drop tests do not reliably capture the dominant mechanisms observed in overheight impacts in the field: partial strand rupture, asymmetric loss of prestress, torsional–flexural interaction, and progressive stiffness degradation.

This article presents the experimental component of a pooled-fund program developed to fill that gap.[3] Full-scale prestressed girders representative of department of transportation inventories were subjected to controlled impact loading, followed by systematic residual capacity testing and repair evaluation. When combined with the companion analytical study, described in the Winter 2026 issue of *ASPIRE*®, this work supports a unified postimpact decision framework for bridge owners that may reduce the number of unnecessary bridge and girder replacements while maintaining public safety.

Experimental Design

Fourteen 46-ft-long Missouri Department of Transportation (MoDOT) Type 2 prestressed concrete girders were fabricated for the study. Twelve girders contained twelve 0.5-in.-diameter low-relaxation Grade 270 strands and were detailed in accordance with the ninth edition of the American Association of State Highway and Transportation Officials' *AASHTO LRFD Bridge Design Specifications*.[4] Two girders contained 16 strands to quantify the influence of prestressing level. Thirteen girders were detailed to achieve flexure-controlled behavior under design loading; one of the 12-strand girders included increased midspan shear reinforcement to investigate the mitigation of torsion-related cracking (**Fig. 1**).

The experimental program was executed in three operational phases. First, eight girders were subjected to lateral dynamic impacts under varying boundary conditions and impact intensities to replicate overheight vehicle collisions. Second, one girder was tested under quasi-static lateral loading to establish a baseline comparison between static and dynamic response and quantify the extent to which dynamic demands exceed static capacity. Third, 12 girders—2 as-built, 3 damaged, and 7 repaired—were tested in four-point bending to quantify residual flexural strength and to validate repair effectiveness.

Lateral Impact Simulator and Impact Scenarios

Lateral impacts were delivered using a purpose-built impact simulator consisting of a 60-ft-long elevated steel track, of which a 22-ft length was flat and 38 ft was inclined (**Fig. 2** and **3**). The impact bogie had an empty weight of 2800 lb and was designed to withstand the AASHTO LRFD specifications' equivalent static force of 600 kip. The steel frame—measuring 7 ft by 3 ft—was configured to accommodate removable concrete slabs for weight augementation. A steel bumper at the front contained load cells for measuring the impact force. During this experimental program, up to three 1400-lb concrete slabs were installed. With three slabs installed, the total bogie weight reached 7000 lb. The

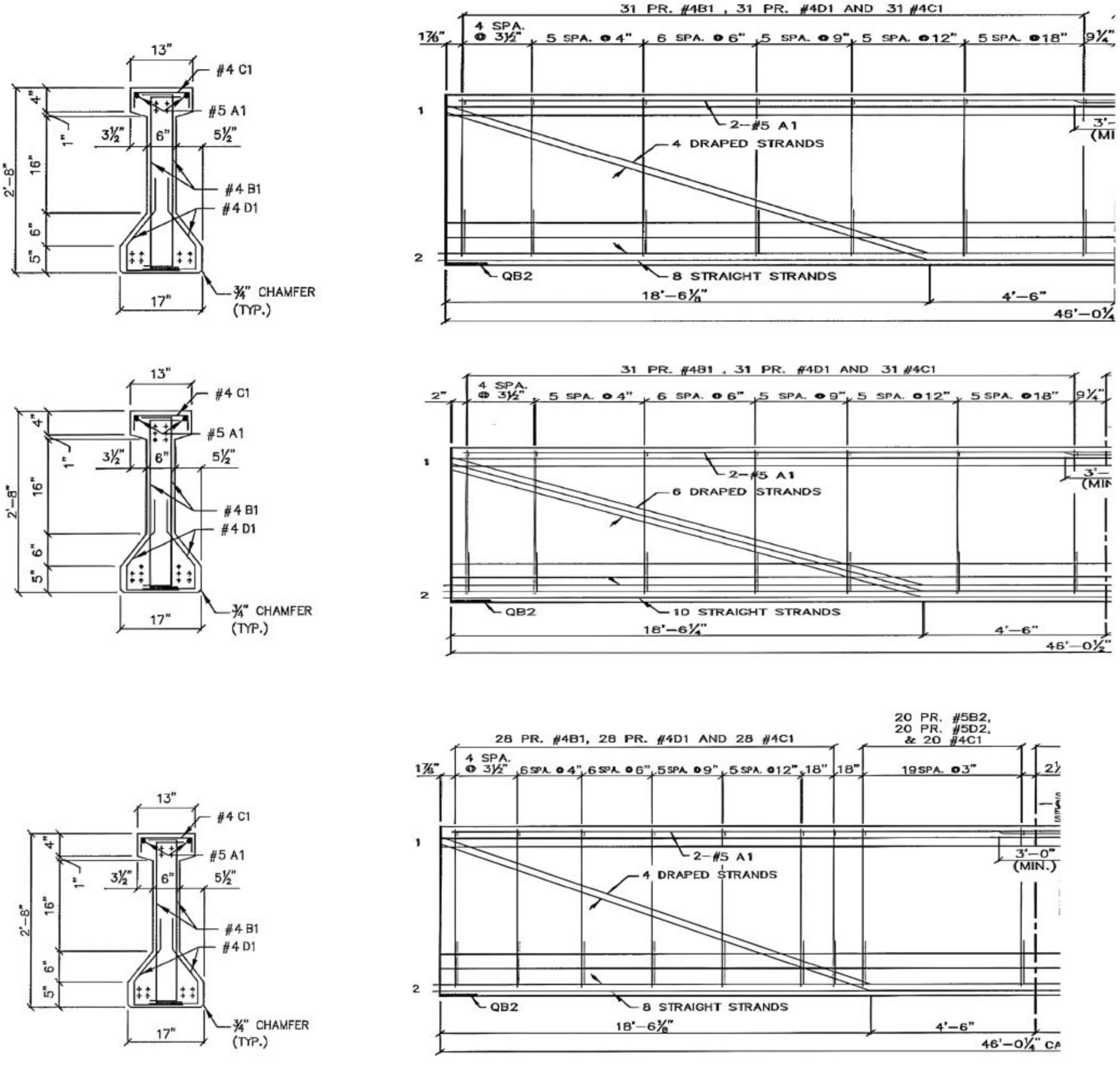


Figure 1. Test specimens' reinforcement details: 12 strands (top), 16 strands (center), and 12 strands with additional stirrups (bottom). All Photos and Figures: Missouri University of Science and Technology.

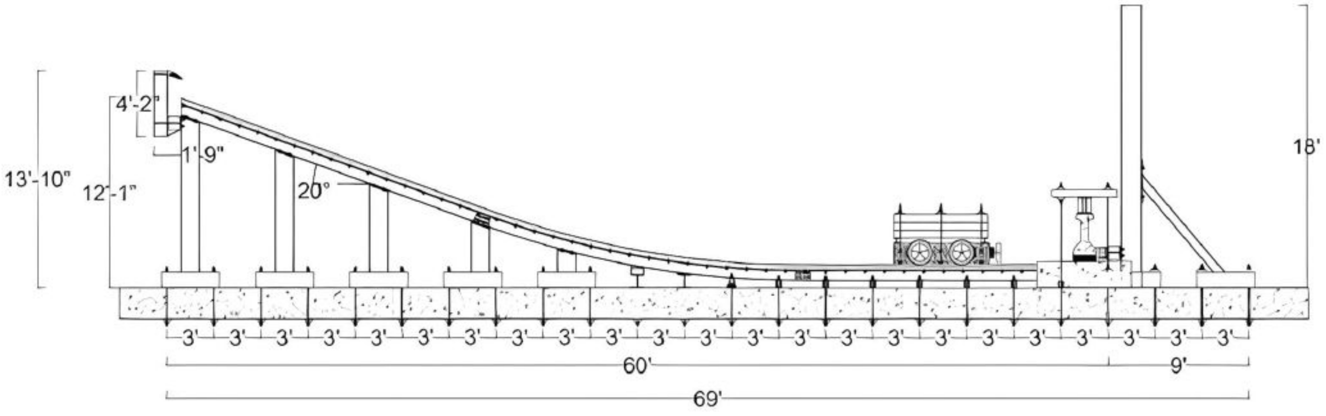


Figure 2. Elevation drawing of the impact testing setup with the 7000-lb steel impact bogie at the bottom of the ramp.

Figure 3. The impact bogie is ready to be released for impact testing of a girder specimen.

bogie, which was equipped with interchangeable impactor heads, served as the surrogate vehicle, providing repeatable kinetic energy under gravity-driven acceleration. A flat head simulated distributed contact consistent with trailer header-bar engagement, and a fork-shaped head reproduced concentrated, asymmetric contact representative of specialized equipment or offset strikes (**Fig. 4**). In all tests, the impactor head targeted the bottom flange near midspan, consistent with documented strike locations in overheight vehicle incidents. This configuration enabled systematic variation of impact intensity and contact geometry while maintaining controlled repeatability.

Figure 4. Impactor shapes: flat head (top) and fork head (bottom). These impactors simulate typical tractor trailer and oversize equipment impacts, respectively.

Boundary Conditions and System Restraint

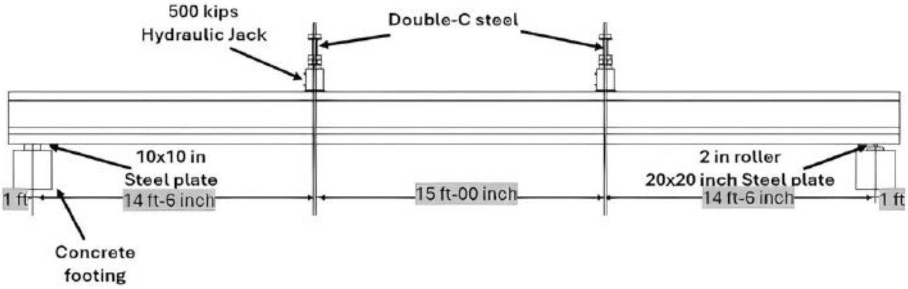
Because a girder in service is restrained by the bridge deck and diaphragms, two primary restraint configurations were employed. In the less-restrained configuration, girders were simply supported, with no top-flange restraint; this configuration allowed free lateral translation and rotation, which provided a lower-bound stiffness case. In the restrained configuration, a continuous steel restrainer was applied along the top flange to simulate the lateral-bracing effect of a reinforced concrete deck. In addition, two specimens were tested with intermediate steel diaphragms. Two built-up intermediate steel diaphrams were installed at the gider web, positioned at midspan with 9-ft spacing. The diaphragm locations are representative of those commonly used to distribute impact energy and stabilize load paths. Together, these testing configurations allowed investigators to quantify how deck restraint and diaphragms influence impact force transmission, lateral displacement demand, and damage localization.

Residual Flexural Strength of Impact-Damaged Girders

Following impact testing, the residual flexural capacities of three girders with approximately 25%, 50%, and 63% loss of prestressing strands on one side were evaluated. A four-point bending test (**Fig. 5**) with incremental monotonic loading and comprehensive instrumentation captured the force–deflection response, crack development, strand response, and failure modes of each girder.

The test results showed a strong, approximately proportionate relationship between loss of prestressing and

Figure 5. The four-point flexural test setup.



reduction of flexural strength. Girders with minor impact damage and no loss of prestress exhibited flexural capacities exceeding estimates based on AASHTO LRFD specifications by approximately 6% to 9%, consistent with typical conservatism in code-based predictions. Reductions of 25% to 63% strand loss resulted in approximately 25% to 62% reductions in flexural strength, respectively, a finding that confirms prestressing continuity as the controlling parameter for postimpact flexural capacity in these specimens (**Table 1**). The results also show that the AASHTO LRFD specifications accurately predict capacity if the damaged strands are discounted.

Repair Using Mechanical Strand Splicing and Carbon-Fiber-Reinforced Polymer Strengthening

Two repair strategies were experimentally evaluated for restoring flexural capacity after overheight impact–type damage: mechanical strand splicing using a proprietary system and externally bonded carbon-fiber-reinforced polymer (CFRP) composites. The strand-splicing program investigated four girders with strand losses between 17% and 33%. Repairs followed a consistent field-relevant sequence, including

- concrete removal and strand exposure,
- staggered splice installation,
- reintroduction of effective prestress by tensioning to approximately 70% of strand yield,
- patching with high-strength non-shrink grout with an average design compressive strength of 10,200 ksi, and
- epoxy injection into diagonal cracks along the web to restore stiffness and limit crack reopening (**Fig. 6**).

Each repaired specimen was then tested in four-point bending in the same manner as the undamaged control girders.

Table 1. Summary of the residual strength results

Girder number	Girder status	Strands		Girder compressive strength f_c', ksi	Flexural strength, kip-ft		Test strength/ AASHTO estimate
		Design number of strands	Damaged, %		Estimated per AASHTO LRFD specifications	Test	
G02	Undamaged control	12	0	10.2	1053	1118	106%
G14	Undamaged control	16	0	10.5	1270	1377	108%
G03	Damaged	12	25	10.6	852	855	100%
G04	Damaged	12	50	11.0	565	526	93%
G06	Damaged	16	63	11.4	476	521	109%

Figure 6. After the girders were impact tested, these steps were taken in the strand splicing repair.

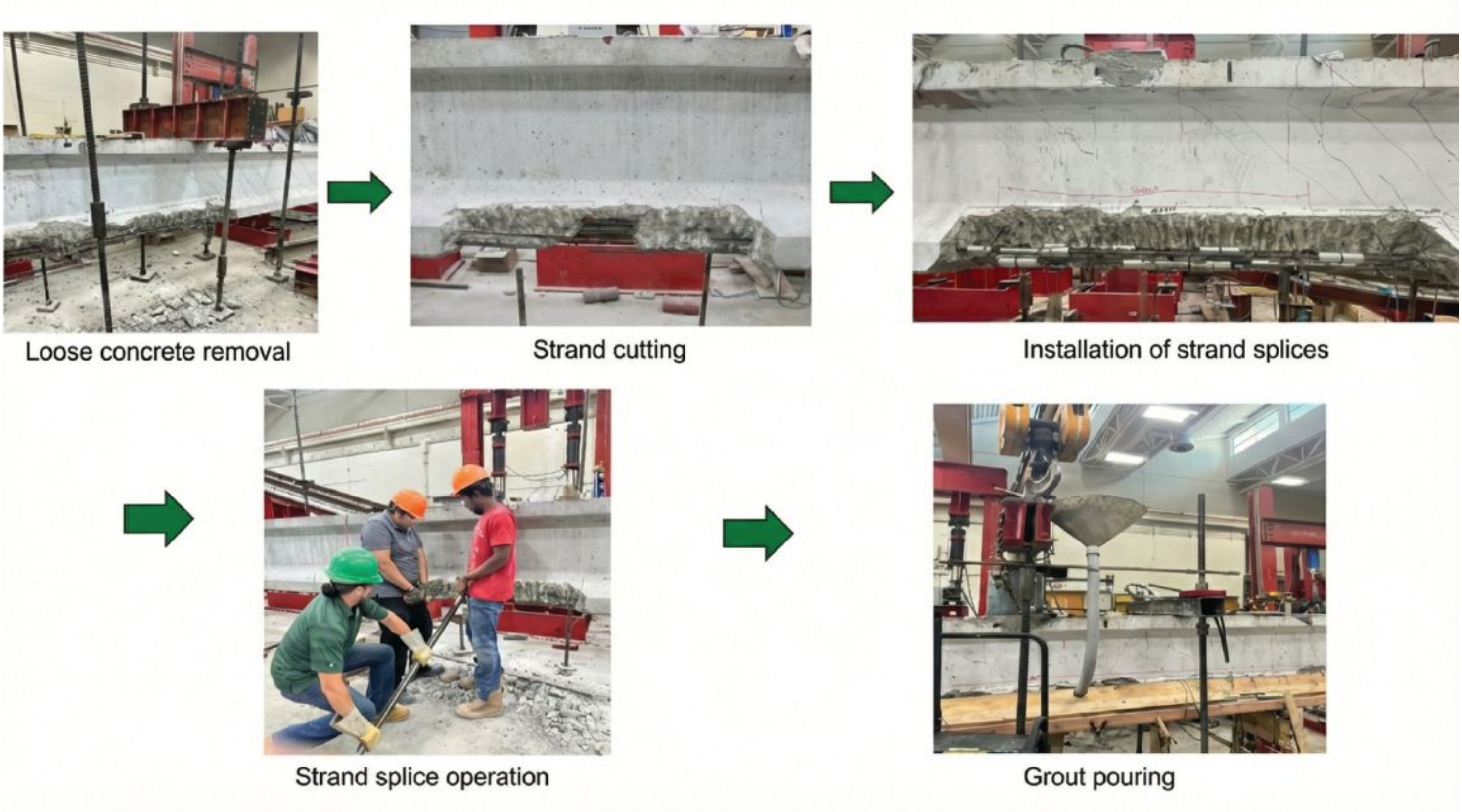


Figure 7. Failure of the carbon-fiber-reinforced polymer (CFRP) repaired girder; only the CFRP-repaired specimens were tested upside down.

Mechanical Strand Splicing

Testing results showed that mechanical splicing restored flexural strength effectively when strand loss is moderate and splice-region confinement is adequate. Girders with 17% strand replacement reached 84% of the control girder's strength and girders with 25% strand replacement reached approximately 95% of the control girder's strength, with initial stiffness and crack development comparable to the undamaged (control) specimen. As the percentage of spliced strands increased, performance became more sensitive to splice-region detailing and local stress concentrations. The impact-damaged girder repaired at 33% strand loss reached about 60% of the control girder's capacity. The relatively lower postrepair capacity for this girder was influenced by cumulative preexisting damage and local cracking at splice chuck locations. In contrast, a companion 33% strand-loss specimen that included added confinement around the splice region reached more than 91% of the control girder's strength, demonstrating that confinement reinforcement can mitigate premature splice-related failures and improve splice integration at higher damage levels. Instrumented tensioning of the spliced strand also showed that the manufacturer's torque-based tensioning procedure overestimated strand strain by approximately 18% on average, indicating the need for improved field calibration and quality-control checks when applying this method in practice.

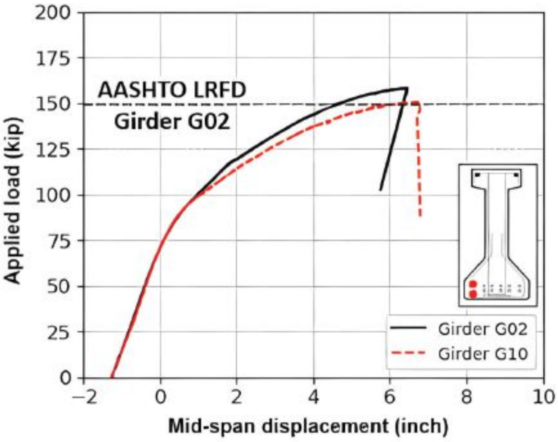
CFRP Strengthening

Externally bonded CFRP composites were evaluated as a flexural strengthening strategy for girders with 17% and 33% strand loss. No strand splicing was done as part of this work. A wet layup system designed in accordance with the American Concrete Institute's *Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures* (ACI PRC-440.2-17)[5] was installed with longitudinal plies (individual layers of fiber reinforcement) on the soffit and U-wrap anchorage to mitigate debonding. Successful performance required thorough surface preparation, including crack injection, patching, corner rounding, and appropriate surface roughness. Pull-off testing[6] indicated excellent bond quality, with failure occurring within the concrete substrate rather than at the interface or within the CFRP system. Under four-point bending, both CFRP-repaired girders exhibited brittle failure initiated by U-wrap rupture followed by longitudinal ply debonding (**Fig. 7**). Despite the brittle failure mode, strength restoration was substantial: the 17% damage girder and the 33% damage girder exceeded the control girder's strength by 23% and 16%, respectively (**Fig. 8**). These results confirm that a CFRP system can restore and even increase flexural strength for moderate damage levels, provided that anchorage and bond quality are properly controlled; however, the repair may shift the failure mode from ductile flexure to anchorage-controlled brittle behavior.

Conclusion

This experimental program provides a practice-relevant dataset linking overheight impact–type damage states, loss of prestress due to strand breakage, and repair effectiveness in full-scale prestressed concrete girders. The testing confirms that postimpact flexural performance is governed primarily by the magnitude of prestress loss, with strength and stiffness reductions tracking strand damage in an approximately proportionate manner over the range studied. Minor impact damage without strand loss produced capacities slightly above AASHTO LRFD specifications' predictions, whereas asymmetric damage levels representative of overheight strikes reduced flexural capacity by

Figure 8. Force-deflection curve of repaired girder G08 (33% strand loss with carbon-fiber-reinforced polymer strengthening) and undamaged control girder G02.



approximately 25% to 62% for strand losses between 25% and 63%.

Repair tests establish that both mechanical splicing and CFRP strengthening can be effective, but their reliability depends on the damage mechanism caused by the overheight impact. Strand splicing is most appropriate when the restoration of prestressing continuity is essential; it can recover 84% to 95% of baseline strength for moderate strand loss, and it can recover more than 90% of baseline strength for higher strand losses when splice-region confinement is included. CFRP strengthening can restore and even exceed baseline flexural strength of girders with moderate strand loss.

Most importantly, the experimental evidence closes a long-standing gap in bridge engineering practice by providing full-scale confirmation of how overheight impact–type damage and asymmetric loss of prestress affect residual flexural strength and repair outcomes.

Acknowledgments

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