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October 4, 2021

David R. Lujan, STR
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P.O. Box 1663
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Dear Mr. Lujan:

Please find enclosed the NMSU inspection documentation for the Los Alamos Canyon Bridge. The documentation includes the following:

1. 2021 NMDOT Bridge Inspection Report including Element Level Data Collection (in digital formats prepared by NMSU) – conforms to the National Bridge Inspection Standards and AASHTO Manual for Bridge Element Inspection
2. 2021 Supplemental Report (in digital formats prepared by NMSU) – provides detailed information related to current condition of major bridge components
3. 2021 Inspection Pictures (in digital format prepared by NMSU)
4. 2021 Delamination Map (in digital format prepared by LANL)

During the 2021 inspection, three critical findings were reported to our LANL contact, Mr. Jonathan Stein, by text and / or email on June 26, 2021.

1. The south joint of the bridge had a modular section of the joint that could potentially come loose during the passing of vehicles. This was reported as a critical finding for safety. If the modular section becomes dislodged or deformed it could pose a serious hazard that could result in a punctured tire to vehicles, motorcyclists and/or bicyclist causing drives/riders to lose control.
2. The bracket plate located at the north end of the bridge on the pedestrian walkway was corroded through and provides little protection to pedestrians and bicyclists. This was reported as a critical finding for safety to prevent injuries to people.
3. The north approach rail has three missing posts. These posts help to ensure that traffic is redirected and the energy is absorbed by the rail. This was reported as a critical finding for safety. Additionally, the approach rail is on a curve with a nearby drop off. Immediate repair is recommended.

Based on the 2021 inspection, the bridge deck is rated in "fair" condition. The chain drag performed on the deck identified several areas with delamination that are concentrated near the expansion joints, in the closure joint of the deck near the bridge centerline, and at the south end of the northbound lanes. The chain drag performed during the 2021 inspection revealed 215,333 sq. in. (1495 sq. ft.) of delaminations and patched areas (not including the sidewalk). This is approximately a 600% increase from 2019. It is recommended that the delaminations and spalls with exposed rebar be repaired. The "delaminated area" map for the 2021 inspection is provided in the supplemental report.

The superstructure is rated in "fair" condition due primarily to moderate to heavy corrosion, with section loss, of the superstructure elements. The floor beams including the outriggers and the spandrel girders of the Los Alamos Canyon Bridge are classified as fracture critical members. The National Bridge Inspection Standards (NBIS) defines a fracture critical member as a steel member in tension or with a tension element whose failure may cause a portion of or the entire bridge to collapse. The NBIS requires that fracture critical members be visually inspected within "arm's length" to assure the structural integrity of the bridge. During the 2021 inspection, the NMSU team used the underbridge access unit to reach the fracture-critical members. Particular attention was given to the connections of the spandrel girders and floor beams for signs of deterioration, damage, and distortion. The tension areas of the floor beams (including outriggers) and spandrel girders were also checked, particularly for corrosion, section loss, and fatigue cracks. Due to the corrosion and section loss on the outriggers, local failures are possible. In the inspection of the arch rib members, areas with corrosion and section loss were found on the top flange plate and bottom flange angles. The arch column to arch rib connections are corroded with pack rust. Corrosion / pack rust is also present at the corners between the plates of the built-up columns where the paint does not thoroughly cover the steel. The steel protective coating (paint) is in fair condition; however, paint failures are progressing leading to corrosion of the structural members. In general, the protective coating failures and corrosion in the affected locations continues to increase.

The substructure is rated in "poor" condition, specifically due to the condition of the abutments. The abutment concrete continues to degrade, particularly on the south end. The full width of the south abutment has numerous defects including cracking, delaminations, spalling, leaching, efflorescence, and corrosion of the reinforcement is evident from staining on the concrete. Additionally, the anchor bolts at the south abutment are in contact with the bearing device due to transverse movement in the east direction. Crack patterns and bridge seat surface measurements indicate minor settlement of the north abutment towards the west side of the bridge. The piers have numerous defects including cracking, delamination, spalling, efflorescence, rust staining, salt build up, and abrasion. The cracks have continued to propagate and increase in width and are characterized as moderate to wide cracks. Some cracks were previously sealed with epoxy but the cracks have progressed through the epoxy at several locations.

It is recommended that the south and north expansion joints be repaired or replaced. To accommodate the significant thermal movements experienced by a bridge of this size, the recommended types of joints are finger joints or modular expansion joints, the latter of which is currently being used. Due to possible misalignment of the "fingers" and increased water leakage through the joint, the finger joint type is not recommended for the Los Alamos Canyon Bridge. Installation of an approach slab may improve the transition on/off the bridge and help to minimize joint damage. It is also recommended that the use of "jointless" bridge technologies be investigated to effectively move the joint away from the abutment areas. This alternative could potentially improve the approach-to-bridge transitions, decrease the amount of water leaking through the joints and reaching the abutment, and reduce equipment-caused damage (e.g., snow plowing). It is imperative that proper design and installation procedures be followed for all joints. To gain a better understanding of the bridge behavior (specifically thermal movement) throughout the year, installation of a network of sensors at the abutment areas and periodic monitoring of the measured deformations is recommended. The bridge deformations collected throughout the year may provide meaningful information regarding the global movement of the bridge that is leading to problems with the expansion joints.

It is recommended that the configuration of the pedestrian rail be improved to meet the AASHTO LRFD Bridge Design Specifications. LRFD Sections 13.8 and 13.9 provide guidelines to protect individuals from falling through. In general, openings between horizontal or vertical members on pedestrian railings must be small enough to prohibit a 6-inch sphere from passing through in the lower 27 inches. For the portion of pedestrian railing that is higher than 27 inches, the openings should be spaced to prohibit an 8-inch sphere form passing through. Repair of the bridge rails is also recommended including repair of damaged concrete, replacement of missing / damaged anchors at the metal bridge rail connections to the concrete barrier rails, and repainting of the metal bridge rails.

Based on the 2021 inspection findings, the immediate, short-term, and long-term recommendations are summarized below:

- Immediate – 1. Install drainage system on west side of pedestrian walkway. 2. Repair north approach guardrail. 3. Upgrade pedestrian rail to current standards. 4. Replace joints using experienced personnel as needed due to damage caused by equipment and wear (potentially yearly). A request for proposal to qualified consultants for design and construction of the bridge joints and drainage system should be considered. The yearly replacement recommendation can be reconsidered based on new design and installation of joints and drainage system. 5. Overall repair of the outriggers is recommended with special attention to those with significant section loss due to corrosion.
- Short-Term – 1. Repair concrete on north and south abutments. 2. Repair the deck locations with delaminations and spalls, particularly those with exposed rebar. 3. Repaint and continue to clean movable bearings at abutments. 4. Repair concrete of CBR and repaint metal railing on top of CBR on east and west sides. 5. Monitor substructure elements for problems associated with soil erosion due to water runoff. 6. Monitor drainage at north and south joints. 7. Repaint the metal pedestrian rail. 8. Install erosion protection in areas surrounding abutments and piers, particularly in areas with undermining and scour.
- Long-Term – 1. Repair collision damage to metal railing on top of CBR on west side near north end of pedestrian fence and near the north end expansion joint. 2. Perform ultrasonic testing of pins at abutment, pier, and arch bearings. 3. Repaint arch rib and outriggers (including seated channel connections to pier columns and spandrel girder). 4. Monitor vertical alignment between deck and approach roadway on south end of bridge and check for associated joint damage. 5. Perform an in-depth inspection of the bottom connections of arch columns (including the rivets and angles) using rope access methods to ensure the connections are sound. 6. Measure section loss (or remaining section) on members with moderate to heavy corrosion. 7. Monitor and evaluate bridge movement (behavior) to provide data necessary to repair or replace the bearings and joints.

In addition to the critical findings, the NMSU team found several structural concerns during the 2021 inspection of the Los Alamos Canyon Bridge. The steel superstructure and bearing devices continue to corrode. The outrigger beams and stringer on the west side of the bridge are heavily corroded due to the lack of an adequate drainage system off of the pedestrian walkway. Additionally, the condition of the substructure continues to get worse, in particular the south abutment due to poor drainage of the water runoff. The substructure elements were previously repaired, however, the concrete repairs continue to deteriorate. In addition, the steel protective coating on the west arch rib is deteriorating due to the poor drainage. The bridge also experiences significant and atypical movement (likely due to temperature) that continues to distress the expansion joints (particularly on the south end). Since the bridge is a critical link between the City of Los Alamos and the LANL, and the bridge services a large volume of traffic, it is important that the issues summarized in this report be addressed. It is noted that the overall rating of the bridge is based on the lowest rated element. At this time the bridge condition is controlled by the bridge substructure (condition rating = 4) followed by the deck and the superstructure (condition rating = 5). Repair of the deteriorated elements to current specifications for capacity and durability could result in a better overall condition rating.

Following this letter, you will find recommendations for updating the load rating or its assumptions. Additionally, a discussion of issues that would necessitate an immediate review/update of the load rating is included. This is followed by information and recommendations for inspection of the bridge following a seismic event for the normal operation of the structure (i.e., vertical loading). These recommendations do not address the operation under extreme events (i.e., lateral loading). These recommendations are based on the findings of the Load Rating and Seismic Screening reports provided by LANL.

For the 2022 inspection, it is recommended that a climbing team be included with an expanded scope of work. In addition to inspecting the arch and fracture critical elements, each column and pier should be thoroughly inspected by the climbing team not reachable by the underbridge access unit. Additionally, section loss on the critical elements should be evaluated in more detail.

Should you have any questions regarding this letter, please get in touch with either David Jauregui at 575-646-3801 (work), 915-346-5170 (cell), or by e-mail at jauregui@nmsu.edu or Brad Weldon at 574-631-1640 (work), 575-993-4323 (cell), or by email at bweldon@nmsu.edu. Thank you for your attention.

Sincerely,

Brad Weldon

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12/8/2021

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Los Alamos Canyon Bridge Inspection and Rating Report

Floor Beams and Outriggers

Table 1 summarizes the inventory rating (RF_i) and operating rating (RF_o) factors for the Strength I limit state determined by Bohannan Huston, Inc. (BHI) in the evaluation of the floor beams and outriggers. In addition, the condition states of the bridge elements determined in the 2019 inspection by NMSU are reported in the table. The rating factors for moment of the floor beam were controlled by positive bending near the centerline of the bridge width and by the local buckling resistance of the compression flange which is a non-compact element (i.e., $\lambda_{pf} < \lambda_f < \lambda_{rf}$). Note that the $b_f / 2t_f$ ratio exceeded 12, however, this limit applies to welded not riveted members. For shear, the floor beam rating factors were controlled by shear near the spandrel beams and by the end panel shear resistance (i.e., no tension field action) of the floor beam.

Table 1. Rating factors and condition states of floor beams.

Component	RF_i, RF_o for Strength I *		Condition State
	Moment	Shear	
FB#3	1.03, 1.34	0.54, 0.71	Good condition – paint peeling on top flange of outrigger (west side); distortion of floor beam bottom flange (near girder G2 on east side)
FB#7	1.12, 1.45	0.57, 0.74	Fair condition – paint peeling and minor corrosion on top flange of outrigger (west side); distortion of floor beam flange (near midspan and under stringer S5 near girder G2 on east side); minor corrosion on top flange of outrigger (east side)
FB#15	1.33, 1.73	0.64, 0.83	Good condition – paint peeling and minor corrosion on top flange of outrigger (west and east sides)
FB#22	0.99, 1.28	0.56, 0.73	Fair condition – minor corrosion on top and bottom flanges of outrigger (west side); paint peeling under stringer S2 near girder G1 on west side); poor paint job between stringers S2 and S4; minor corrosion on top flange of outrigger (east side)
FB#27	0.82, 1.06	0.71, 0.92	Fair condition – minor corrosion on top and bottom flanges of outrigger (west side); pack rust on bottom flange connection between outrigger and spandrel beam (east side)

* Note: Controlling RF_i and RF_o values for outrigger beam equaled 1.47 and 1.91 (for moment), and 1.45 and 1.89 (for shear)

ACTION: Since the floor beams are in good condition and no signs of corrosion were observed on the floor beam flanges or web between the spandrel beams (i.e., no section loss), there is no immediate need to rerate the floor beams for moment or shear. Furthermore, deterioration of the floor beam elements is not anticipated since these elements are not directly exposed to rain, snow, or water runoff.

For the outriggers, the critical locations are at the end connection to the east spandrel beam G2 for moment and at the exterior stringer S6 for shear. The web and flange proportions were met for the outriggers. The moment capacity was controlled by flange yielding of the compression flange which is a compact element (i.e., $\lambda_f < \lambda_{pf}$) and the shear capacity was controlled by shear buckling with tension field action.

ACTION: The outriggers at four of the floor beams listed in Table 1 (FB#3, FB#7, FB#15, and FB#22) have minor corrosion on the top flanges on the west and/or east sides, mainly in the area under the exterior stringers. The top flange corrosion is not a significant concern for bending since the moment capacity is more critical at the spandrel beam connection location. The two outriggers at FB#22 and FB#27 also have minor corrosion on the bottom flange and the outrigger at FB#27 has pack rust (on east side); however, no corrosion was observed on the outrigger webs. There is no immediate need to rerate the outriggers, however, it is recommended that section loss be measured on the outrigger bottom flanges with pack rust.

Columns

Table 2 summarizes the Strength I rating factors determined by BHI and the condition states determined by NMSU for the pier (PC), skewback (SC), and arch (AC) columns. The rating factors for the columns considered axial force and bending moment interaction and the member capacities were controlled by local buckling of the non-compact compression plate elements (i.e., $\lambda_{pf} < \lambda_f < \lambda_{rf}$).

Table 2. Rating factors and condition states of columns.

Component	RF_i, RF_o for Strength I Interaction	Condition State	
		East Side	West Side
PC#1	1.10, 1.43	Good	Good
PC#2	1.21, 1.57	Good	Good
SC#1	2.01, 2.60	Good	Good
AC#1	1.41, 1.88	Good	Fair
AC#2	1.03, 1.33	Good	Fair
AC#3	0.90, 1.17	Good	Fair
AC#4	0.80, 1.04	Good	Fair
AC#5	0.62, 0.81	Good	Good
AC#6	0.52, 0.67	Good	Good
AC#7	0.72, 0.93	Good	Good
AC#8	0.92, 1.19	Good	Good
AC#9	0.52, 0.67	Good	Fair
AC#10	0.55, 0.72	Good	Fair
AC#11	0.70, 0.91	Fair	Fair
AC#12	0.81, 1.05	Fair	Fair
AC#13	0.95, 1.24	Fair	Fair
AC#14	1.37, 1.78	Fair	Fair
SC#2	1.63, 2.19	Good	Good
PC#3	1.24, 1.60	Good	Good
PC#4	1.18, 1.53	Good	Good

ACTION: As shown in Table 2, arch columns #1 through #4 were rated in fair condition on the west side arch due to corrosion at the interior angles connecting the plates. Since the angles are positioned in the interior of the built-up section, quantifying the extent of corrosion is difficult. However, the corrosion has not progressed to the outside faces of the plate elements and thus, there is no immediate need to rerate these four columns. However, the use of advanced techniques to determine the level of corrosion in the interior angles is recommended. Arch columns #9 through #14 on the west side were also rated in fair condition due to corrosion at the bottom connections of the columns and/or corrosion of the arch rib top flanges at these connection locations. Arch columns #10 through #14 on the east side arch were also rated in fair condition due to corrosion of the arch rib top flanges at the bottom column connections. The corrosion observed at columns #9 through #14 may reduce the stiffness of the column connection to the arch rib top flanges which was assumed as a "fully rigid connection" in the load rating study conducted by BHI. Since the assumed connection stiffness results in the worst case scenario (i.e., lowest rating factors), there is no immediate need to rerate these six columns. However, an in-depth inspection of the bottom connections of arch columns #9 through #14 (including the rivets and angles) on the north side of the arch is suggested using rope access methods to ensure the connections are sound.

Spandrel Beams

Table 3 summarizes the Strength I rating factors determined by BHI and the condition states determined by NMSU for the spandrel beams. The rating factors for moment of the spandrel beam were controlled by positive bending near midspan and negative bending near the column locations of the 62 ft. end spans. The spandrel beams are composite with the reinforced concrete deck in Bays 1-2 and 27-28, and non-composite in Bays 5-6 and 22-23. In the positive moment region of the non-composite section, the local buckling resistance of the compression flange which is a non-compact element (i.e., $\lambda_{pf} < \lambda_f < \lambda_{rf}$) controlled the moment capacity. For shear, the spandrel beam rating factors were controlled by shear buckling with no tension field action.

Table 3. Rating factors and condition states of floor beams.

Location	RF_i for Strength I *	RF_o for Strength I *	Condition State
	+Moment	-Moment	
Bays 1-2 and Bays 27-28	1.37	1.78	+Moment (composite section) – fair condition due to pack rust at outrigger connections to spandrel beam -Moment (composite section) – good condition
Bays 5-6 and Bays 22-23	0.67	0.87	+Moment (non-composite section) – fair condition due to pack rust at outrigger connections to spandrel beam -Moment (non-composite section) – good condition

* Note: Controlling RF_i and RF_o values for spandrel beam equaled 1.60 and 2.07 (for shear)

ACTION: Although the spandrel beams were rated in fair condition at midspan of the 62 ft. end spans (due to pack rust at the outrigger connections), only freckled rust (i.e., no section loss) was observed on the bottom flanges of the spandrel beams at these midspan locations. The top flanges and web have isolated areas with paint peeling but minimal corrosion was observed. Thus, there is no immediate need to rerate the spandrel beams for moment or shear.

Stringers

Table 4 summarizes the Strength I rating factors determined by BHI and the condition states determined by NMSU for the stringers. The stringer rating factors for moment were controlled by negative bending between Bays 2-3, near midspan of Bay 8, and near midspan of Bay 27. The moment and shear capacities of the stringers were controlled by plastic behavior (i.e., plastic moment and shear yielding).

Table 4. Rating factors and condition states of stringers.

Location	RF_i, RF_o for Strength I *	Condition State
	Moment	
Bays 2-3	1.01, 1.35	-Moment of interior stringer (composite section) – good condition, paint peeling on top flange of interior stringers
Bay 8	1.79, 2.41	-Moment of exterior stringer (non-composite section) – good condition, paint peeling and freckled rust on bottom flange of exterior stringer on east side
Bay 27	1.17, 1.57	-Moment of interior stringer (composite section) – good condition, minor deterioration

* Note: Controlling RF_i and RF_o values for spandrel beam equaled 2.01 and 2.71 (for shear)

ACTION: Since the stringers are in good condition and signs of only freckled rust were observed (i.e., no section loss), there is no immediate need to rerate the stringers for moment or shear. Furthermore, the exterior stringer in Bay 8 is more directly exposed to rain, snow, or water runoff but the rating factors exceed those of the interior stringers.

Arch Ribs

The controlling rating factors for Strength I determined by BHI for the east arch rib were RF_i = 1.19 and RF_o = 1.80 for the maximum axial case and RF_i = 0.85 and RF_o = 1.11 for the maximum moment case. The web and flange proportions and the slenderness limits were met for the arch ribs. The moment capacity was controlled by elastic lateral torsional buckling and the axial capacity was controlled by inelastic flexural buckling.

ACTION: Findings from NMSU's latest inspection of the arch ribs included the following: (1) west arch – heavy corrosion on top flange on north side of arch rib, moderate corrosion on bottom flanges on south side of arch rib; and (2) east arch – heavy corrosion on top flange on north side of arch rib. As previously discussed, the south side of the east arch rib had the lowest rating factors and this portion of the arch is currently in good condition. There is no immediate need to rerate the arch ribs, however, it is recommended that section loss be measured on both ribs using rope access methods.

Post Seismic Event Assessment Recommendations

Based on the findings of the Seismic Screening Report by Bohannan Huston Inc., the seismic performance of the bridge is governed by the columns. The flexural column capacities are limited by local buckling of the non-compact or slender built-up plate elements. As a result, flexure failure will not be ductile where yielding of the cross-section allows for significant displacement (and energy dissipation) of the member prior to a catastrophic failure. This can potentially lead to a progressive collapse where as a column fails, the load is transferred to other members. As the load is transferred, these members are overloaded causing additional non-ductile failures.

Two seismic events were evaluated in the Seismic Screening Report, a lower level and an upper level. In both seismic events, the bridge was found to have a strong beam-weak column condition where the global strength of the frame is controlled by the strength of the columns. This condition is highly susceptible to creating a "weak story" collapse mechanism. Because the column members' capacities are controlled by local buckling, a non-ductile failure condition exists and the failure of the columns would limit the deflection capacity and energy dissipation of the structure.

Under the lower level seismic event, the floor beams, spandrel beams, arch ribs and the majority of the columns were found to be adequate. However, arch columns No. 7 and 8 on the west and east face of the bridge as well as the tops of the skewback columns on the east face of the bridge were found to exceed their capacity in flexural-axial interaction. For the upper level seismic event, the floor beams, spandrel beams, arch ribs, and some columns were found to be sufficient. However, the majority of the columns were found to exceed their capacity under flexural-axial interaction. The column members' capacities are controlled by local buckling (non-compact section). This will lead to a non-ductile failure condition and are susceptible to a progressive collapse of the structure. The following columns exceed the flexural-axial interaction limits during the upper level earthquake event:

East skewback columns 1 and 2, arch columns 7, 8, 9, 10, 11 and 12
West skewback columns 1 and 2, arch columns 7, 8, 9, 10, 11 and 12

Based on the findings presented in the Seismic Screening Report, following a seismic event, the following steps are recommended to assess the state of the bridge:

- A cursory, visual inspection from ground level surrounding the bridge to identify any structural damage.
    - General walk around the bridge.
    - Check vertical, lateral, and longitudinal alignments.
    - Evaluate settlement and damage to substructure elements.
    - Particular attention should be given to the arch columns, skew-back columns, column connections to the arch rib, and bearing devices. All damage should be noted, photographed and assessed.

  - Assess the damage to the bridge and determine if the damage warrants a structural review or if the bridge is safe to conduct a more in-depth, physical inspection.

- Using a rope access inspection team, all columns and the two arch ribs should be inspected for damage including evidence of local buckling, failure of or missing rivets of the built up section, and connection failures to the arch and/or bearing devices.
  - Damage should be noted, photographed and assessed (e.g., distortion, tear out, local buckling, failure of connectors, etc.).
  - If deemed necessary, a structural analysis should be conducted with consideration of the recorded damage to ensure the adequacy of each member.
  - Once the support structure (e.g., columns, piers, and abutments) of the floor system has been deemed stable and adequate for strength, a full inspection of the bridge is recommended.

- Using a rope access team and under-bridge access unit, a full bridge inspection of the bridge should be conducted.
  - The entire superstructure should be inspected. Particular attention should be given to rivets of the built up sections, splice connections, and non-redundant members.
  - If necessary, nondestructive methods should be employed to determine the level of damage.
  - Damage should be assessed, and if necessary, a structural review should be conducted. Load ratings should be re-assessed based on the recorded damage from the post-earthquake inspections.