06/10/2024

Diamond Creek Vineyards
Attn.: Nicole Carter
1510 Diamond Mountain Rd
Calistoga, CA 94515

RE: Bridge Conditions Review
ZFA Job No.: 21695

To Nicole Carter:

Conditions Review Update

This letter is in response to the Napa County Comments regarding the analysis of the headwalls and current conditions of the bridge. As requested, a representative from ZFA Structural Engineers visited the site on May 1st, 2024 to review the current condition of the subject Diamond Creek Vineyards Bridge. The foundation and stone construction conditions of the bridge were reviewed and compared to the original ZFA condition review findings, issued December 23rd, 2021. The observations and recommendations from the previous report appear to still apply, with the additional observation below.

1. The erosion undermining the west arch foundation has worsened since the original report, with measurements up to 27-inches horizontally perpendicularly underneath the stone arch foundation. This undermining could lead to loss of soil support of the bridge structure.

ZFA recommends that the undermined arch foundations be repaired to regain their full soil bearing support and protected from future erosion/scour. This repair could be conducted by structurally and completely filling the void under the undermined foundation walls with a concrete or soil slurry and covering the repair with rubble rock to reduce the risk of repeated erosion.

Any repair should be coordinated with the appropriate state and local jurisdictions. ZFA recommends these repairs to be commenced as soon as possible and completed prior to next winter's rainy season. Until this repair is in place, we recommend that the foundations and bridge be reviewed during each month from November to March to review for further erosion or signs of structural destress.

Load Rating Update

The Napa County comments also requested to update the previously provided load rating to include the entire bridge, including the unreinforced masonry (URM) head walls. Although the end wall thicknesses could not be directly observed or measured in the field, the wall thickness was assumed to be uniform to the parapet portion of the wall.

The wall was analyzed using an assume active loading of 40 pounds per cubic foot, and a soil surcharge load equivalent to an additional 2' of retained soil height was analyzed placed on the wall per AASHTO Table 3.11.6.4.2. The analysis concluded that the stress on the mortar was less than the allowable default lower-bound tensile strength per the ASCE 41-17 / Table 11-2a,

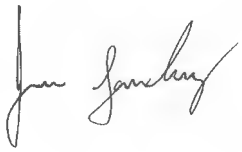
concluding that the head walls are adequate to support the vehicular surcharge loading and retained soil.

Closing

Once full soil bearing is restored to the undermined arch foundations, and mortar repairs have been completed, ZFA has shown that the Diamond Creek Vineyards Bridge can support the design truck required by the Napa County Road and Street Standards.

This conditions and load rating report is based on that which was plainly visible at the time of site review. The items discussed are subject to revision should more information become available. ZFA understands you may have questions regarding this report and are available for comment and explanations, please contact ZFA for clarification of any questions you may have. We look forward to assisting with any future design efforts for mitigation, repair, or strengthening, as needed, of the subject bridge structure.

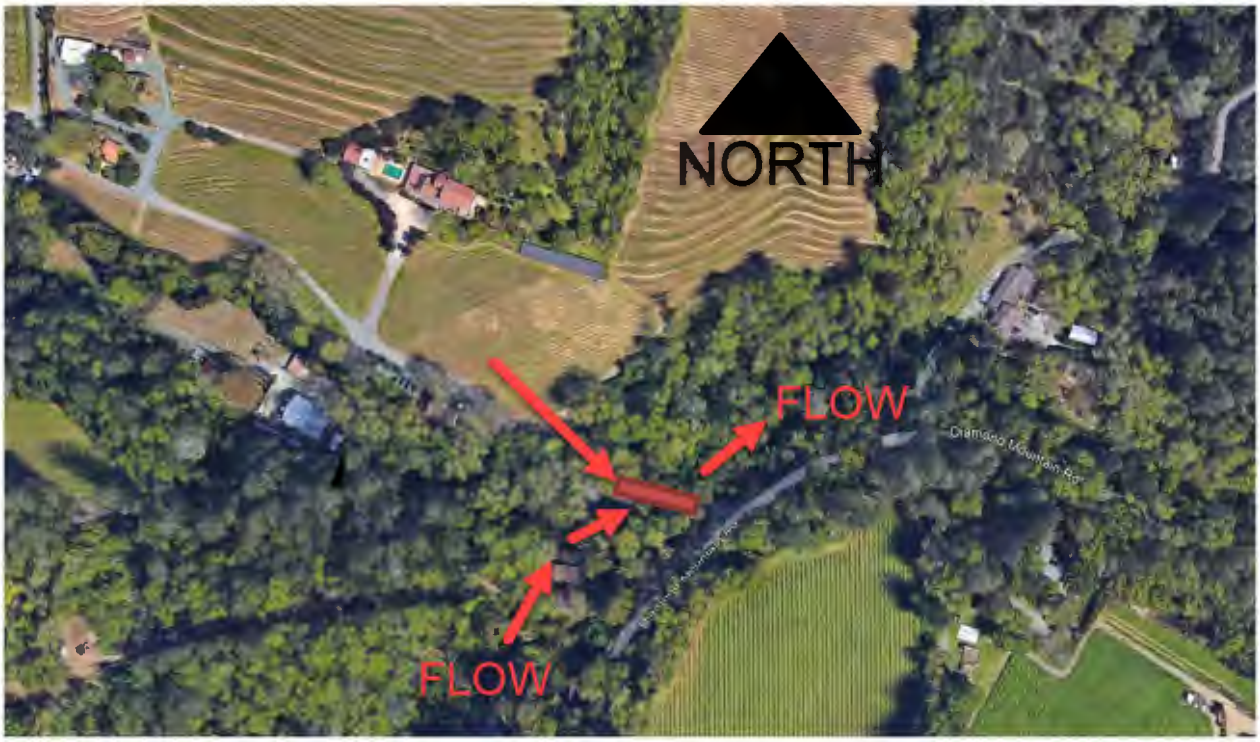
Sincerely,
ZFA STRUCTURAL ENGINEERS



Jesse Sanchez
Designer

Chris Meade, SE
Senior Associate

Appendix A: Arial View of Structure Location

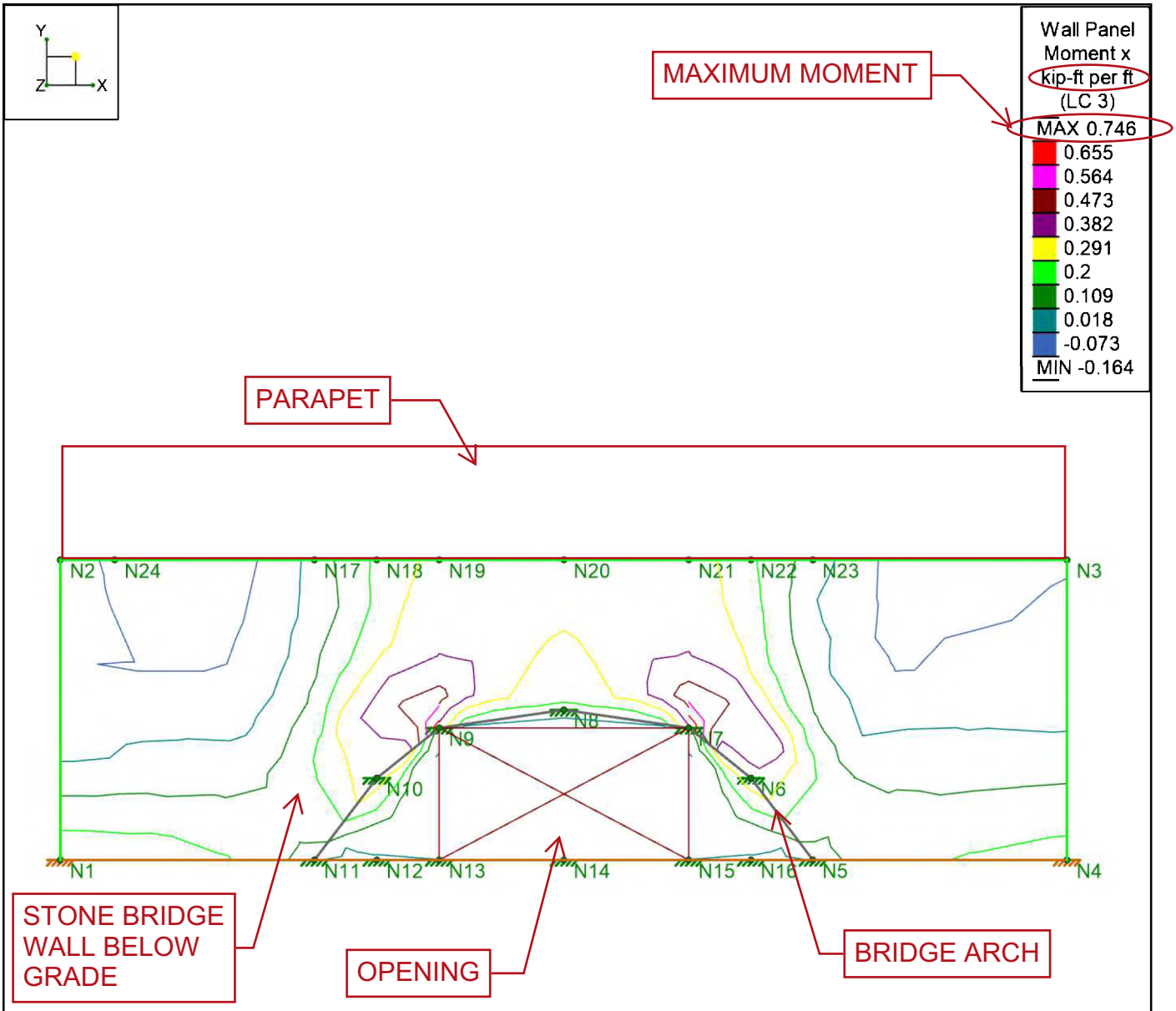


Appendix B: Site Photographs

Photo 1: West arch foundation

Photo 2 & 3: Scour depth at two locations under the west arch foundation

Appendix C – Calculations



Results for LC 3, Horizontal Only

IRISA	ZFA Structural Engineering		SK-1
	JesseS		Jun 07, 2024 at 03:29 PM
			Wall Check - Opening.r3d

Node Coordinates

	Label	X [ft]	Y [ft]	Z [ft]	Detach From Diaphragm
1	N1	0	0	0	
2	N2	0	5.5	0	
3	N3	18.5	5.5	0	
4	N4	18.5	0	0	
5	N5	13.83	0	0	
6	N6	12.6941	1.5	0	
7	N7	11.55	2.42	0	
8	N8	9.255	2.75	0	
9	N9	6.9625	2.42	0	
10	N10	5.81625	1.5	0	
11	N11	4.67	0	0	
12	N12	5.81625	0	0	
13	N13	6.9625	0	0	
14	N14	9.255	0	0	
15	N15	11.55	0	0	
16	N16	12.6941	0	0	
17	N17	4.67	5.5	0	
18	N18	5.81625	5.5	0	
19	N19	6.9625	5.5	0	
20	N20	9.255	5.5	0	
21	N21	11.55	5.5	0	
22	N22	12.6941	5.5	0	
23	N23	13.83	5.5	0	
24	N24	1	5.5	0	

Wall Panel Data

	Label	A Node	B Node	C Node	D Node	Material Type	Material Set	Thickness [in]	Design Rule	Panel/Spacing
1	WP1	N2	N1	N4	N3	Masonry	Gen Masonry	12	R2	72

RETAINED SOIL PRESSURE

Wall Panel Surface Loads (BLC 2 : Soil Load)

	Wall Panel Label	Direction	Top Magnitude [ksf, F]	Bottom Magnitude [ksf, F]	Start Location [ft]	Height [ft]
1	WP1	z	0	-0.22	0	0

SURCHARGE LOAD FROM VEHICLE = 2'x SOIL PRESSURE

Wall Panel Surface Loads (BLC 3 : LL)

	Wall Panel Label	Direction	Top Magnitude [ksf, F]	Bottom Magnitude [ksf, F]	Start Location [ft]	Height [ft]
1	WP1	z	-0.08	-0.08	0	0

SELF WEIGHT ADDED SEPARATELY

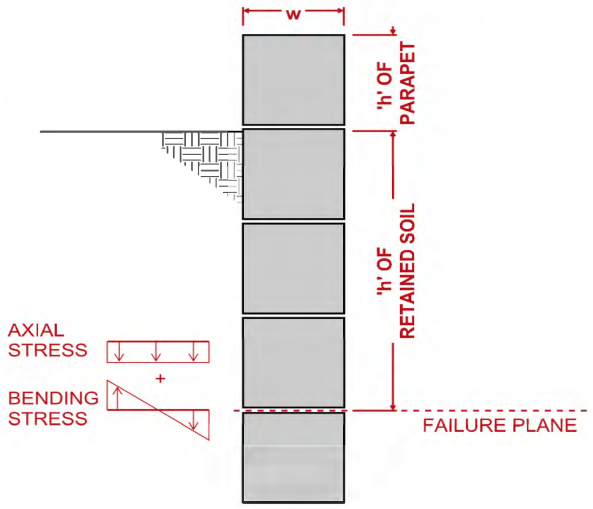
Basic Load Cases

	BLC Description	Category	Surface(Plate/Wall)
1	DL	DL	
2	Soil Load	EPL	1
3	LL	LL	1

Load Combinations

	Description	Solve	P-Delta	BLC	Factor	BLC	Factor	BLC	Factor
1	ASD W/ Surcharge		Y	1	1	3	1	2	1
2	ASD		Y	1	1	2	1		
3	Horizontal Only	Yes	Y	2	1	3	1		

Spandrel Walls as Gravity Retaining Walls



Find 'h' max such that the tension caused by bending minus the axial compression is equal to the rupture stress of grout

Assumed Soil Properties

ϕ =	30	deg
γ =	120	pcf
Eq Fluid Pressure =	40.0	pcf

Wall Properties

w =	18	in	
hp =	24	in	parapet
h =	**66.0**	in	retained soil (MAX)
	5.5	ft	
γ =	165	pcf	quarried limestone, AISC SCM 14th edition Table 17-12
fcr =	60	psi	2017 ASCE 41-17 Seismic Evaluation and Retrofit of Existing Buildin
Sx =	648	in^3	
A =	216	in^2	

Table 11-2a. Default Lower-Bound Unreinforced Masonry Strengths

Material	Solid Units	Hollow Concrete Units
Compressive strength[a]	600 lb/in.^{2}	1,000 lb/in.^{2}
Flexural tensile strength[b]	60 lb/in.^{2}	38 lb/in.^{2c} (95 lb/in.^{2})^{d}
Shear strength[e]		

[a] Clay f'_m is based on 2,100 lb/in.^{2} unit compressive strength and Type N mortar. Hollow concrete f'_m is based on 1,900 lb/in.^{2} unit net compressive strength and Type N mortar on face shells only.
[b] Values based on Portland cement/lime or mortar cement, Type N mortar.
[c] Ungrouted hollow concrete blocks.
[d] Solid grouting of hollow concrete blocks; may be interpolated for partial grouting based on net area.
[e] Strength shall be taken as 80% of shear strength values determined in accordance with Section 9.2.6 of TMS 402.

Wall Self Weight

h + hp =	90.0	in	
V =	19440	in^3	
	11	ft^3	
W =	1856	lbs	

Stress

Bending Stress

M =	8952	lb-in	=0.746k-ft*1000*12
M/Sx =	14	psi	
x 1.10	15	psf	

Axial Stress

W/A =	9	psi

Combined Stress =	7	psi
DCR =	0.11	

FROM RISA MODEL

INCLUDES THE PARAPET WEIGHT

OK

ZFA STRUCTURAL ENGINEERS

san francisco
silicon valley
sacramento
santa rosa
napa

Diamond Creek - Stone Arch Bridge Review

Calistoga, California

ZFA Project Number: 21695

December 23, 2019

Prepared For

Diamond Creek Vineyards

Calistoga, California

Prepared By

Austin Spinelli, Designer

Chris Meade, Associate

Kevin Zucco, Principal in Charge

Nicole Carter
DIAMOND CREEK VINEYARDS
1510 Diamond Mountain Rd
Calistoga CA, 94515

December 23rd, 2021

RE: Diamond Creek Bridge Review and Recommendations

Overview

At your request, ZFA has performed a general condition review and load rating analysis of the subject bridge, located at 1510 Diamond Mountain Rd, Calistoga. The roadway structure is located on private property and shared by multiple parcels. It is our understanding that the County is requiring the bridge be load rated as part of a Use Permit Modification for Diamond Creek Vineyards which is serviced by the bridge. This letter summarizes ZFA's findings from our review and analysis and provides recommendations for the existing roadway structure.

Description

On December 6th, ZFA visited the project site to review and document the historic masonry bridge structure. The bridge structure is comprised of a stone masonry barrel arch and spandrel walls with an engraving indicating the structure was constructed in 1885. The barrel arch spans approximately 9'-2" with a rise at the crown of approximately 4'-0", see Appendix B photos of the existing bridge. The masonry arch soffit has what appears to be the original lime-based mortar, but portions of the spandrel walls appear to have been repointed with a cement-based mortar.

Observations and Recommendations

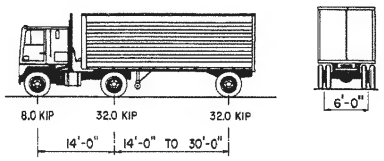
1. Erosion has removed soil support from under sections of the arch at each side of the bridge, partially undermining its support (Photo 5 & 6). No significant signs of distress were observed in the masonry bridge itself, but if erosion remains or is allowed to progress it could lead to progressive failure.
    - ZFA recommends that the undermined foundations be repaired to regain their full soil bearing support. This repair could be conducted by structurally filling the void under the undermined foundation walls with a concrete and covering the repair with rock to reduce the risk of repeated erosion. The base of the headwall should be repaired similarly. Any repair should be coordinated with the appropriate state and local jurisdictions including the County of Napa and the California Fish and Wildlife. While not an immediate danger, ZFA recommends these repairs to be engaged as soon as possible and that a civil or environmental engineer be engaged to coordinate this scope of work with the pertinent jurisdictions. Until this repair is in place we recommend that the foundations be reviewed after each significant storm event to review for further erosion under the arches.
2. The mortar between many of the stones is highly weathered. At one location the mortar was completely missing so that the stones were loose to the touch (Photos 7 & 8).
    - ZFA recommends that the entirety of the bridge be repointed with mortar. See Appendix D for masonry repair specifications. Furthermore, we recommend that the masonry structure be reviewed annually for weathering, movement, cracking, and deterioration. After the above repairs have been completed and three years

without additional required maintenance, this recommended review interval can be increased to once every three years.

Load Rating Analysis

Since original drawings of the bridge were not available, field measurements were recorded and used to calculate the strength of the existing masonry arch structure. The superstructure was then analyzed for conformance with the requirements of the current Napa County Road and Street Standards. These standards specify that existing bridges be evaluated and maintained for HS20-44 truck loading within the American Association of State Highway and Transportation Officials Standard Specifications for Highway Bridges, Edition 17 (HB-17) in addition to a 75,000 lb. fire apparatus.

The most widely used empirical assessment for stone arched bridges is the Military Engineering Experimental Establishment (MEXE) method originally developed by the British Military and adopted for civilian use under the Design Manual for Roads and Bridges CS 454. Because the geometry of this bridge is outside the applicable span range of the MEXE, the Pippard equation, which the MEXE method was derived from was utilized to calculate the provisional axle load. Then the reductions from the MEXE method were applied to adjust the capacity for the condition of the existing bridge. This rating analysis found that the masonry arch bridge could support an axle weight of 61,000 lbs, which is significantly larger than that required by the Napa County Road and Street Standards. See Appendix C for calculations.



AASHTO HS20-44 Truck Loading

Because the insides of the spandrel walls were covered with soil and could not be measured, the depth of wall at the base of the structure is unknown; therefore, the analytical capacity of the end walls could not be calculated. The walls show no significant signs of movement or distress. ZFA recommends that the spandrel walls be reviewed regularly. See recommendation 2 above for more information regarding recommended review intervals.

Closing

This conditions and load rating report is based on that which was plainly visible at the time of site review. The items discussed are subject to revision should more information become available. ZFA understands you may have questions regarding this report and are available for comment and explanations, please contact ZFA for clarification of any questions you may have. We look forward to assisting with any future design efforts for mitigation, repair, or strengthening, as needed, of the subject bridge structure.

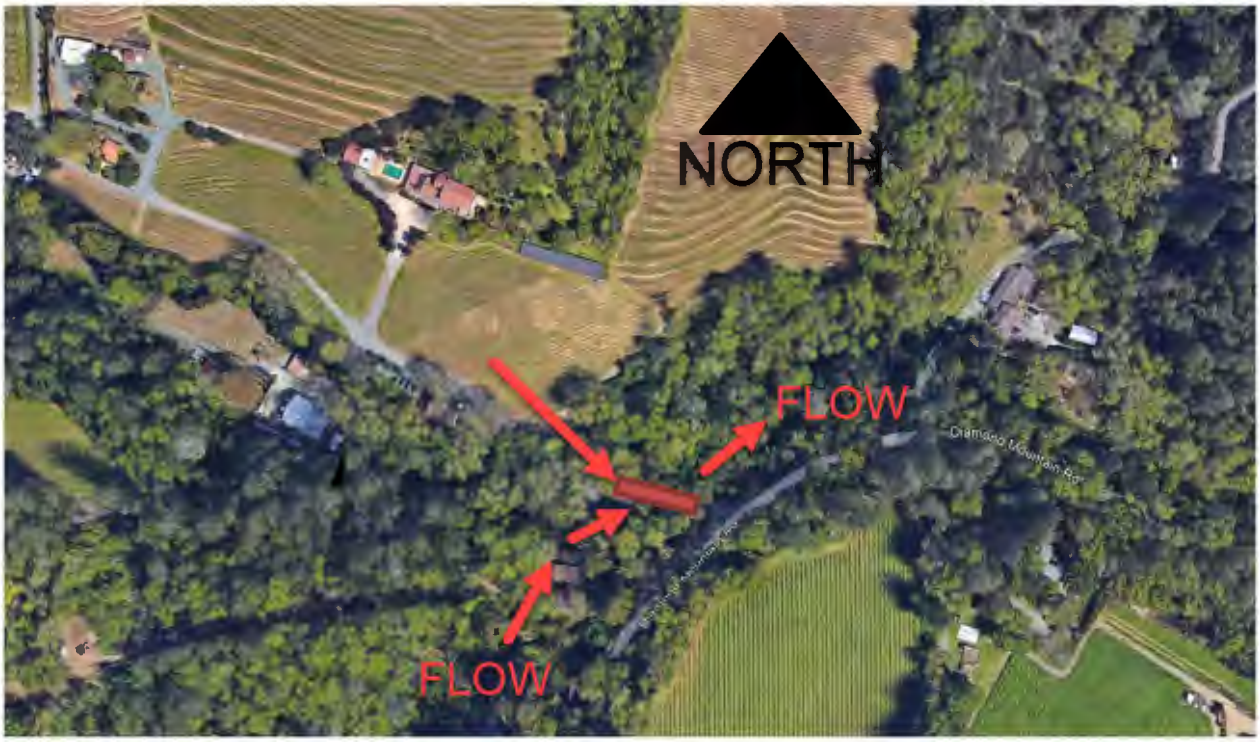
Sincerely,

Austin Spinelli
Designer

Chris Meade, SE
Associate

Kevin Zucco, SE
Executive Principal

Appendix A: Arial View of Structure Location



Appendix B: Site Photos

Photo 1: Labeled Elevation View

Photo 2: Downstream Elevation

Photo 3: Upstream Elevation

Photo 4: Roadway Surface

Photo 5: Partially Undermined Foundation at West Wall

Photo 6: Partially Undermined Foundation at Upstream East Wall

Photo 7: Missing Mortar and Loose Masonry

Photo 8: Loose Stone at Parapet

Appendix C – Calculations

ZFA STRUCTURAL ENGINEERS

DIAMOND CREEK
project name

MASONRY BRIDGE CAPACITY	AJS	21704	12/15/21	
section	engineer	job #	date	page

MODIFIED MEXE METHOD PROVSIONAL AXLE LOAD IS UNCONSERVATIVE FOR SHORT SPAN BRIDGES. USE PIPPARD'S EQUATION TO DETERMINE A MORE CONSERVATIVE PROVISIONAL AXLE LOAD (PAL)

PIPPARD'S EQUATION

$$W = \frac{\left(\frac{256\, f_c\, h d}{L}\right) - 128\rho L h\left(\frac{1}{21} + \frac{h+d}{4a} - \frac{a}{28d}\right)}{\left(\frac{25}{a} + \frac{4L}{J}\right)}$$

METRIC

L = SPAN (m)

d = ARCH BARREL THICKNESS (m)

h = FILL DEPTH (m)

a = ARCH CENTRAL RISE (m)

f_c = COMPRESSIVE STRESS (kPa)

ρ = DENSITY (kg)

$L = 9'-2'' = 2.79\ m$

$d = 15'' = 0.381\ m$

$h = 16'' = 0.457\ m$

$a = 33'' = 0.838\ m$

$f_c = 400\ psi = 2758\ kPa$ — SEE AASHTO THE MANUAL FOR BRIDGE EVALUATION TABLE 6B.5.2.3. STONE ASHLAR, LIMESTONE or MARBLE

$\rho = 120\ PCF = 18.85\ kN/m^3$

$W = 309.77\ kN \Rightarrow 31.59$ METRIC TON $= 69.6\ K$

$W_A = 2W = 63.18$ METRIC TON = PAL

2 WHEELS PER AXLE

AFTER MODIFIED MEXE METHOD ADJUSTMENT FACTORS

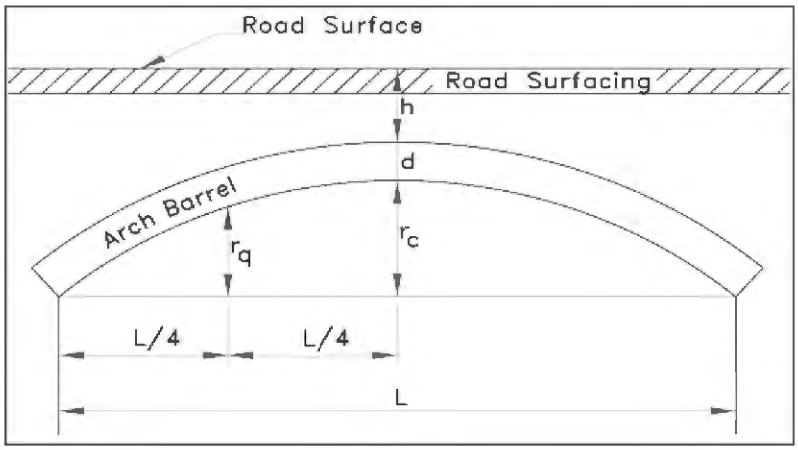
$W = 27.89$ METRIC TON $= 61.48\ K$

SEE MODIFIED MEXE METHOD BELOW FOR ADJUSTMENT AND MODIFIED AXLE LOAD

Appendic D: Calculations

MEXE Method per CS 454

The most widely used empirical assessment for stone arches is the Military Engineering Experimental Establishment (MEXE) method originally developed by the British Military and adopted for civilian use under the Design Manual for Roads and Bridges CS 454. The following calculations are per appendix E of CS 454. Calculations and tables use meters and metric tonnes.



Arch Dimensions

(feet)	L =	9.17	r_c =	2.75	r_q =	2.42	d =	1.25	h+d =	2.75
(meters)		2.79		0.84		0.74		0.38		0.84

As h > d MEXE may be unconservative. Limit h+d to 2d? (1 = Yes, 0 = No) 1
Calculations use h+d = 0.76

Section

E4	Provisional Axle Load (PAL) = $\min(740(d+h)^2/L^{1.3}, 70)$ =	63.18	Metric tonnes
E5.1	Span to Rise Ratio (L/r_c) =	3.33	
	Span to Rise Factor (F_{sr}) =	1.00	(Per Fig E.3)
E5.2	Profile Factor (F_p) = $2.3([r_c-r_q]/r_c)^{0.6}$ =	0.65	
E5.3	Barrel Material Factor (F_b) =	1	(Per Table E.1)
	Fill Material Factor (F_f) =	0.9	(Per Table E.2)
	Material Factor (F_m) = $([dF_b]+[hF_f]/[d+h])$ =	0.95	
E5.4	Joint Width Factor (F_w) =	0.8	(Per Table 7.5.1b)
	Joint Depth Factor (F_d) =	1.0	(Per Table 7.5.1c)
	Mortar Factor (F_{mo}) =	1.0	(Per Table 7.5.1d)
	Joint Factor (F_j) = $F_wF_dF_{mo}$ =	0.80	
E5.5	Arch Barrel Condition Factor (F_{CM}) =	0.9	(Per Table 7.5.1a)
E7	Modified Axle Load (MAL) = $F_{sr}F_pF_mF_jF_{CM}$ (PAL) =	27.89	Metric tonnes
		61.48	kips

E8 Axle Lift-Off Condition (1 = Lift-Off, 0 = No Lift-Off) = 0

a) Single Axle (A_{f1}) = 1.00 Allowable Axle Load = 27.89 Metric tonnes
b) Double Axle (A_{f2}) = 1.00 Allowable Axle Load = 27.89 Metric tonnes
c) Triple Axle (A_{f3}) = 1.00 Allowable Axle Load = 27.89 Metric tonnes

E10 Correlate Allowable Axle Load to Max Gross Vehicle Weight

Table E.3 Load capacity and weight restrictions for masonry arches

Allowable axle load per axle (tonnes)			Max gross vehicle weight (gvw) (tonnes)	Weight restriction (tonnes)
Single axle	Double axle bogie	Triple axle bogie		
11.5	10	8[1]	40/44	N/A
11.5	9.5		32	33
11.5	9.5	-	26	26
11.5 9 7	- - -	- - -	18 12.5 10	18 13 10
5.5 2	- -	- -	7.5 3	7.5 3

Note 1: An assessment for the 24 tonne 3 axle bogie (8 tonnes axle) is only necessary for arches where "no axle lift-off" conditions prevail.

Since the allowable axle load for the single, double, and triple axle bogies are all greater the max gross vehicle weight is 40/44 tonnes

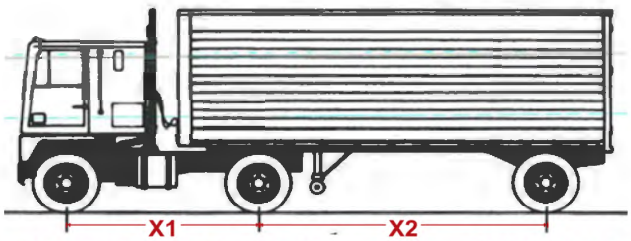
Below is a list of British standard vehicles in the 40/44 tonne weight class

Table B.1 Vehicle load models

	Vehicle details			Axle weights and spacings[2]												
Assessment live loading level	Ref[1]	Gross vehicle weight (tonnes)	No. of axles	O_1 (m)	W_1 (kN)	A_1 (m)	W_2 (kN)	A_2 (m)	W_3 (kN)	A_3 (m)	W_4 (kN)	A_4 (m)	W_5 (kN)	A_5 (m)	W_6 (kN)	O_2 (m)
Normal traffic	A	32	4	1.00	64	1.20	64	3.90	113	1.30	74					1.00
	B	38	4	1.00	64	3.00	113	5.10	98	1.80	98					1.00
	C	40	5	1.00	59	3.00	113	4.20	74	1.35	74	1.35	74			1.00
	D	40	5	1.00	59	2.80	113	1.30	64	5.28	78	1.02	78			1.00
	D	40	5	1.00	59	2.80	64	1.30	113	5.28	78	1.02	78			1.00
	E	40	5	1.00	49	2.80	103	1.30	44	4.80	98	1.80	98			1.00
	E	40	5	1.00	49	2.80	44	1.30	103	4.80	98	1.80	98			1.00
	F	41	6	1.00	49	2.80	103	1.30	49	4.18	67	1.35	67	1.35	67	1.00
	F	41	6	1.00	49	2.80	49	1.30	103	4.18	67	1.35	67	1.35	67	1.00
	G	44	6	1.00	59	2.80	103	1.30	49	4.70	74	1.35	74	1.35	74	1.00
	G	44	6	1.00	59	2.80	49	1.30	103	4.70	74	1.35	74	1.35	74	1.00
	H	44	5	1.00	69	2.80	113	1.30	74	7.60	88	1.35	88			1.00
	H	44	5	1.00	69	2.80	74	1.30	113	7.60	88	1.35	88			1.00

CS 454 Revision 0

Design truck E is most similar to the AASHTO HS20-44 standard design truck for highway loading
See below for a comparison of truck E to HS20-44 in standard units
Note closely spaced axle loads for truck E have been grouped into a single bogie



	W1(kips)	X1 (ft)	W2(kips)	X2 (ft)	W3(kips)
Truck E	11.0	11.3	33.0	20.8	44.1
HS20-44	8.0	14.0	32.0	14.0	32.0

Truck E and HS20-44 have a similar wheelbase but truck E has higher loads for each bogie

Based on the Above Evaluation the barrel arch section of this bridge is acceptable to support the HS20-44 typical highway truck

Appendix D – Unreinforced Masonry Repair Specifications

MATERIALS

1. IN GENERAL, MATERIALS SHALL COMPLY WITH THE FOLLOWING REQUIREMENTS AND SHALL MATCH THE COLOR & TEXTURE OF ADJACENT EXISTING MATERIALS AS BEST POSSIBLE. IT IS UNDERSTOOD THAT IN ORDER TO MATCH EXISTING MATERIALS, SOME DEVIATION MAY BE REQUIRED.

2. MASONRY SAND: ASTM C144.

3. LIME: ASTM C207 TYPE S OR SA.

4. LIME PUTTY: ASTM C5.

5. PORTLAND CEMENT: ASTM C150, NO GREATER THAN 0.6% ALKALI.

6. MASONRY CEMENT IS NOT PERMITTED UNLESS IT IS SHOWN TO BE STRENGTH-COMPATIBLE WITH EXISTING MORTAR.

7. FLY ASH: ASTM C618 TYPE F.

8. WATER SHALL BE POTABLE.

REPOINTING EXISTING HISTORIC MASONRY

1. ALL REPOINTING WORK SHALL CONFORM TO UNIFORM BUILDING CODE (UBC) STANDARD 21-8 "POINTING UNREINFORCED MASONRY WALLS" AS REFERENCED IN THE 2016 CALIFORNIA EXISTING BUILDING CODE (CEBC) AND FOLLOW THE RECOMMENDATIONS OF "PRESERVATION BRIEFS 2: REPOINTING MORTAR JOINTS IN HISTORIC MASONRY BUILDINGS" BY THE US DEPARTMENT OF THE INTERIOR, NATIONAL PARKS SERVICE (HTTPS://WWW.NPS.GOV/TPS/HOW-TO-PRESERVE/BRIEFS/2-REPOINT-MORTAR-JOINTS.HTM).

2. MORTAR JOINTS WHICH ARE UNFILLED, CRACKED, SOFTENED, DETACHED FROM EACH SIDE, OR ERODED TO A LEVEL OF 1/8 INCH OR MORE BELOW THE GENERAL MORTAR JOINT PROFILE, ARE TO BE CONSIDERED DEFECTIVE AND SHALL BE REPOINTED.

3. RAKE OUT MORTAR FROM JOINTS TO A MINIMUM DEPTH OF 3/4 INCH, 2 TIMES THE JOINT WIDTH, OR THAT REQUIRED TO EXPOSE SOUND, UNWEATHERED MORTAR (WHICHEVER IS GREATEST). WHERE DEPTH OF REMOVAL EXCEEDS DEPTH OF MASONRY UNIT, REMOVE UNIT AND CONTINUE REMOVAL OF MORTAR. PROVIDE SHORING AS REQUIRED.

4. DO NOT DAMAGE MASONRY UNITS DURING MORTAR REMOVAL. MASONRY CAN BE EASILY DAMAGED BY UNSKILLED USE OF POWER TOOLS. USE SMALL HAND TOOLS APPROPRIATE FOR THE WORK AS NEEDED.

5. RINSE RAKED OUT MASONRY JOINTS WITH CLEAN WATER TO REMOVE DUST AND MORTAR PARTICLES. TIME THE APPLICATION OF RINSING SUCH THAT AT TIME OF REPOINTING, JOINT SURFACES ARE DAMP BUT FREE OF STANDING WATER.

6. THE FOLLOWING MORTAR MIX MAY BE ADJUSTED AS REQUIRED SUCH THAT STRENGTH PROPERTIES, COLOR, AND TEXTURE MATCH THAT OF THE EXISTING MORTAR. MIX PARTS ARE MEASURED BY VOLUME. WHERE EXISTING MORTAR IS LIME-BASED, PROVIDE THE FOLLOWING MIX, BASED ON ASTM C270 TYPE K:

   1 PART PORTLAND CEMENT
   3 PARTS HYDRATED LIME
   8 TO 12 PARTS MASONRY SAND
   JUST ENOUGH WATER TO PROVIDE A WORKABLE CONSISTENCY

   WHERE EXISTING MASONRY IS SET IN STANDARD MORTAR, PROVIDE THE FOLLOWING MIX, BASED ON ASTM C270 TYPE O:

   1 PART PORTLAND CEMENT
   2 PARTS HYDRATED LIME
   6 TO 9 PARTS MASONRY SAND
   JUST ENOUGH WATER TO PROVIDE A WORKABLE CONSISTENCY

7. APPLY FIRST LAYER OF POINTING MORTAR TO THE MOST DEEPLY RAKED OUT JOINT IN AREA OF WORK. APPLY IN LAYERS NOT GREATER THAN 1/4 INCH UNTIL A UNIFORM DEPTH IS FORMED. COMPACT EACH LAYER THOROUGHLY AND ALLOW TO BECOME THUMB PRINT HARD BEFORE APPLYING NEXT LAYER.

8. IMMEDIATELY REMOVE SPILLED MORTAR FROM EXPOSED SURFACES.

9. WHEN FINAL LAYER OF MORTAR IS THUMB PRINT HARD, TOOL JOINTS TO MATCH ORIGINAL APPEARANCE OF JOINTS. REMOVE EXCESS MORTAR FROM EDGE OF JOINT BY BRUSHING.

10. CURE MORTAR BY MAINTAINING IN DAMP CONDITION FOR NOT LESS THAN 72 HOURS.

11. WHERE REPOINTING PRECEDES CLEANING OF EXISTING MASONRY, ALLOW MORTAR TO HARDEN NOT LESS THAN 14 DAYS BEFORE BEGINNING CLEANING.

GROUT INJECTION OF EXISTING MASONRY

1. CRACKS UP TO 3/4" INCH MAY BE INJECTED. NOTIFY ENGINEER FOR REVIEW IF CRACK EXCEEDS THIS WIDTH. CRACKS AT ARCHES ARE GENERALLY NOT INJECTABLE. FOR ADDITIONAL INFORMATION REGARDING GROUT INJECTION OF CRACKED MASONRY, SEE LOS ANGELES DEPARTMENT OF BUILDING & SAFETY DOCUMENT P/BC 2015-056 “CRACK REPAIR OF UNREINFORCED MASONRY WALL WITH GROUT INJECTION”.

2. REMOVE ALL LOOSE AND/OR CRACKED MORTAR AND MASONRY MATERIAL. REMOVE WALL FINISHES TO EXPOSE BOTH SIDES OF WALL WHERE GROUTING IS TO BE PERFORMED.

3. PROVIDE 3/4" INCH DIAMETER VERIFICATION HOLES SPACED AT 8 TO 12 INCHES FROM EACH SIDE OF VERTICAL & DIAGONAL CRACKS. SPACE HOLES MAXIMUM 48 INCHES ON CENTER ALONG CRACK. WHERE VOID IS ENCOUNTERED AT THE HOLES, GROUT SHALL BE SEEN FLOWING FROM THE HOLE DURING INJECTION. PLUG HOLES TO MATCH APPEARANCE OF SURROUNDING WALL AFTER PLACEMENT OF GROUT.

4. INJECTION PORTS AND VERIFICATION HOLES SHALL BE PROVIDED AT MORTAR JOINTS ONLY.

5. THOROUGHLY CLEAN THE WALL CAVITY BY FLUSHING WITH WATER. USE WATER AT NO GREATER THAN 10PSI THROUGH AN INJECTION WAND THAT CAN BE INSERTED THE FULL DEPTH OF THE CAVITY. FLUSH FROM TOP TO BOTTOM. FLUSH UNTIL THE WATER FLOWS CLEAR. PROTECT EXISTING PORTIONS OF THE BUILDING FROM CLEANING WATER.

6. REPOINT AS REQUIRED PER 'REPOINTING EXISTING HISTORIC MASONRY' SECTION SUCH THAT AREA TO BE INJECTED IS SEALED ON BOTH SIDES, INSTALLING INJECTION PORTS AS REQUIRED. INJECTION PORTS SHALL BE SPACED AT 4 INCHES ON CENTER FOR FINE CRACKS AND NO GREATER THAN 32 INCHES ON CENTER FOR LARGE CRACKS OR WHERE COLLAR JOINT VOID IS TO BE FILLED.

7. INJECTION MATERIAL SHALL BE VOIDSPAN "PHLc70 INJECTION GROUT". MIX GROUT POWDER WITH WATER AS DIRECTED BY MANUFACTURER. PRODUCT USE SHALL CONFORM TO MANUFACTURER'S RECOMMENDATIONS. ALTERNATIVELY, THE FOLLOWING GROUT MIX MAY BE PROVIDED (PARTS BY VOLUME):

   6 PARTS #60 SILICA SAND
   2 PARTS #90 SILICA SAND
   2 PARTS PORTLAND CEMENT
   1 PART TYPE S LIME
   1 PART TYPE F FLY ASH
   5± PARTS WATER TO PROVIDE A FLOWABLE CONSISTENCY

8. FOR THE ALTERNATIVE GROUT MIX: POUR A 2 INCH DIAMETER BY 4 INCH TALL CYLINDER OF GROUT FROM 12 INCHES ABOVE A HARD LEVEL SURFACE. PROPER CONSISTENCY HAS BEEN REACHED IF THE RESULTING PUDDLE IS BETWEEN 6 AND 8 INCHES IN DIAMETER. ADJUST WATER AND REPEAT AS REQUIRED.

9. IMMEDIATELY PRIOR TO GROUT INJECTION, DAMPEN CAVITY. ALLOW CAVITY TO DRAIN. STANDING WATER IS NOT PERMITTED.

10. GROUT SHALL BE PRESSURE INJECTED FROM BOTTOM OF WALL UPWARD, SUCH THAT AIR VOIDS ARE AVOIDED. DO NOT EXCEED 10 PSI INJECTION PRESSURE. MAINTAIN REASONABLE LIFT HEIGHTS (24 INCHES MAX) SUCH THAT FLUID PRESSURE OF GROUT DOES NOT DAMAGE EXISTING WALL. ALLOW GROUT TO SET BETWEEN LIFTS.

11. IMMEDIATELY REMOVE SPILLED GROUT FROM EXPOSED SURFACES.

12. REMOVE INJECTION PORTS (IF EXPOSED). POINT OVER PORTS TO MATCH ADJACENT MORTAR.

13. PROVIDE 2-1/2 INCH DIAMETER CORES CENTERED ON THE CRACK AFTER GROUT HAS HARDENED. SPACE CORES NOT GREATER THAN 8 FEET ON CENTER ALONG CRACK. MINIMUM (1) CORE PER CRACK. SPECIAL INSPECTOR TO REVIEW CORE TO VERIFY SUCCESSFUL SOLID GROUTING OF CRACK & VOIDS.

RECONSTRUCTION OF EXISTING MASONRY

1. REUSE EXISTING MASONRY UNITS FOR OUTSIDE COURSES/WYTHES AND AS MUCH AS POSSIBLE AT INTERIOR OF WALL. CRACKED OR BROKEN UNITS, IF SALVAGEABLE (AS DETERMINED BY THE ENGINEER), SHALL BE REPAIRED WITH SIKA "SIKADUR 32, HI-MOD" ADHESIVE. FOLLOW MANUFACTURER'S INSTALLATION INSTRUCTIONS. NO ADHESIVE SHALL BE VISIBLE. REPLACEMENT UNITS SHALL MATCH EXISTING UNITS IN STRENGTH & APPEARANCE AND SHALL BE APPROVED BY THE ENGINEER. ENGINEER SHALL REVIEW ANY NEW UNITS PRIOR TO INSTALLATION.

2. REMOVE STONES THAT ARE DETERMINED VISUALLY OR BY TOUCH TO BE DEBONDED FROM ADJACENT STONES. PROVIDE SHORING AS REQUIRED AND/OR DECONSTRUCT UP TO THE TOP OF THE WALL IF REMOVAL OF STONE(S) WILL RESULT IN INSTABILITY OF WALL.

3. REMOVE ALL LOOSE MORTAR AND OTHER MATERIAL FROM CAVITY. FLUSH CAVITY WITH WATER TO REMOVE DUST.

4. MORTAR SHALL MATCH THE COLOR, TEXTURE, COMPOSITION, AND STRENGTH OF THE EXISTING MORTAR. SEE SECTION “REPOINTING EXISTING HISTORIC MASONRY” FOR MORTAR SPECIFICATION & REQUIREMENTS.

5. THE WALL SHALL BE MORTARED SOLID, INCLUDING FILLING OF COLLAR JOINTS AND/OR WALL CORE SPACES.