Cedar Valley Engineering Co.
PO Box 827
Waverly, Iowa 50677
(319) 352-3213

September 21, 2015

Mr. Phil Jones, Administrator
City of Waverly
PO Box 616
Waverly, IA 50677

Re: 3rd Street SE (Green) Bridge

Dear Administrator Jones:

As requested by Council, Cedar Valley proposes to provide the following professional services for the fees shown. The fees are all inclusive; there will be no other reimbursable expenses claimed unless the scope of work changes and both parties agree to the change.

1. Confirm the prior inspections of the captioned bridge and report to Council what work, if any, would be required to return the bridge to the service level existing at the time of its closure.

$ 10,050.00

2. Prepare plans, specifications, and solicitation documents to cover the work, if any, required to return the bridge to the service level it provided at the time of closure and administer the solicitation and award of any contract for completion of the work.

$ 11,250.00

3. Administer the contract and provide inspection as required, if any, for the duration of the bridge repair contract.

$ 21,600.00

It must be understood that at the time of closure, the bridge offered limited service value. Our proposal is intended only to assist in returning the bridge to the same limited service value through minimal repair. The work undertaken will not "restore" or "upgrade" the structure beyond the service value it had at the time of closure.

All work will be personally supervised by the undersigned as principle. If the terms presented above are satisfactory, please sign and date the acceptance below.

Sincerely,

L. William Kehe, P.E.
President

Accepted by:____________________

For: City of Waverly

Date:____________________

PSJ

Cedar Valley Engineering Co.
PO Box 827
Waverly, Iowa 50677
(319) 352-3213

October 5, 2015

Mr. Phil Jones, Administrator
City of Waverly
PO Box 616
Waverly, IA 50677

Re: 3rd Street SE (Green) Bridge

Dear Administrator Jones:

I have completed the work I proposed as Item 1 in my letter of September 21, 2015.

In summary, it is my opinion that the bridge does require some immediate repair work, but that the work required does not prevent the bridge from being re-opened to the vehicular service level it provided at the time of its closure. Thus, the bridge could be opened while a repair contract is established. The repairs will not increase the capacity of the bridge, but simply extend its life in its current configuration another five years.

It is also my opinion that repairing the structure has only one purpose: to provide time for the construction of a vehicular crossing to serve the southeast at another location. Ultimately, the cost of maintaining the current structure for any purpose beyond another five years will be unsupportable.

My report, including a list of required repairs with an opinion of estimated costs is enclosed.

Sincerely,

L. William Kehe, P.E.
President

Enclosures:
Structural Study
Opinion of Estimated Cost

1) superstructure: $76,501.30
2) bearing rockers: $13,616.00
const. est. $90,117.30

does not include painting.

5+ years - consider painting.

P.E.: open bridge tonight.
do program next summer.

Cedar Valley Engineering Co
October 2, 2015
3rd Street Bridge, Waverly, Iowa, Structural Study

Object: Confirm prior engineering reports and report to Council what work, if any, is required to return the bridge to the service level existing at the time of closure, specifically a five ton one way vehicular load and an open sidewalk.

Reports Considered: The prior reports to be considered include the biennial condition report prepared by WHKS and forwarded to the City February 18, 2015, and an evaluation and feasibility study prepared by VJ Engineering dated July22, 2015. The WHKS report recommended that the bridge be closed to all traffic based upon the general degradation of the structure including numerous problems with superstructure, sidewalk, abutments and piers. The WHKS transmittal letter offers the opinion that future repairs are no longer feasible. It concludes correctly that long term repairs on a structure of this type are seldom undertaken because the repair process could be continual and repetitive. The report does not preclude short term repairs, but recommends against short term repairs because of the volume of work that may be required. The report offers no quantitative information from which an executive decision could be made other than to close the bridge permanently.

Because of the disruption of traffic patterns caused by the bridge closure, the Council requested VJ Engineering to develop quantitative data so that action could be taken to minimize the impact to the public. The VJ report agreed with the qualitative observations of the WHKS report but failed to quantify conditions and provide correcting repairs that would allow the bridge to be reopened to the service level it provided at the time of its closure. The VJ report did offer a detailed numerical analysis of the superstructure as designed. However, the only part of the analysis that recognized the poor condition of the bridge was the sidewalk analysis. It is not clear why the sidewalk analysis was the only one to reflect the condition of the bridge. The report also offered cost estimates for converting the crossing through restoration or replacement into a facility that meets current AASHTO design standards.

Superstructure:

Bearings: The first paragraph of the WHKS report under Superstructure deals with the bearings, or the feet upon which the trusses rest. The VJ section "Records Review, Superstructure" essentially repeats the WHKS observations. First, one must separate comments regarding the "bearing" from the L0 truss connection. Both occur at the same location and, as designed, use the same framing plate. But the impact each has on the bridge integrity is quite different. Should the L0 connection fail, the truss would drop in an instantaneous and catastrophic way. Should the bearing fail, the action would be a slow settling of the corner of the bridge. Even if one accepts that the settlement may distort a truss component, a catastrophic failure is unlikely. It should be understood that the "overshoe" installed at the southwest corner of the south truss were installed to reinforce the L0 connection, not the bearing assembly.

Incl 1

As the pictures found in the two reports clearly show, the bearing assemblies are in very bad shape. But, as the pictures also show, some of the deteriorated shape existed before the 1983 repairs as testified by the paint over the pitting. It should be noted that the rockers were out of line in 1983 and were probably moved out of line as the shoring was removed following the lower chord replacement of 1967. It should also be noted that the ends of the bridge have not settled measurably since the 1983 repairs. But, even if the bearings may not pose an immediate safety threat, replacement to avoid torquing the trusses should be considered.

Main Trusses: The second paragraph of the WHKS report under Superstructure deals with the trusses. The VJ section "Records Review, Superstructure" essentially repeats the WHKS observations. The principle observation is the existence of "pack rust" reported to be present in many of the connections along the lower chord and pitting of the gussets or framing plates. The entire lower chord assembly, including gussets or framing plates, was replaced in 1967. Because some of the pitting reported is covered by paint last applied in 1983, some of the pitting of the plates existed prior to the spot repairs completed in 1983. The "overshoe," found at the southwest corner of the south span, was installed in 1983 where again much of the pitting was sand blasted and painted.

Other concerns reported include out of plane bending of diagonal components. All diagonal components are tension components. Unlike compression components, minor out of plane bending of a tension component is not critical. The one diagonal member that has suffered extreme bending is not even critical to the strength of the truss. Consequently, straightening of the bent members has no value other than appearance. Also reported was the deterioration of existing repairs. However, the report failed to identify in which span or truss the deterioration occurred. Re-examination of both sides revealed no critical deterioration. Finally, distortion and section loss of the lower chord angles in the west truss of the south span was reported. The distortion is apparent, but no section loss could be measured. The lower chord is also a tension assembly and the distortion observed is not critical to performance. The current condition of the trusses does not demand any immediate repair.

However, the VJ numerical analysis identifies the trusses as under designed and controlling the strength of the bridge. The report concludes that the trusses require reinforcing to even carry the original design load. The numerical analysis was completed by a computer program. But, the computations are clearly shown in the report. Starting at page 1 of the analysis it is noted that the assumed design dead load was used; no consideration was given to the reduction in dead load provided by the 1983 reconstruction of the deck and the 2004 reconstruction of the sidewalk. Attachment A tabulates the current truss dead loads that in total amount to a 22% reduction of dead load on the east truss and a 33% reduction of dead load on the west truss.

The design live load of 90 psf used as the pedestrian load in the VJ report is also acceptable for a design vehicle load as the bridge is currently configured. Over the deck width of eighteen feet the 90 psf design load becomes 1620 pounds per lineal foot. Assuming the design vehicle is twenty-two feet long and one way traffic prevails, a 90 psf design load allows the design vehicle to weigh 17.82 tons. Converting vehicular loads to uniform loads will not work for the floor structure, but the trusses will see little difference.

In the truss data table on sheets 1 and 2 one can find five data entry errors in the member identification columns and nine data entry errors in the gross area columns. Attachment B offers corrected input and results. The first tabulation identified as "VJ Corrected" uses the assumed dead load but corrects for the data entry errors. It concludes that the trusses achieve an inventory rating of 84.53, or 94% of AASHTO design standards. The second tabulation identified as "DL Reduced" adjusts the dead load as well as correcting the data entries resulting in an inventory rating of 106.73 or 18% higher than current AASHTO design standards. Contrary to the VJ report, no reinforcing of either truss is required.

Floor Beams: The third paragraph of the WHKS report under Superstructure deals with the floor beams and stringers and, again, the VJ study essentially repeats the WHKS observations. While specific examples were identified, neither report quantified the possible problems. Our examination found floor beams with up to 40% lower flange section loss at mid span. In the interest of understanding by non-engineers, in this case the condition of the flanges is most important at the middle. At the ends of the beam, the web, or the material between the flanges, is most important. This understanding applies to the stringers as well. Because of the deterioration, the capacity of the floor beams drop from 280 foot kips to 206 foot kips. Adjusting for the reduced capacity, but also for the lighter dead load, the inventory rating under a 90 psf uniform load becomes 138psf or 1.53 times greater than current design requirements.

An inventory rating for vehicular traffic constrained as it was at the time of closing can best be determined by assuming two small straight trucks traveling bumper to bumper such that one axle of each is three feet either side of the beam. This is not a typical AASHTO model, but does reflect a possible occurrence on the bridge. Allowing for a 40% reduction in the lower flange of a floor beam yields an inventory rating of 7 tons. But, it is highly unlikely that one will find two small trucks moving so closely together. ?

Stringers/Deck: Not acknowledged by either report is the fact that the grid deck installed in 1983 not only reduced the dead load but became part of a composite stringer/deck unit. The decking is tightly installed, attached to the stringers by welding and by the addition of shear keys such that much of the compression originally carried by the stringers is transferred to the deck. Replacement of the deck with wood will not only add dead load but reduce the stringer capacity.

Both reports indicate that the stringers are in extremely poor condition. However, the condition of the stringers is not acknowledged in the VJ computations. Examination of the stringers indicates that there could be as much as 50% loss of flange, not even considering possible discontinuities. Applying this observation to the VJ computations yields an inventory rating of 60.3psf or 67% of the current design requirements. It is the condition of the stringers that controls the bridge capacity, not the trusses. To return to a wood deck would require the replacement of all of the existing stringers.

The stringer deficiency was addressed in 1983 by applying cover plates to the lower flanges and installing the steel deck such that it would contribute to the strength of the upper flange of the stringer. The resulting vehicular inventory rating was computed in 1983 as 9 tons. The stringers show some further loss of section from the lower flange such that additional plating may be required. However, the decking was also selected to allow greater lateral distribution of loads than typically considered by AASHTO. The section of the decking is such that a single stringer failure will be barely noticed by a passing vehicle.

The connections from stringer to floor beam appear extremely poor in the photographs found in the report. However, chipping away the flowering rust reveals section loss in the floor beam web and connection clips to be not as severe as the pictures imply. Further examination and selective repair would still be in order. The stringer web cracking as shown in the pictures offers a challenge in that is not typical of a failing section. Normal failure analysis does not identify what may be the cause. But, it does need further consideration because it appears to be moving into the critical portion of the stringer web. Needs time.

Looks worse than it is?

Sidewalk: The sidewalk was rebuilt in 2004 using stay-in-place galvanized corrugated forming. What has been identified as "white rust" appears to be the residue of cementitious water from the concrete placement. The lower sidewalk to truss connections are severely rusted. The poor condition of the lower sidewalk brackets is the first location where VJ computations acknowledge the existing condition. Their computations rate the current sidewalk at 87 psf without acknowledgement of the reduced dead load. Introducing the reduced dead load into the VJ computations yields an inventory rating of 136.45 psf, 1.51 times the current design standard.

VJ poor condition

Concrete Abutments and Piers: As should be understood by the brief comments found in the WHKS report, determining the true remaining capacity of the concrete substructure is very problematic. Unless significant drilling into the core of the structure or removing of poor concrete is undertaken, one cannot estimate whether or not the concrete deterioration truly influences its ability to adequately support the bearings. Because there appears to have been no settling of the bearings at the bearing/ concrete interface and based upon experience with other similar structures, the damage to the concrete substructure can be identified as mostly sloughing due to ice and weather erosion. Of course, there is no way to confirm whether or not settling has occurred any more than one can confirm that the sloughing has not penetrated deep enough to jeopardize bearing.

Not addressed in either report is the bearing of the concrete substructure below the surface. The original plans ghost in piling. But, the size, type and bearing are not recorded. After ninety-eight years without showing movement or scouring, it is reasonable to say the concrete substructure is firmly founded. It would appear that in the short run the concrete substructure will perform as it has for the last ninety-eight years without immediate repair.

Conclusions: The bridge can be reopened to the level of service existing at the time of closure for the short term while a repair contract is entered into that addresses the bearings, and the stringer/ deck system. Following completion of the repairs the bridge should serve for another five years allowing for completion of a new crossing serving the southeast part of the city.

Assumed wt c. 1917
Actual wt. below post 1983, 2004 work

3rd Street SE Bridge
Waverly, Iowa
Estimated Weights
22-Sep-15

Location	Description	Quantity	Weight
East Truss			
Vertical Struts	2 ea C8x 11.25	126	2835.00
Diagonal Ties	2 ea A 5x3x.3125	54.16	888.22
	2 ea A 3.5*2.5*.25	54.16	530.77
	2 ea A 2.5x2.5x.25	54.16	444.11
	2 ea 3.5x2.5x.25	27.08	265.38
Lower Chord	2 ea A 5x3.5x.3125	68.4	1190.16
	2 ea A 5x3.5x.4375	34.2	820.80
	2 ea A 5x3.5x.5	17.1	465.12
Upper Chord	2 ea C 8x11.25	85.5	1923.75
	1 ea PL 14x.3125	85.5	1276.52
Diagonal Chord	2 ea C 8x16.25	54.16	1760.20
	1 ea PL 14x.3125	54.16	808.61
Vertical Lacing	PL 2x.25x10	432	613.44
Tie Plates	PL 1'-7"x8x.25	28	301.28
Subtotal			14123.36
Connections		0.25	3530.84
Total East Truss Alone			17654.20
Weight per Foot			147.12
West Truss			
Vertical Struts	2 ea C8x 11.25	126	2835.00
Diagonal Ties	2 ea A 6x3.5x.4375	54.16	1657.30
	2 ea A 5*3*.3125	54.16	888.22
	2 ea A 2.5x2.5x.25	54.16	444.11
	2 ea 3.5x2.5x.25	27.08	265.38
Lower Chord	2 ea A 5x3.5x.4375	68.4	1641.60
	2 ea A 5x3.5x.375	34.2	711.36
	2 ea A 5x3.5x.3125	34.2	595.08
	4 ea A 5x3.5x.4375	17.1	820.80
Upper Chord	2 ea C 9x13.25	34.2	906.30
	1 ea PL 15x.375	34.2	765.40
	2 ea C 9x20	51.3	2052.00
	1 ea PL 15x.3125	51.3	980.86
Diagonal Chord	2 ea C 9x20	54.16	2166.40
	1 ea PL 15x.375	54.16	1212.10
Vertical Lacing	PL 2x.25x10	432	613.44
Tie Plates	PL 1'-7"x8x.25	28	301.28

Gross area calculations on 9 areas?

ATTACHMENT A 1/2

Subtotal			18856.63
Connections		0.25	4714.16
Total East Truss Alone			23570.79
Weight per Foot			196.42
Deck			
Floor Beam	I18x 55x 19'-4"	8	8505.20
Edge Stringers	C9x13.25x17'-1.25"	7	1586.03
	C12x20.5x17-1.25"	7	2453.85
Interior Stringers	I9x21x17'-1.25"	49	17595.90
Diagonal Bracing	A3.5x2.5x.25x25'-9"	14	1764.00
Decking	Grid @ 11psf	2319.6	25515.60
Subtotal			57420.58
Connections		0.25	14355.14
Total			71775.72
Weight per Foot per Truss			299.07
Sidewalk			
SB1 Brackets	PL 16.75x.25x5'-8.75"	2	200.00
	2 ea A2.5x2.5x.25	24.32	199.42
SB2 Brackets	PL 21.25x.25x5'-8.75"	6	780.00
	2 ea A2.5x2.5x.3125	39.48	394.80
	2 ea A3x2.5x.3125	45.48	509.38
Stringers	C5x6.5	120	780.00
	C8x11.25	120	1350.00
	I8x18	120	2160.00
Subtotal Steel			6373.60
Connections		0.25	1593.40
Concrete & Forms	50psf	699.6	34980.00
Total			42947.00
Weight per Foot of Truss			357.89
Total Wt per Foot of East Truss			446.18
Total Wt per Foot of West Truss			853.38

ATTACHMENT A 2/2

3rd Street SE Bridge
City of Waverly
Truss Member Load Tabulation

Member	A_g	$C_{allowable}$	$T_{allowable}$	P_{actual}	S.R.	
Unit Load Distribution						
L0-U1				3.87		
U1-U2				4.06		
U2-U3				4.87		
U3-U4				4.83		
L0-L1				-2.44		
L1-L2				-2.44		
L2-L3				-4.06		
L3-L4				-4.92		
L1-U1				-1		
L2-U2				1		
L3-U3				0.05		
U1-L2				-2.58		
U2-L3				-1.28		
U3-L4				0.07		

Bold Italics updated by Rich Kelm. PE

East Truss (VJ Corrected)

L0-U1	14	336.48	399	142.49	0.42	
U1-U2	11.1	249.24	316.92	149.49	0.60	
U2-U3	11.1	249.24	316.92	179.31	0.72	
U3-U4	11.1	249.24	316.92	179.31	0.72	Compression added to offset U3-L4
L0-L1	5.12	28.4	145.92	-89.84	0.62	
L1-L2	5.12	28.4	145.92	-89.84	0.62	
L2-L3	***7.04***	***37.53***	***200.50***	-149.49	***0.75***	
L3-L4	8.01	42.7	228.29	-181.15	0.79	
L1-U1	6.61	134.98	188.39	-36.82	0.20	
L2-U2	6.61	134.98	188.39	36.82	0.27	
L3-U3	6.61	134.98	188.39	1.84	0.01	
U1-L2	4.81	7.41	137.09	-95.00	0.69	
U2-L3	2.9	3.3	82.6	-47.13	0.57	
U3-L4	***2.37***	***2.63***	***67.545***	0.00	***0.00***	Section incapable of carrying compression

L3-L4 Controls	
$C_{truss\ 1}$	174.64
γDL	51.13
γLL	130.20
Inventory Rating	85.38
Operating Rating	110.68

ATTACHMENT B 1/3

BOLD ITALICS updated by B-tch Kelm, PE

West Truss (VJ Corrected)

L0-U1	16.45	402.13	468.26	252.52	0.63	
U1-U2	12.6	293.09	358.25	264.92	0.90	
U2-U3	***16.45***	***382.6***	***468.26***	317.77	0.83	
U3-U4	***16.45***	***382.6***	***468.26***	317.77	0.83	Compression added to offset U3-L4
L0-L1	***7.06***	***35.41***	***201.07***	-159.21	0.79	
L1-L2	***7.06***	***35.41***	***201.07***	-159.21	0.79	
L2-L3	11.2	62.23	319.77	-264.92	0.83	
L3-L4	***14.12***	***78.62***	***402.14***	-321.03	0.80	
L1-U1	6.61	134.98	188.39	-65.25	0.35	
L2-U2	6.61	134.98	188.39	65.25	0.48	
L3-U3	6.61	134.98	188.39	3.26	0.02	
U1-L2	9.04	18.14	257.64	-168.35	0.65	
U2-L3	4.81	7.41	137.09	-83.52	0.61	
U3-L4	2.37	***2.63***	***67.545***	0.00	0.00	Section incapable of carrying compression

L3-L4 Controls

$C_{truss\ 1}$	307.64
γDL	104.49
γLL	216.30

Inventory Rating	84.53
Operating Rating	109.57

East Truss (DL Reduced by Decking & Sidewalk)

L0-U1	14	336.48	399	129.10	0.38	
U1-U2	11.1	249.24	316.92	135.44	0.54	
U2-U3	11.1	249.24	316.92	162.46	0.65	
U3-U4	11.1	249.24	316.92	162.46	0.65	Compression added to offset U3-L4
L0-L1	5.12	28.4	145.92	-81.40	0.56	
L1-L2	5.12	28.4	145.92	-81.40	0.56	
L2-L3	8.01	42.7	228.29	-135.44	0.59	
L3-L4	8.01	42.7	228.29	-164.13	0.72	
L1-U1	6.61	134.98	188.39	-33.36	0.18	
L2-U2	6.61	134.98	188.39	33.36	0.25	
L3-U3	6.61	134.98	188.39	1.67	0.01	
U1-L2	4.81	7.41	137.09	-86.07	0.63	
U2-L3	2.9	3.3	82.6	-42.70	0.52	
U3-L4	***2.37***	***2.63***	***67.545***	0.00	***0.00***	Section incapable of carrying compression

L3-L4 Controls

$C_{truss\ 1}$	174.64
γDL	33.93

ATTACHMENT B 2/3

γLL	130.20
Inventory Rating	97.27
Operating Rating	126.09

→ Above 90 lb. ft. sq.

West Truss (DL Reduced by Decking & Sidewalk)

L0-U1	16.45	402.13	468.26	210.35	0.52	
U1-U2	12.6	293.09	358.25	220.67	0.75	
U2-U3	***16.45***	***382.6***	***468.26***	264.70	***0.69***	
U3-U4	***16.45***	***382.6***	***468.26***	264.70	***0.69***	Compression added to offset U3-L4
L0-L1	***7.06***	***35.41***	***201.07***	-132.62	***0.66***	
L1-L2	***7.06***	***35.41***	***201.07***	-132.62	***0.66***	
L2-L3	11.2	62.23	319.77	-220.67	0.69	
L3-L4	***14.12***	***78.62***	***402.14***	-267.42	***0.66***	
L1-U1	6.61	134.98	188.39	-54.35	0.29	
L2-U2	6.61	134.98	188.39	54.35	0.40	
L3-U3	6.61	134.98	188.39	2.72	0.02	
U1-L2	9.04	18.14	257.64	-140.23	0.54	
U2-L3	4.81	7.41	137.09	-69.57	0.51	
U3-L4	2.37	***2.63***	***67.545***	0.00	***0.00***	Section incapable of carrying compression

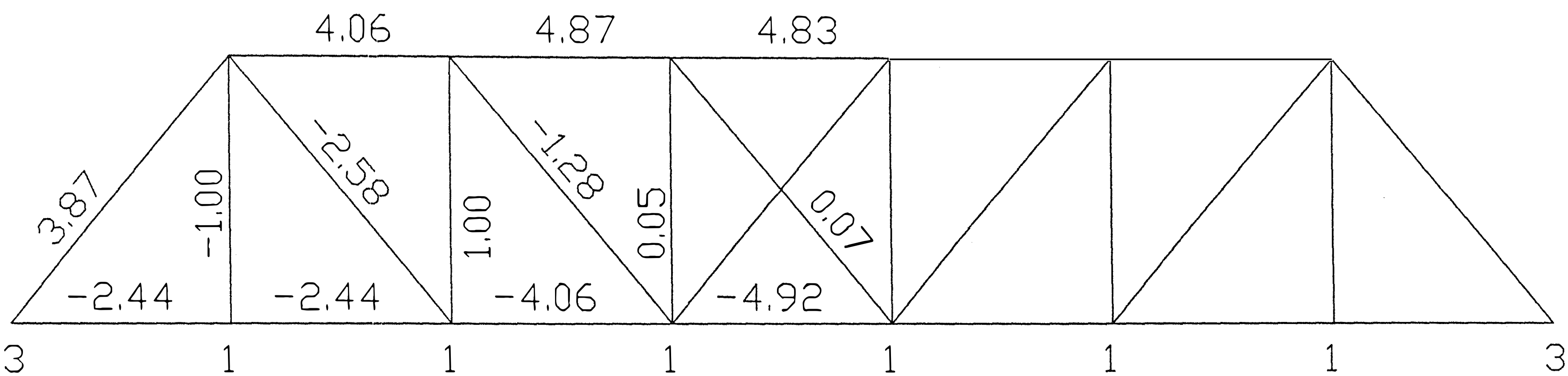
L3-L4 Controls

$C_{truss\ 1}$	307.64
γDL	51.12
γLL	216.30
Inventory Rating	106.73
Operating Rating	138.36

- Above 90 lb ft²

☆ Trusses do not control capacity of bridge.

ATTACHMENT B 3/3



UNIT LOAD DIAGRAM

ATTACHMENT C