Instrumentation of Bridge #MEG-124-6.78

by

Richard A. Miller

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University of Cincinnati
Department of Civil and Environmental Engineering
P. O. Box 210071
Cincinnati, OH 45221-0071

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16. Abstract

Bridge MEG 124-6.78 was a single span, skewed, composite, adjacent, prestressed box girder bridge. After being in service about 1 year, the three edge beams on the inside of the curve showed extensive cracking. Subsequent inspection revealed several construction errors, including loose bearing pads and a large bump in the approach slab. In an attempt to determine the cause of cracking, the bridge was subjected to both static and dynamic truck load testing using single axle dump trucks. Under 10 different static load combinations, the bridge showed expected the load distributions between the beams. Therefore, the cause of failure was not overload in the edge beams due to improper load distribution.

Due to construction errors, some of the bearing pads were loose and easily removed. Under load, the end of the beam with the loose pads twisted. However, the amount of twist was small and subsequent analysis estimated the torsional shear stresses as < 75 psi; about 15% of allowable. Thus, torsion may have contributed to the failure was not the cause of the failure.

Dynamic testing was done. The maximum dynamic magnification factor was 30%. This is consistent with the value used in the AASHTO *Standard Specifications*. The effect of coal trucks running over the bridge was also assessed, but the measurements showed that the effect of the coal trucks was in line with design assumptions and was not the cause of failure.

The bridge was assessed under 1994 *AASHTO Guide Specification* which accounts for skew and has a different distribution factor for shear. However, if the beams are analyzed using the higher distribution factor, the beams are still safe in shear.

The results of these tests show that beam failure did not occur due to normal traffic effects on the bridge. To find the cause of failure, additional work is needed.

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Adjacent, bridge, box beams, concrete, cracking, diagonal cracking, failure, load tests, prestressed, shear

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Richard A. Miller
University of Cincinnati
Department of Civil and Environmental Engineering
P. O. Box 210071
Cincinnati, OH 45221-0071

EXECUTIVE SUMMARY

Bridge MEG 124-6.78 was a single span, composite, adjacent, prestressed box girder bridge. The bridge had a 45° right forward skew. After being in service about 1 year, the three edge beams on the inside of the curve showed extensive cracking. Diagonal and vertical cracks were visible on the fascia beam. Subsequent inspection revealed several construction errors, including loose bearing pads and a large bump in the approach slab.

In an attempt to determine the cause of cracking, the bridge was subjected to both static and dynamic truck load testing using single axle dump trucks. Each truck weighed weighed between 27 kips and 32 kips. Under 10 different static load combinations, the bridge showed expected the load distributions between the beams. Uniform loads were equally shared between the girders. Loads which were more toward one edge were distributed more to that edge of the bridge, but the distribution was still reasonable. It did not appear that one or two beams were carrying the entire load. Therefore, the cause of failure was not overload in the edge beams due to improper load distribution.

Due to construction errors, the top of the abutments were not even and the bearing pads were not shimmed. Some of the bearing pads were loose and easily removed. Under load, the end of the beam with the loose pads twisted. However, the amount of twist was small and subsequent analysis estimated the torsional shear stresses as < 75 psi; about 15% of allowable. Thus, torsion may have contributed to the failure was not the cause of the failure.

Dynamic testing was done by running a full dump truck over the bridge at 15, 30 and 45 mph. The maximum dynamic magnification factor was 30%. This is consistent with the value used in the AASHTO *Standard Specifications*. The effect of coal trucks running over the bridge was also assessed, but the measurements showed that the effect of the coal trucks was in line with design assumptions and was not the cause of failure.

The bridge was originally designed using the *AASHTO Standard Specifications for Highway Bridges*. This specification does not account for skew or whether the beam is an edge beam, or have separate distribution factors for shear and moment. The 1994 *AASHTO Guide Specification* adjusts the distribution factors for these effects. When the bridge is analyzed under the *Guide Specification*, the wheel load shear distribution factor rises from 0.698 to 1.05. However, if the beams are analyzed using the higher distribution factor, a 30% impact and allowing for 20% overload, the beams are still safe in shear.

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ABSTRACT

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CHAPTER 1

DESCRIPTION OF THE BRIDGE AND TESTING PROGRAM

1.1 Description of the Bridge

Bridge MEG - 124 - 6.78 is located in Meigs County, Ohio on State Route 124, just east of the town of Salem Center. Built in 1994, this structure is a single span, composite, adjacent box girder bridge consisting of nine box girders (eight CB21-48 and one CB21-36) with a 5.5" thick composite deck (Figures 1.1 and 1.2). The bridge has a 45'-0" span and a 45° right forward skew. Because it is on a curve, the bridge has a standard superelevation.

The bridge runs northwest to southeast (Figure 1.2), thus having a NW bound lane and a SE bound lane. For convenience, the beams have been numbered. Beam #1 is on the NE edge of the bridge under the NW bound lane. Beam #9 is on the SW edge under the SE bound lane (Figure 1.1).

In May, 1995, Malloon's Run (the creek under the bridge) flooded, covering the bridge with water. After the flood, a neighboring property owner checked the bridge and noticed cracking near the support (note: this is not to imply that the flooding caused the cracking; only that the flooding was the impetus which caused the neighbor to inspect the bridge). He notified ODOT District 10. District 10 engineers immediately inspected the bridge and found diagonal cracking at the obtuse corner of beam #9 (Figure 1.3). Additional, less severe, cracking was found near the acute corner of the beam. In all, the inspection revealed the following problems:

1) The beam on the southwest edge (numbered 9 in Figure 1.1 and shown in Figure 1.3) had diagonal cracks and straight cracks in the exposed side of the beam. These cracks continued to the bottom of the box where they became transverse (perpendicular to the span). Whether the cracks continued up the other side of the beam was not clear as the other side of the beam (being up against the the adjacent beam) was not visible. The worst cracking occurred at the northwest corner where the cracks appeared to be diagonal shear cracks. Less severe cracking, which appeared to a vertical flexural crack, occurred at the other end of the beam as well. In both cases, the cracks formed about 4 ft. from the obtuse corner.

2) Each of the adjacent beams (numbered 7 and 8 in Figure 1.1) had transverse cracks along the bottom. The sides of the beam were not visible so it could not be ascertained if the diagonal cracks existed along the sides of the beam. Again, cracking occurred at both ends of the beams approximately 4 ft. from the obtuse corner.

3) The abutments were poorly finished. Due to the superelevation of the structure, the top of the abutments should have had a constant slope. However, the slope was not constant causing the abutment to have a "wavy" appearance.

4) The bearing pads were not tight. It was possible to pull bearing pads out from under some of the beams.

5) The approach slab in the SE corner of the bridge (in the NW bound lane) had a large bump.

Due to the observed problems with the bridge, the engineers elected to close the SE bound lane (which passed over the cracked beams) while the cause of cracking was investigated and until the bridge could be replaced.

ODOT Engineers decided the first course of action would be to test the bridge under static and dynamic traffic loads and then, during replacement, remove beam specimens for later destructive testing. To this end, investigators from the University of Cincinnati were called upon to conduct the testing.

1.2 Testing and Analysis Program

In order to determine the cause of cracking in the beams, a three part testing program was designed. The three phases were:

1) Testing the existing structure as a unit under both static and dynamic truck loads.

2) It was clear that due to the severe cracking, the superstructure beams would need to be replaced. Some of the beams were salvaged and taken to Cincinnati for destructive load testing.

3) After replacement, the new superstructure was subjected to static truck loading, similar to that done in part 1.

Phase 1 is described in this report. Phases two and three are described in the companion report *Evaluation of the Cause of Cracking in Bridge #MEG-124-6.78*.

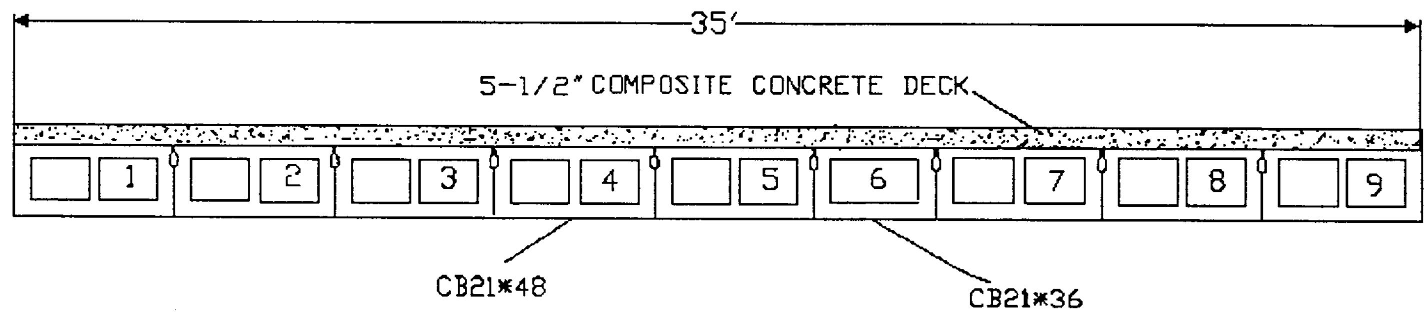

Figure 1.1 - Bridge Cross Section

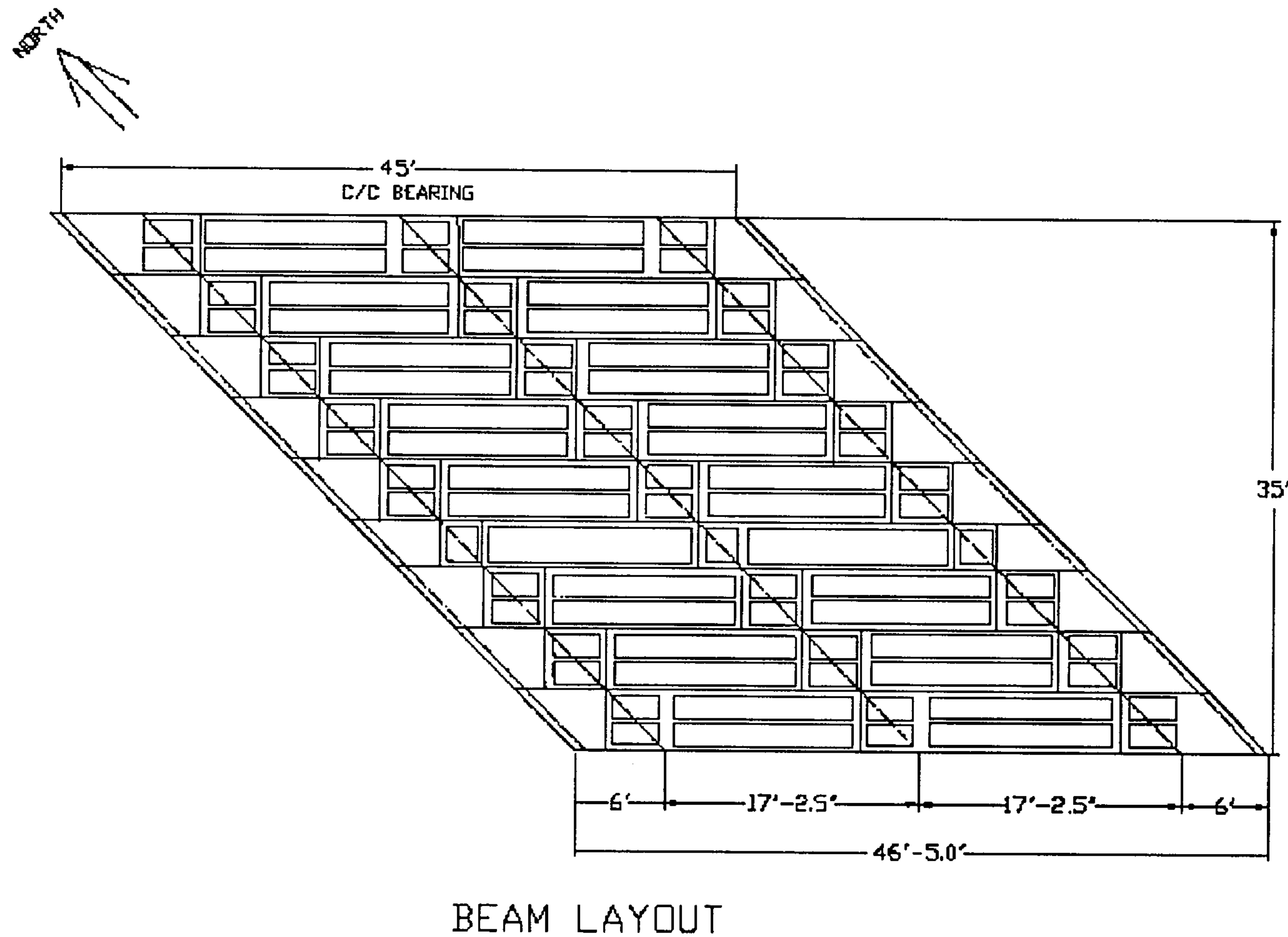

Figure 1.2 - Bridge Plan

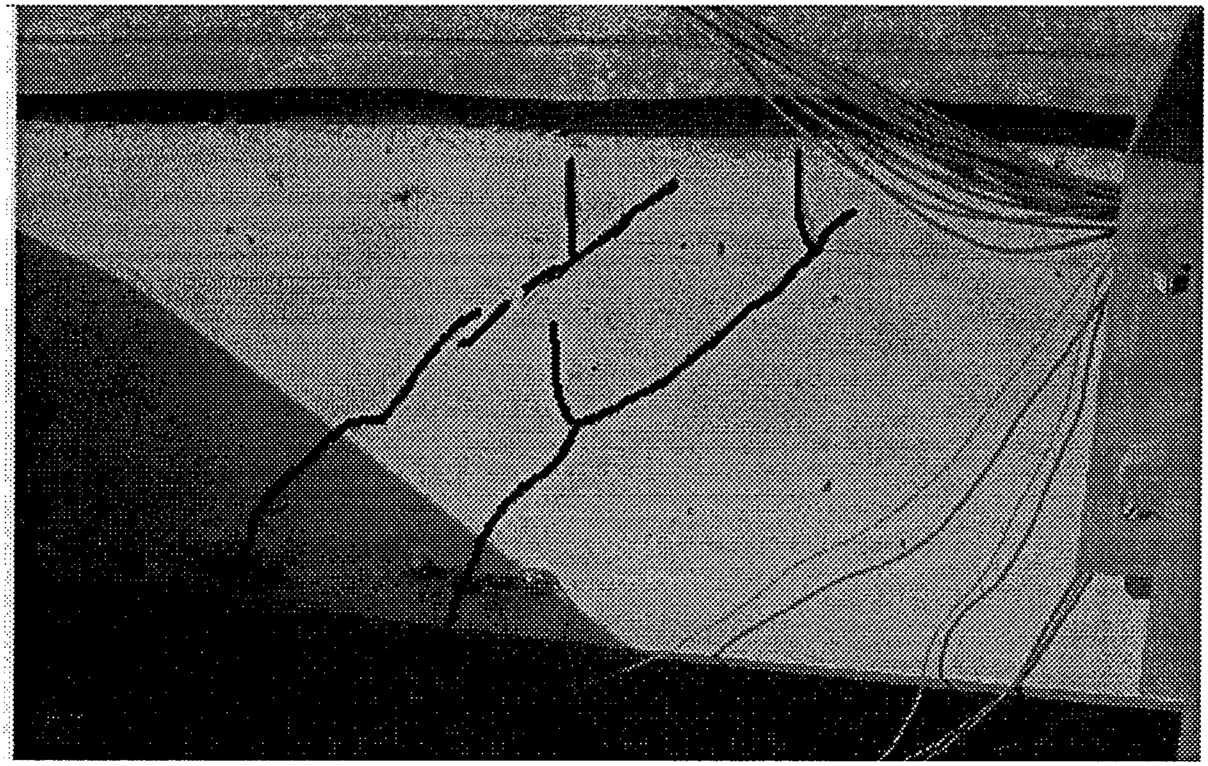

Figure 1.3a - Diagonal Cracks in Beam #9 (cracks highlighted to show detail)

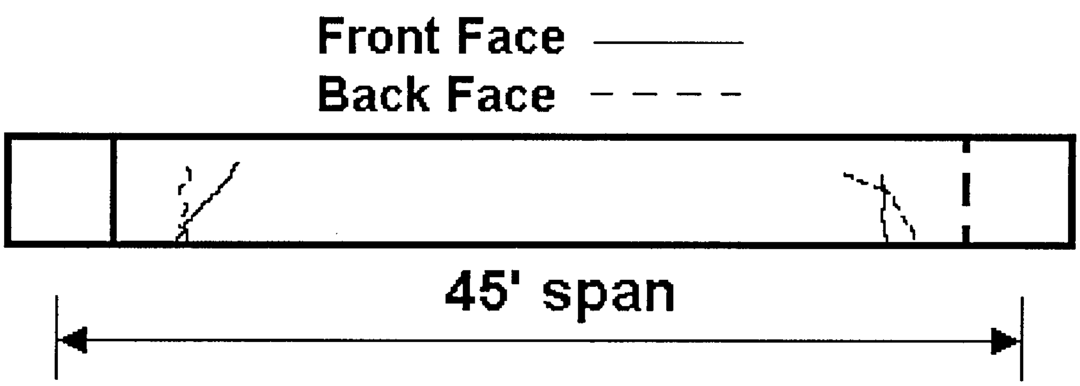

Figure 1.3b - Typical Crack Patterns - Beams # 7, 8, 9.

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CHAPTER 2

TEST RESULTS

2.1 Testing Program

Inspection of the MEG-124-6.78 bridge revealed that Beam #9 (Figure 1.1), an edge beam, had a large, diagonal crack at one end and what looked to be a flexural crack at the other. Two other beams, #8 and #7 had cracks in the bottom flanges at each end (cracking in the webs could not be seen because the structure is made of adjacent boxes). As an initial attempt to determine the cause of cracking, three possibilities were explored:

1) The beams had cracked due to an overload condition. This may have been caused by the heavy coal trucks which used the bridge. Full trucks used the southwest bound lane and would travel over the cracked beams. The beams may have cracked because the trucks were simply too heavy and/or because the load sharing mechanisms in the adjacent structure failed and all the load was concentrated on one or two beams.

2) The diagonal cracks were shear cracks caused by torsion. As noted in Chapter 1, there were loose bearing pads under some of the beams and the beams may have been able to twist under load.

3) Excessive dynamic impact forces may have caused an overload condition.

To test these theories, a series of static and dynamic tests were conducted.

2.2 Static Loading

Static truck loading consisted of placing up to four loaded dump trucks on the span in various configurations to provide maximum responses at various points. The dump trucks were standard ODOT two axle trucks loaded with sand. It was not possible to get four identical trucks and Figure 2.1 shows the dimensions and weights of each truck. Figure 2.2 shows the instrumentation layout for the tests (explained below). In all, ten separate test positions were used. Figures 2.3-12 show the truck positions (the driver's side front tire position is marked).

2.2.1 Instrumentation

A total of 36 separate instruments were attached to the bridge (Figure 2.2). The instruments used were:

DCLVDT: DCLVDT (or DCDT) stands for DC linear voltage differential transformer. This instrument consists of a metal core which is inserted into an electrical coil. As the core moves, the output voltage of the coil changes. Since the output voltage is directly related to core movement, the DCDT can directly measure the displacement of any object to which it is connected. DCDTs were used to measure vertical deflections of the beams, movement at the supports (to see if the beams twist where the bearing pads are loose) and horizontal movement of the bridges. The accuracy of a DCDT is about $\pm$ 0.001"

Clip Gages: Clip gages consist of a strain gaged piece of metal bent into a semicircular shape As the bottom of the semicircle is moved, the metal is strained. Since the amount of strain in the metal is directly proportional to the amount the bottom of the semicircle is moved, the device can measure the strain over a given length. The clip gages used in this test were set to have gage lengths of 5". The accuracy was $\pm$ 25 x 10^{-6} in/in when used to measure strain and 0.0001" when used to measure displacement or crack opening.

The following quantities were measured:

1) Deflection at the midspan of beams 1, 3, 7 and 9.

2) Support movement under both bearing pads at both ends of beams 2 and 8. Note that beam 8 was chosen because it was found to have loose bearing pads. Beam 2 was chosen because it was the symmetrical counterpart and it did not have loose bearing pads. Support movement of the northwest end of beam 7, adjacent to beam 8, was also measured to assess stress transfer between beams.

3) Deflections at the obtuse corners of the bridge.

4) Midspan strains at the top and bottom of beams 1 and 9.

5) Opening of the two cracks at the northwest end of beam 9 and the cracks in beams 7 and 8.

6) Differential vertical displacement of the two sides of the cracks on the bottom of beam 9 at the northwest end.

The static data was collected using two Strawberry Tree (brand) ACPC 16-16 data acquisition cards with Labtech (brand) *Notebook* data acquisition software installed in a 486DX-33 IBM compatible personal computer. Each card is capable of reading 16 analog data channels, thus, with two cards a total of 32 channels were available. Data was collected at the rate of 0.5 hertz (one reading every 2 seconds).

For static testing, the following routine was used:

1) After the bridge was cleared of all traffic, the data acquisition system was run with the bridge in an unloaded condition to acquire 10 data points for each instrument. The ten points were then averaged to obtain the "pretest zero" reading. This "zero" reading is necessary because the numerical values recorded by the instruments will slowly change over a long period of

time, even if no load is applied. This condition is called "drift". To account for drift, a "zero" reading is taken when the structure is not loaded. All subsequent instrument readings under load are then corrected by subtracting the "zero" reading.

2) The trucks were placed on the bridge in the required location. After allowing a few seconds for any vibrations to damp out, the data acquisition system was run to acquire ten data points for each instrument. Again the ten points were averaged to obtain the under load reading.

3) After the data was acquired, the trucks were removed. The bridge was again given a few seconds to allow vibrations to damp out and a "posttest zero" was then acquired to assure excessive drift had not occurred during the test.

2.2.2 Static Test Results

To determine if failure was caused by uneven load distribution or torsion, a series of static tests were conducted. Figures 2.3 - 2.12 show the results of the static tests. In each figure, the truck configuration, including truck number and position of the left (driver's side) front wheel, is shown in the upper right corner. The measurement from each instrument is shown next to the instrument symbol. Four clip gages were used to measure bending strain on the sides of beams #1 and #9 (two on each beam; one top and one bottom), but virtually all cases the measured strains were so small that the data was judged unreliable. These instruments are not shown in Figures 2.3 - 2.12.

From these figures, the following items of interest are noted:

1) The midspan deflections of the beams are consistent with the bridge acting as a single unit for the given loading patterns (e.g. in Case #1 the load is distributed along the midspan line and all the beams deflect approximately the same amount; in Case #5 the trucks are parked on the center beams and the center beams deflect more than the edge beams). To the extent which they can be verified by simple calculations, the midspan deflections appear correct. In Case #1, a simple calculation was done assuming that the truck loads were equally distributed to all beams. A theoretical deflection of 0.06" was found and this agrees with the deflection measured values. Based on the fact that the data shows the bridge behaving as a single unit, it does not appear that the cause of failure is overload due to uneven load distribution .

2) It is clear that some of the beams are twisting. This twist appears to be caused by two factors: loose bearing pads and the skew of the bridge. A loose bearing pad was found under beam #8. This can be seen in Figure 2.3 where there is a measured twist in the NW end (left side as viewed in Figure 2.3) of beam #8. This end shows a differential deflection of 0.005" (0.013" - 0.008") between the sides of the beam. The other end shows no appreciable twist as the 0.001" difference is within the accuracy of the instrument. Twist is found in beam #8 in Load Cases 1, 2, 3, 8, 9, 10. In all of these cases the trucks were positioned to load beam #8. In the remaining cases, the trucks do not directly load beam #8 and no twist is found.

The angle of twist of beam #8 is small and box beams are very stiff in torsion, so effect of this twist on the beam stresses is expected to be small. The actual bridge system is very complicated due to the slab and shear keys connecting the beams and accurate evaluation of the stresses caused by beam twist would require finite element verification. However, it is possible to obtain an estimate of the stress caused by twisting by treating beam #8 as a single unit, disconnected from the bridge. The analysis is done by applying classical thin wall torsional theory which assumes the shear stress is constant through the wall. Applying the torsion theory and assuming simple support, the effective torque would be about 9k-ft and the simple, St. Vennant shear stress less than 15 psi.

As with St. Vennant shear stress, the warping stresses are also hard to evaluate given the complexity of the beam interconnection and end conditions. A simple finite element analysis was done and it was found that the total St. Vennant and warping shear stresses were less than 75 psi. This is a small value (≈ 15% of allowable) and it is unlikely that this torsion alone was the cause of failure, although it may have contributed to the failure.

The obtuse corners of the bridge show movement. In any load case where the trucks are distributed across the bridge, there is a clear pattern of deflection where deflections at the end of the beams are small near the acute corners and increase toward the obtuse corner. This can be seen in Load Cases 1, 2, 3, 4, 5, 8, 9, 10. The movement at the obtuse corners seems to cause twist in the edge beams even though the bearing pads were not loose. Consider the obtuse corner of beam #1 in Load Case #1. The movement of one edge of the beam was measured. Since there is considerable evidence that the bridge acts as a single unit, it can be assumed that movement of the other edge of beam #1 (which was not measured) is the same as the movement of the adjoining edge of beam #2. With this assumption, the differential movement at the end of beam #1 is 0.004" (= 0.013" - 0.009"); almost the same magnitude of twist as found in beam #8, which has a loose bearing pad. Also note that the twist at the obtuse corner of beam #9 is 0.010"; although this twist may be affected by both the presence of the crack and the loose bearing pad under beam #8. With the exception of load case 9; significant twist occurs at the obtuse corners in any load case where a significant load flows to these corners. Although these beams show measurable twist, the magnitude of the twist is small and, as argued in the preceding paragraph, the shear stresses are probably small.

Based on data obtained, it did not appear that the failure was caused by torsion.

2.3 Dynamic Load Testing

After completion of the static tests, a series of dynamic tests were conducted. In these cases, a single dump truck was used to determine the dynamic response of the bridge under a moving load. The heaviest truck (594) was used. Only midspan deflections were used in the dynamic tests. A Hewlett Packard HP35670 dynamic analyzer was used to read the deflections.

Prior to conducting the dynamic test, static data was taken. The truck was moved across the bridge in the northeast bound direction (toward Salem Center) and was stopped with the front axle at the first quarter point, midspan, second quarter point and the far abutment. With the truck positioned at each point, midspan deflection data was taken for beams 1, 3, 7 and 9 . From this data, the maximum midspan static deflection of beams 1, 3, 7, and 9 was found. For beams 1 and 3, the maximum deflection occurred when the front wheels were near the far abutment (ant the rear wheels near midspan), while the maximum midspan deflection for beams 7 and 9 occurred when the front wheels were at one quarter point and the rear wheels near the other quarter point.

After the static test, the truck was run over the bridge in both directions at three different speeds: 15 mph, 30 mph and 45 mph. Deflection data was taken while the truck crossed the bridge. Since the static data was only taken in one lane, the moving truck always traveled in the same lane (i.e. when the truck was headed southwest toward Rutland, it was on the wrong side of the road). Table 2.1 shows the maximum deflection recorded for each speed and the dynamic magnification factor. The dynamic magnification factor is the maximum dynamic deflection divided by the maximum static deflection.

Table 2.1 Dynamic Data

Location	Max Static	Deflections from Salem Center		SW Bound			
		15 mph	DMF	30 mph	DMF	45 mph	DMF
	inch	inch		inch		inch	
Beam 1	0.031	0.036	1.161	0.033	1.065	0.037	1.161
Beam 3	0.035	0.038	1.086	0.035	1.000	0.041	1.171
Beam 7	0.017	0.014	0.824	0.014	0.824	0.018	1.059
Beam 9	0.010	0.007	0.700	0.007	0.700	0.007	0.700
		Deflections from Rutland			NE Bound		
		15 mph	DMF	30 mph	DMF	45 mph	DMF
	inch	inch		inch		inch	
Beam 1	0.031	0.035	1.129	0.040	1.290	0.037	1.194
Beam 3	0.035	0.033	0.943	0.040	1.143	0.041	1.171
Beam 7	0.017	0.009	0.529	0.010	0.588	0.018	1.059
Beam 9	0.010	0.003	0.300	0.003	0.300	0.007	0.700

DFM = dynamic magnification factor (impact factor)

From Table 2.1, the maximum dynamic magnification factor was 1.29. This is consistent with the 30% impact factor used in the *AASHTO Standard Specifications (16th Ed.)*. Note that the DFM is usually less than 1 for beams 7 and 9 and the highest DFM occurs for beam 1, not beam 3 which was under the loaded lane. This indicates that load does not spread as effectively under moving load conditions and that beam 1 carries more load and beams 7 and 9 carry less.

The results of the dynamic test using dump trucks clearly indicate that moving loads are not providing higher than expected impact load. Therefore, the beams did not fail due to overload in the dynamic or impact condition.

2.4 Effect of Coal Trucks

During the time the tests were being conducted, coal trucks passed over the bridge. Recall that ODOT had restricted traffic to the northwest bound lane (i.e. trucks traveling southeast to Rutland traveled in the wrong lane). This was the same lane used for dynamic testing described in the previous section. Unlike the dump trucks, it was not possible to stop the coal trucks for static measurements, so the trucks are compared to ODOT dump trucks.

Table 2.2 - Coal Truck Data

	ODOT Dump Truck Static Deflection	Full Coal Trucks from Salem Center		Empty Coal Trucks from Rutland	
Location	inch	Deflection - inch	Ratio	Deflection - inch	Ratio
Beam 1	0.031	0.084	2.71	0.053	1.71
Beam 3	0.035	0.084	2.40	0.043	1.23
Beam 7	0.013	0.024	1.85	0.021	1.62
Beam 9	0.008	0.009	1.12	0.018	2.25

The maximum ratio, 2.71, occurs for beam 1 with the loaded coal truck. This means the loaded coal truck "feels" like 2.71 static dump trucks. This DOES NOT mean that an impact factor of 2.71 is appropriate since the table above compares apples (dynamic coal truck loads) with oranges (static dump trucks). However, it is possible to do some analysis.

Information supplied by District 10 indicates that the coal trucks are longer than 45 feet and the wheel base (distance from front tractor wheels to rear trailer tandem axle) is about 35 feet. Since the bridge is only 45 feet long, the critical loading when one of the tandem axles is directly over midspan (the other wheels are off the bridge). Information from weigh-in-motion studies (again from District 10) show that these tandem axles are about 4 feet apart and each axle has a load of 25 - 30 kips. If a comparison study is done, it can be shown that a coal truck with 30 kip tandem axles will cause 2 times the deflection of the ODOT dump truck (the multiplier is 1.6 if the 25 kip axle is used). Therefore, if a 30% impact factor is used, ratio of the dynamic deflection of the coal truck to the static deflection of the ODOT dump truck should be 2 * (1+0.30) = 2.60; again assuming the 30 kips axles.

The actual maximum ratio is 2.71, which is reasonably close to the theoretical ratio of 2.60. The error is about 3% and this is well within acceptable engineering standards. Therefore, the bridge did not fail due to excessive dynamic loads.

2.5 Analysis of the Beams for Shear

Since the failure was a diagonal crack, which suggests shear, the bridge beams were analyzed for the effect of shear. The shear analysis was done according to *AASHTO Standard Specifications* assuming the design strength of 5500 psi. The *Standard Specifications* do not account for skew.

In 1994, AASHTO issued a *Guide Specification for Distribution of Loads in Highway Bridges*. This *Guide Specification* uses different distribution factors for shear and moment and modifies the live load distribution factors to account for interior vs. exterior beams and the effect of skew. This is similar to the *AASHTO Load and Resistance Factor Design (LRFD) Method* which has separate distribution factors for shear and moment and accounts for interior vs. exterior beams and skew.

Using the *AASHTO Standard Specifications*, which ignore the effect of skew, the wheel load distribution factor is 0.698. However, if the *Guide Specifications* are used, the wheel load distribution factor increases to 1.05. This increase comes from three factors:

1) There is a separate wheel load distribution factor for shear. This factor is given as:

$$DF = \left(\frac{b}{3.2}\right)^{0.4}\left(\frac{b}{L}\right)^{0.1} = \left(\frac{4}{3.2}\right)^{0.4}\left(\frac{4}{45}\right)^{0.1} = 0.858$$

where: b = beam width
L = span

2) There is a modifier for edge beams. This modifier would not normally apply to this bridge since the traffic lane is not over the outside bridge beam. The worst case for this bridge would be if the coal trucks used the shoulder as lane. This does happen as the trucks attempt to "cheat" the curve by driving as close to the edge of the bridge as possible in order to travel at higher speeds. However, even in the worst case for this bridge, this modifier is 1. The modifier is greater than 1 only if there is an overhang.

3) There is a skew modifier. This is given as:

$$1.0 + c_1 \tan\theta$$

$$c_1 = \frac{L}{90d}$$

For $\theta = 45°$, L = 540 inches (45 feet) and d = 26"; the modification factor becomes 1.22.

Therefore, the total distribution factor is:
DF = 0.858 * 1 * 1.22 = 1.05

Figure 2.12 shows the shear analysis. The shear capacities, V_{cw} + stirrups and V_{ci} + stirrups, are calculated according the 16th Edition of the *AASHTO Standard Specifications* and plotted. The applied shear forces are also plotted. In both cases, the controlling HS-20 truck is used to determine the applied forces (alternate military does not control). The lower curve plots the shear forces calculated according the *Standard Specifications*, using the calculated shear impact factor and a distribution factor of 0.698.

The upper curve uses the wheel load distribution factor of 1.05, as calculated under the 1994 *Guide Specifications* and an assumed 30% maximum impact factor. In addition, the upper curve allows for overloaded trucks by arbitrarily increasing the live load by 20%. Figure 2.12 shows that, even in the worst case, the beam is adequate for shear.

2.6 Conclusions from the First Field Test

From the first field test, the following conclusions can be drawn:

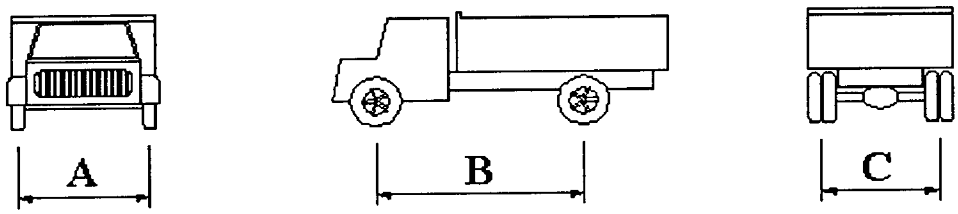
1) The static deflections indicate that the bridge is behaving as a unit, therefore, it does not appear that failure was caused by overload due to uneven distribution of load in the beams.

2) Although the beams twist, the amount of twist is small and significant torsional stresses are not generated. Failure was not caused by torsion.

3) Dynamic testing showed that moving loads do not distribute as evenly as static load. However, the dynamic magnification factor (or impact factor) was, at most, 30%. This is consistent with the *AASHTO Standard Specifications*. Therefore, overloading due to excessive dynamic loads did not cause failure.

4) The bridge had a large skew, which the standard specifications do not account for. Also, the bridge is on a coal route where trucks may be overloaded. However, even when accounting for the increased load effect of skew (using the *AASHTO 1994 Guide Specifications*) and allowing for overload by using an impact factor of 50%, the beams still would have had sufficient shear capacity. Therefore, the effect of skew did not cause failure.

While the field test did not produce the cause of failure, it did eliminate several possible causes. However, since the cause of failure was still unknown, the beams were removed from the bridge and subjected to load testing.



TRUCK NUMBER	FRONT AXLE LOAD POUNDS	REAR AXLE LOAD POUNDS	TOTAL LOAD POUNDS	A INCHES	B INCHES	C INCHES
594	7460	24400	32060	80	136	73
601	7460	22800	30420	80	138	72
768	8920	21600	30700	80	136	73
822	7300	19040	26560	79	137	73

Figure 2.1 - Truck Weights

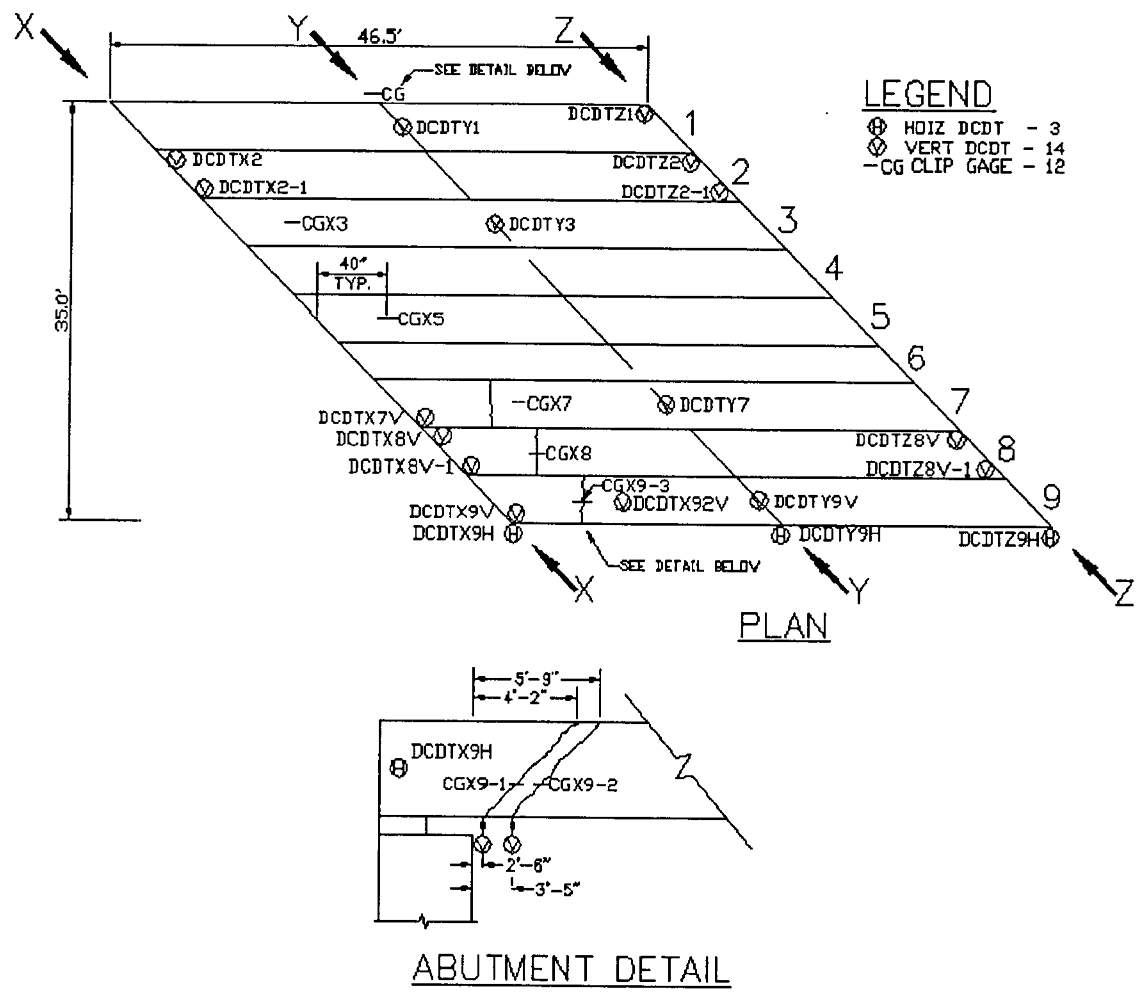


Figure 2.2 - Instrumentation Plan

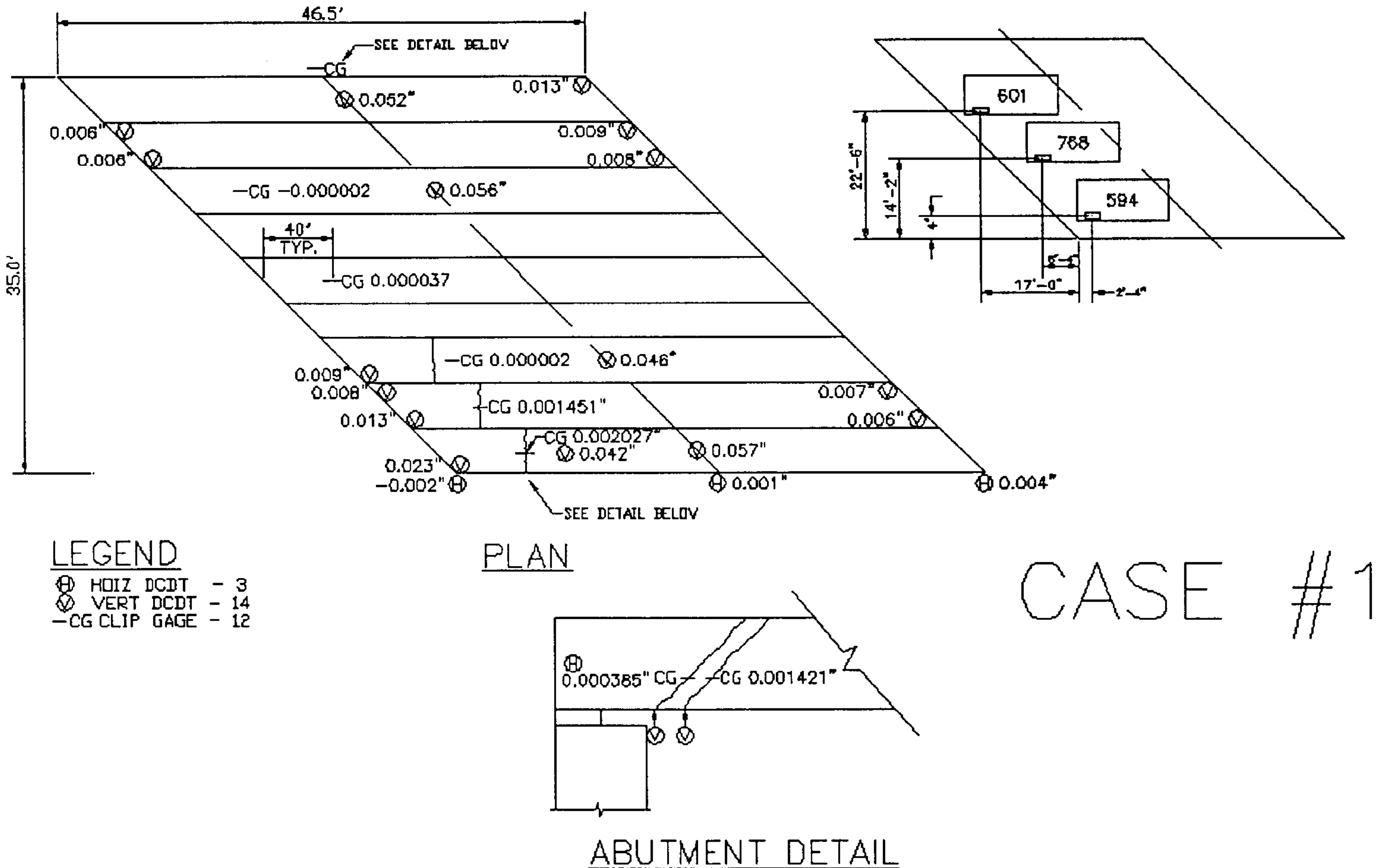


Figure 2.3 - Load Case #1

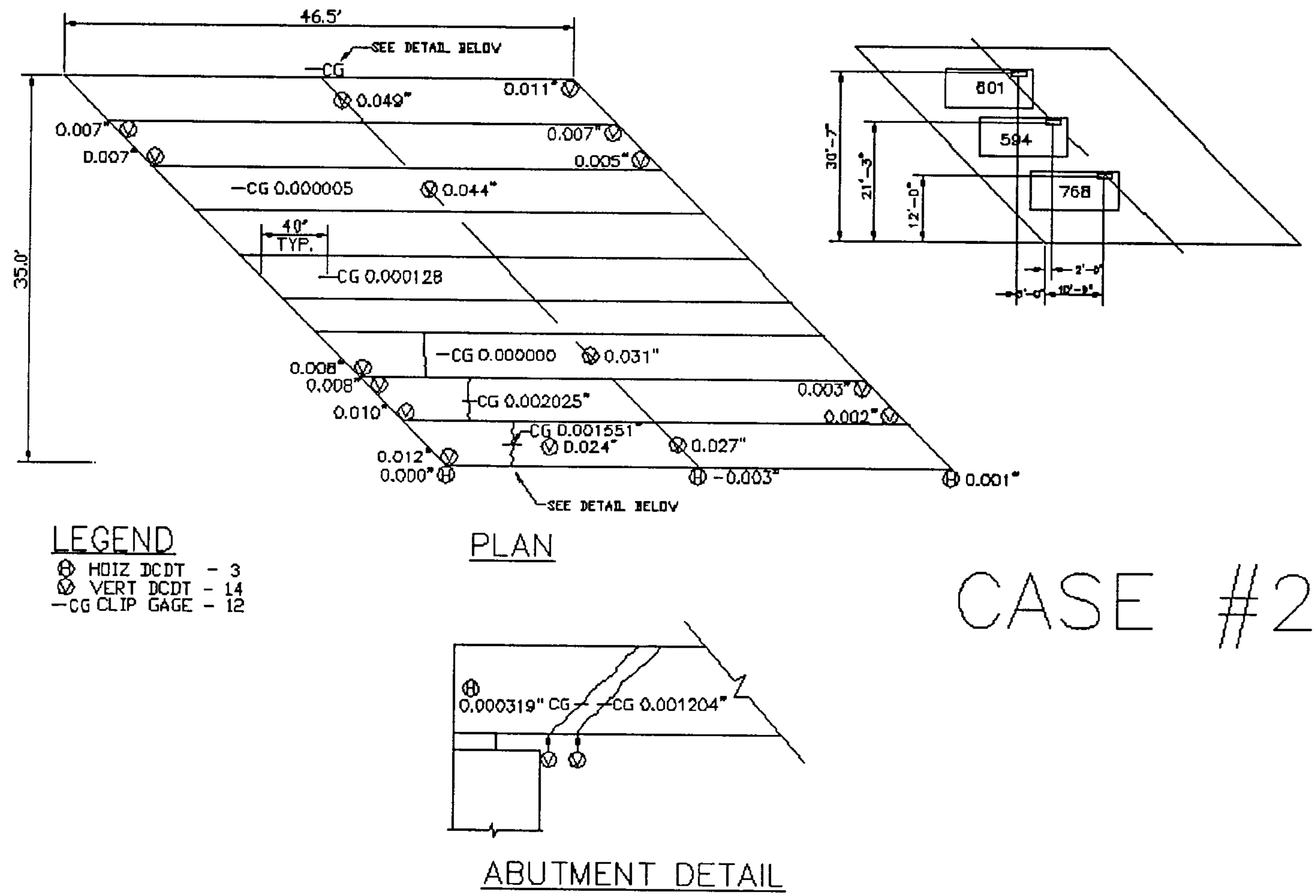


Figure 2.4 - Load Case #2

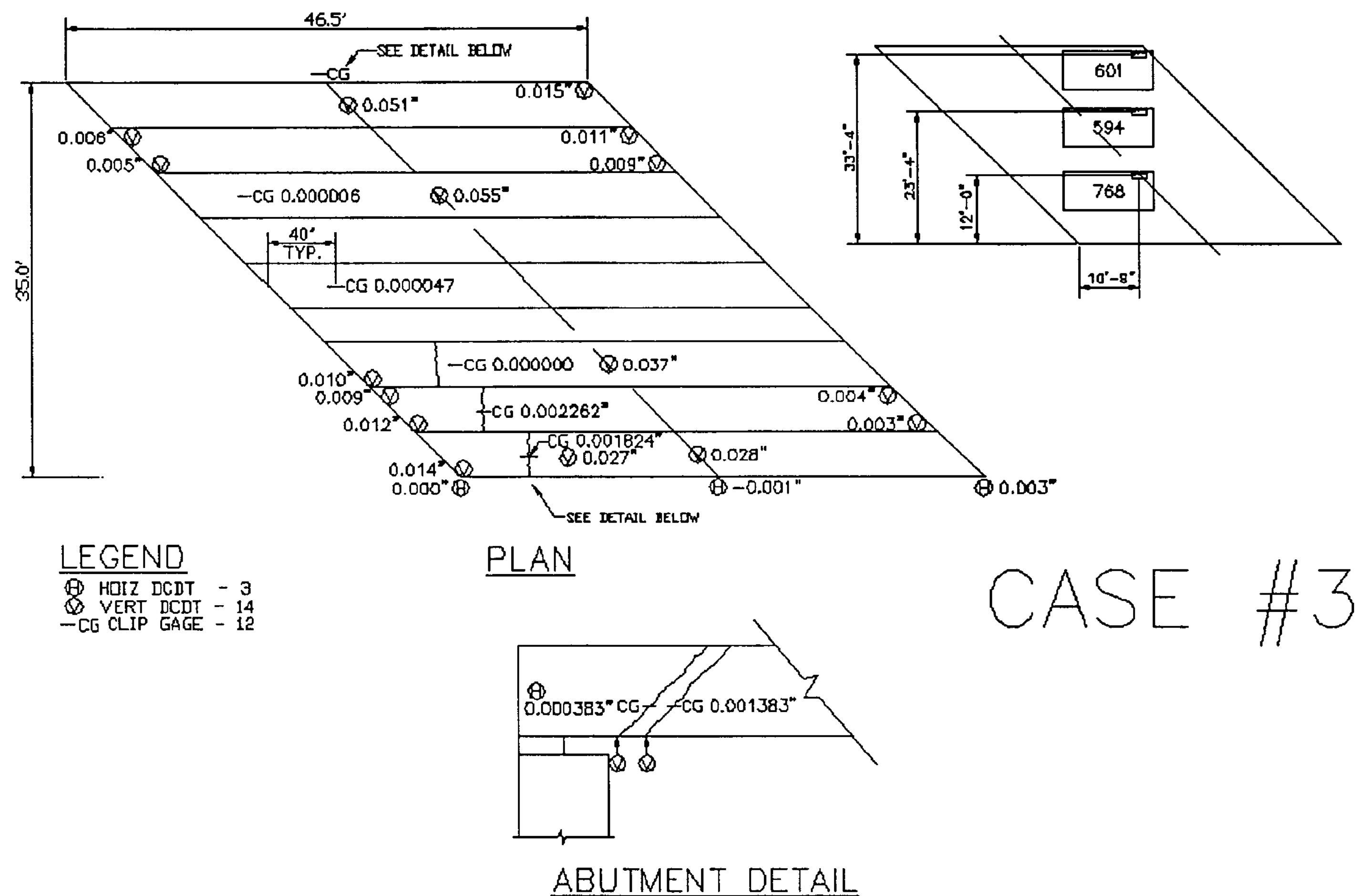


Figure 2.5 - Load Case #3

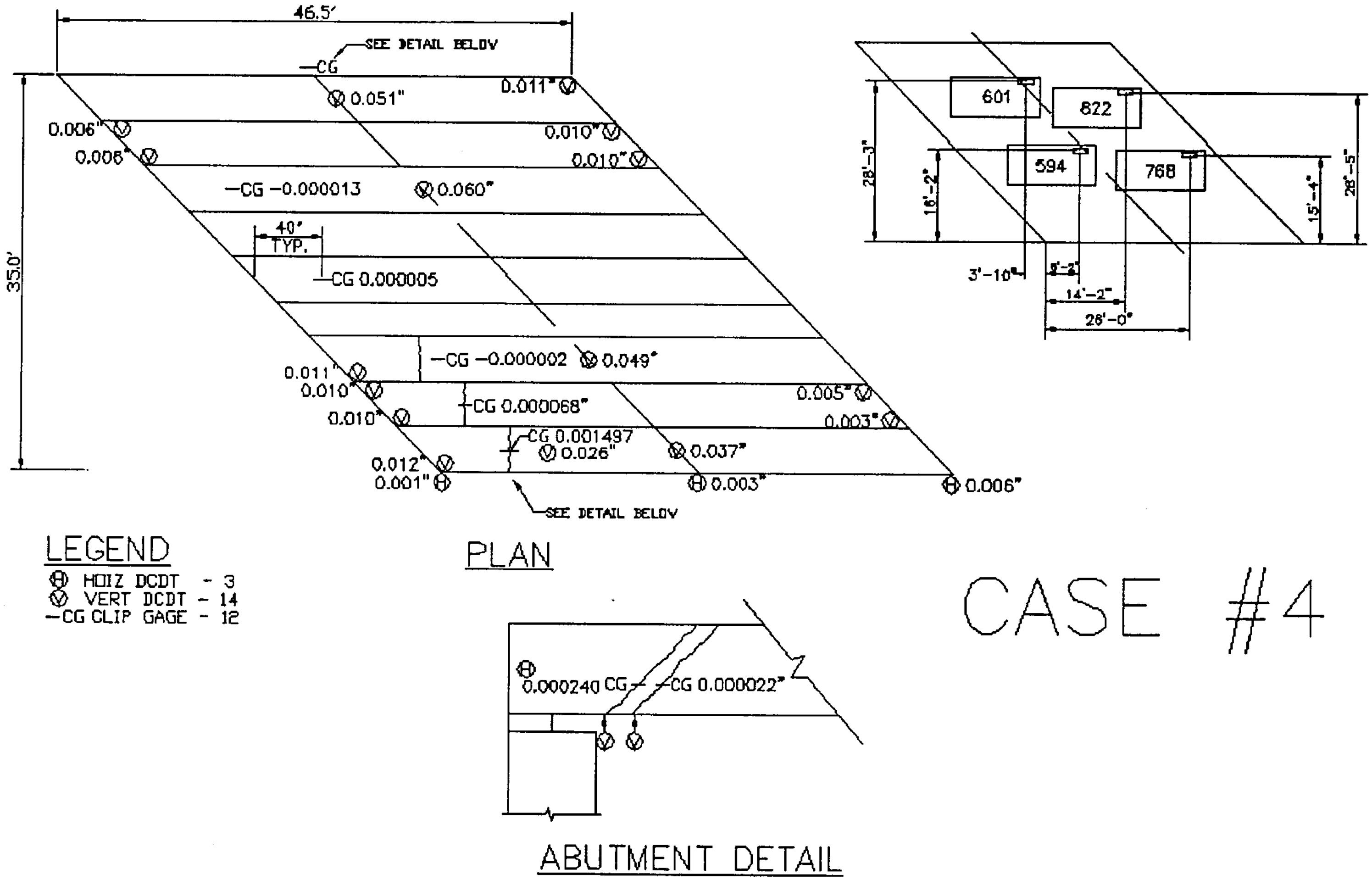


Figure 2.6 - Load Case #4

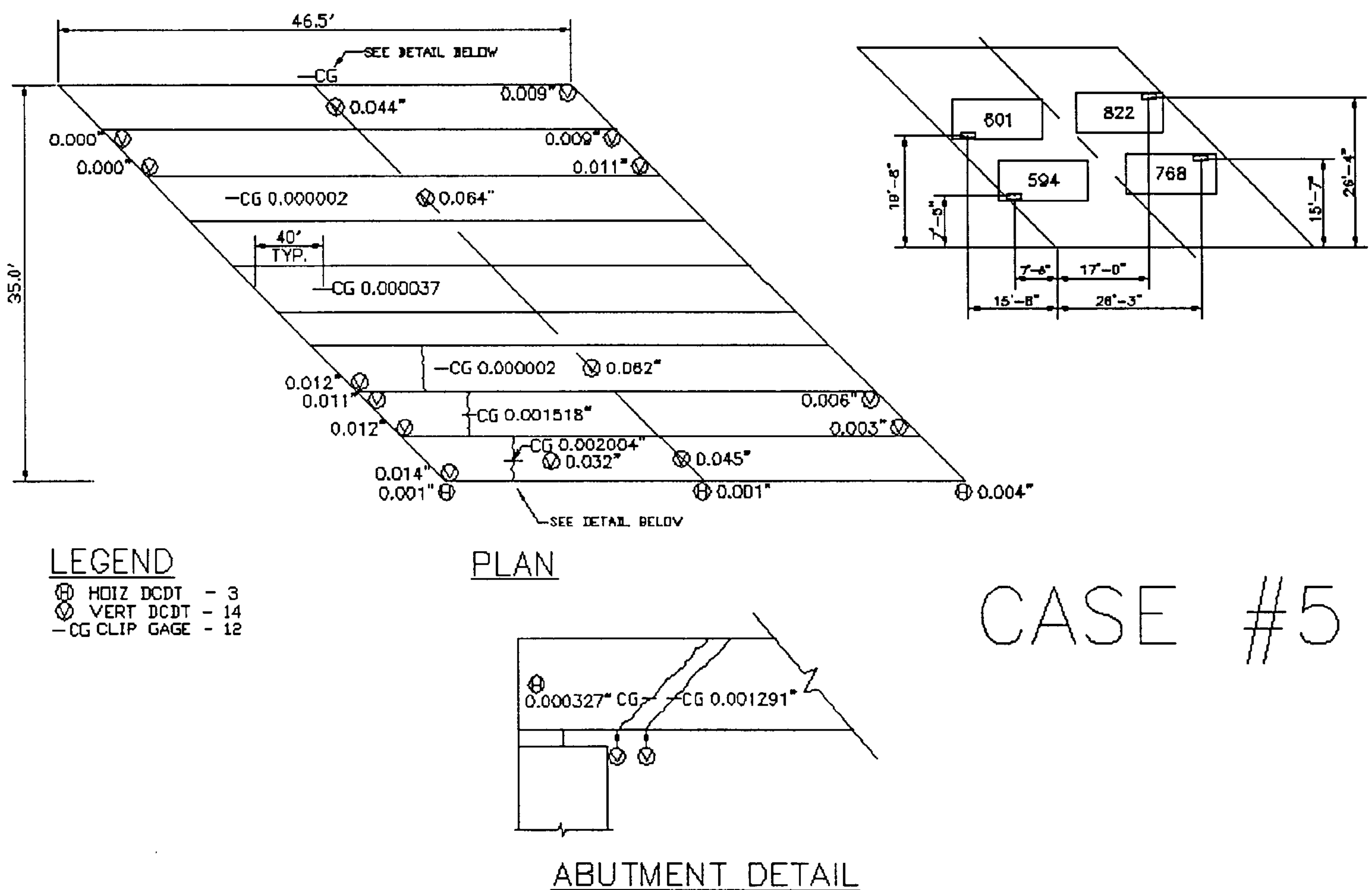


Figure 2.7 - Load Case #5

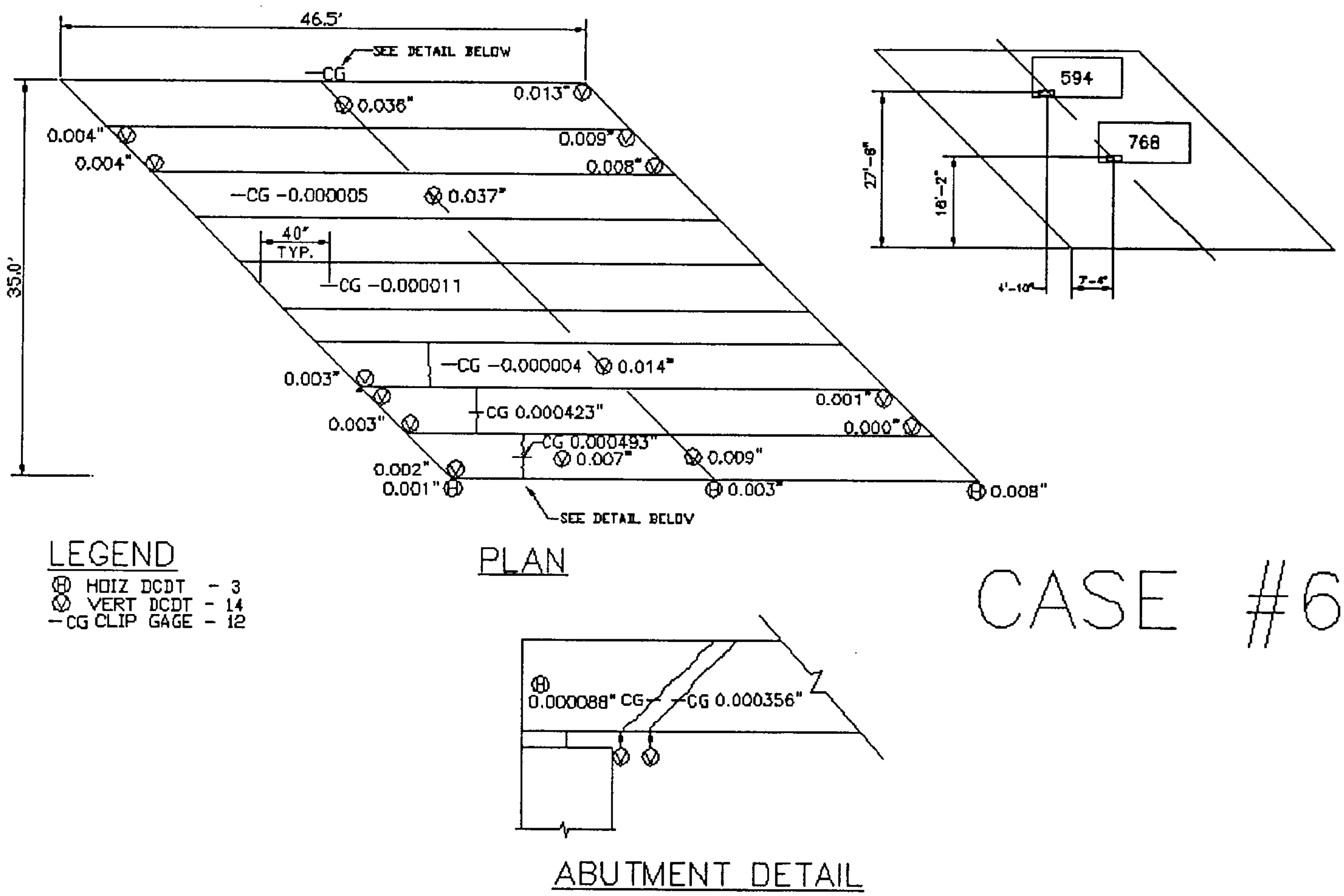


Figure 2.8 - Load Case #6

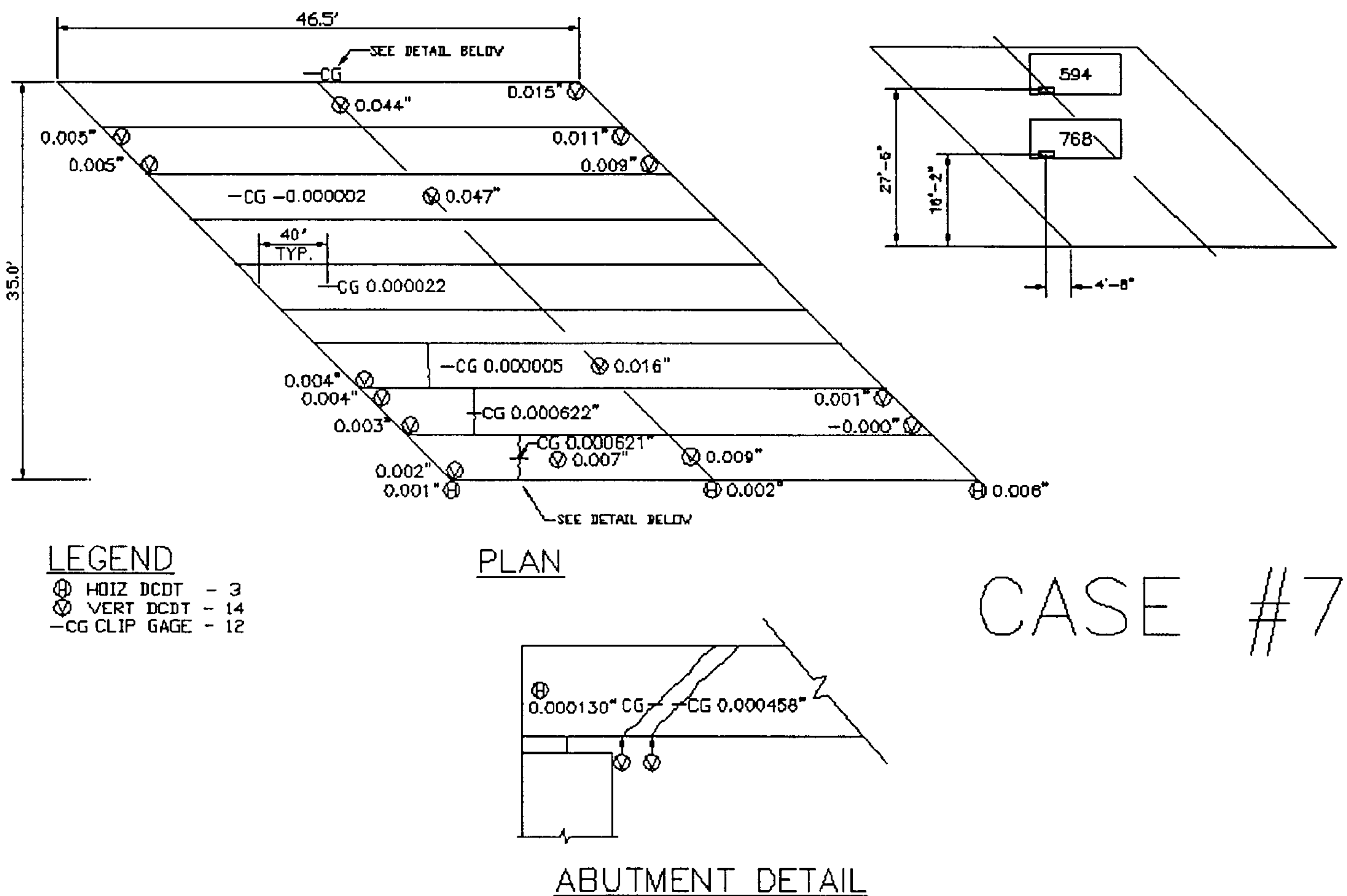


Figure 2.9 - Load Case #7

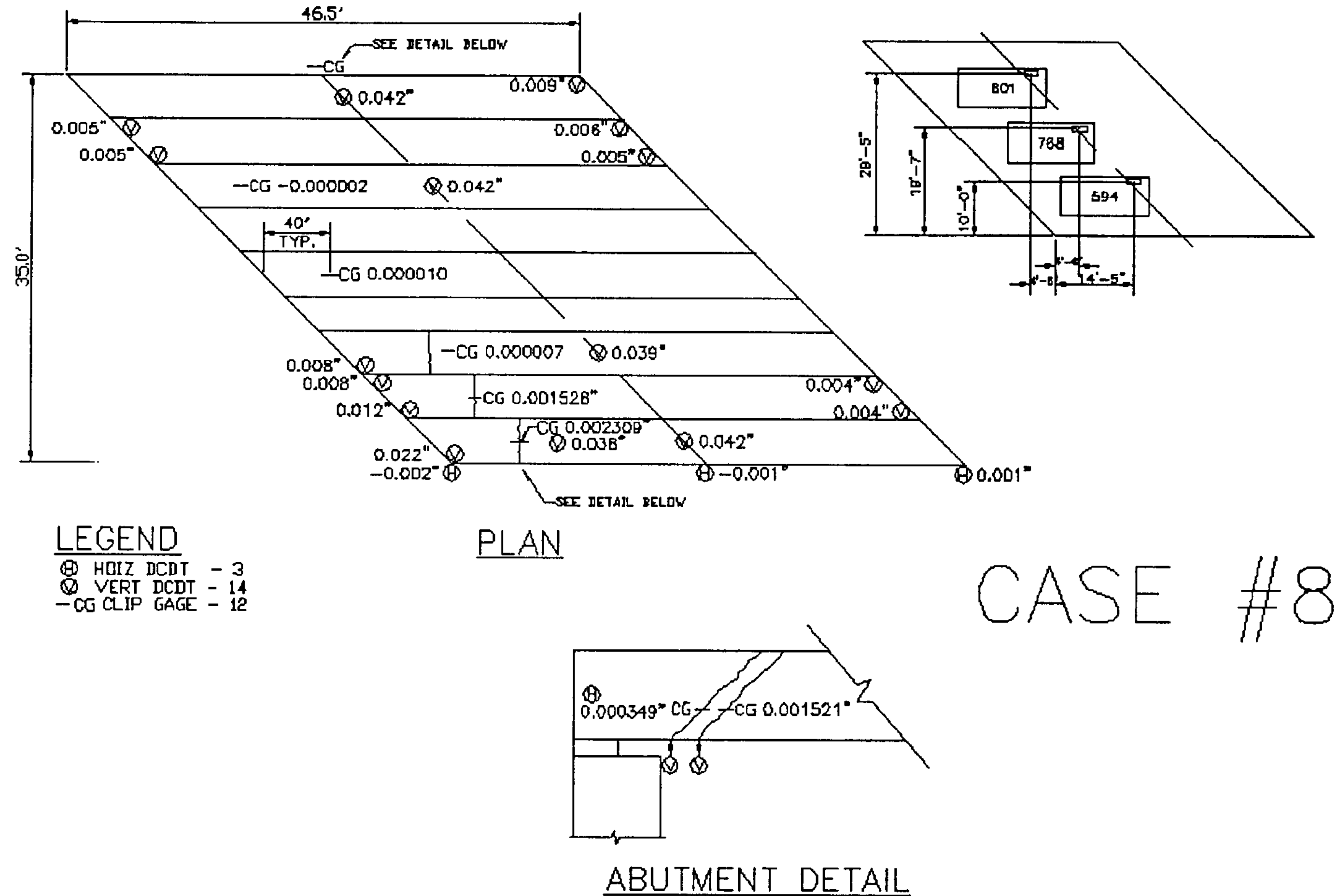


Figure 2.10 - Load Case #8

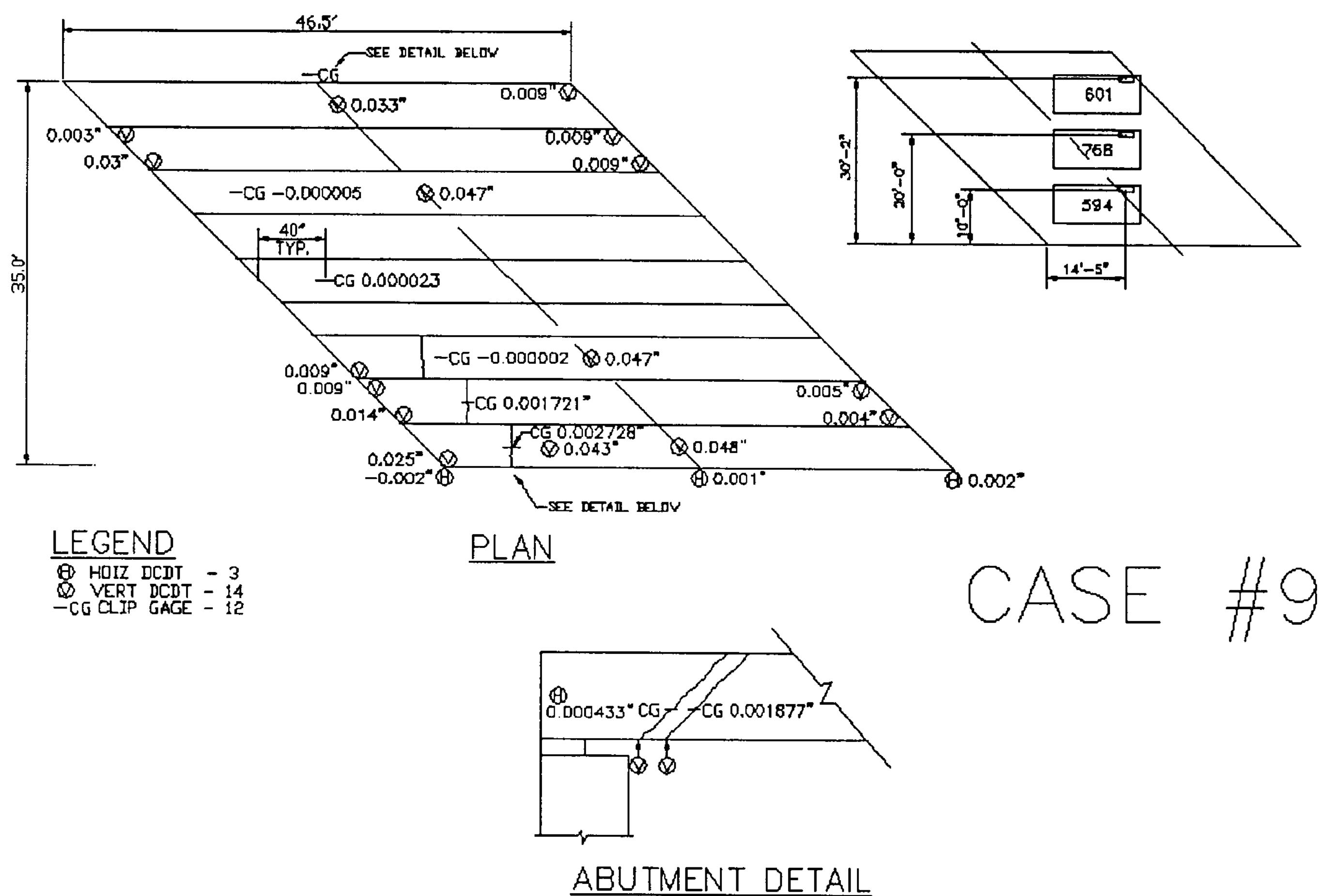


Figure 2.11 - Load Case #9

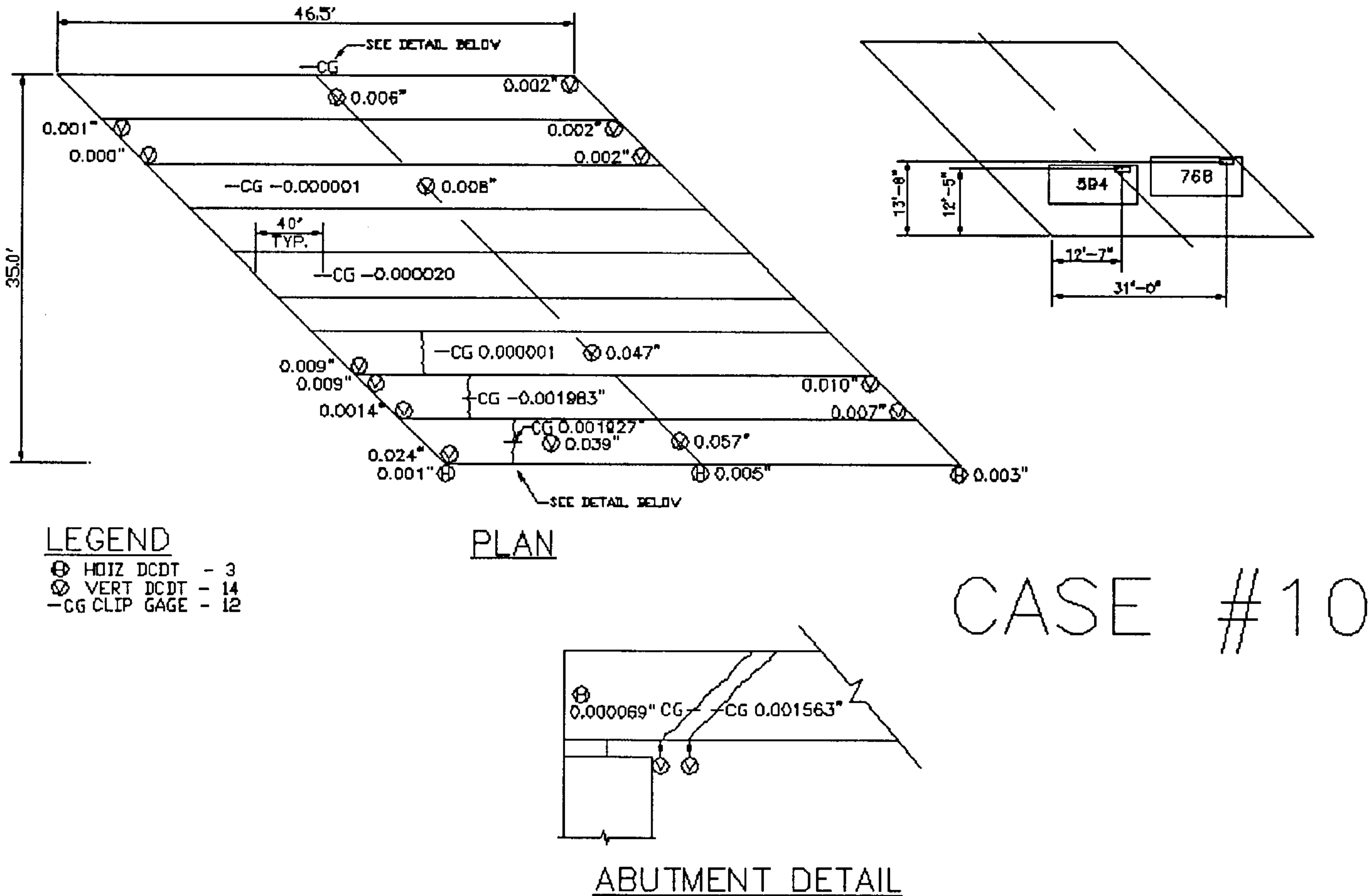


Figure 2.12 - Load Case #10

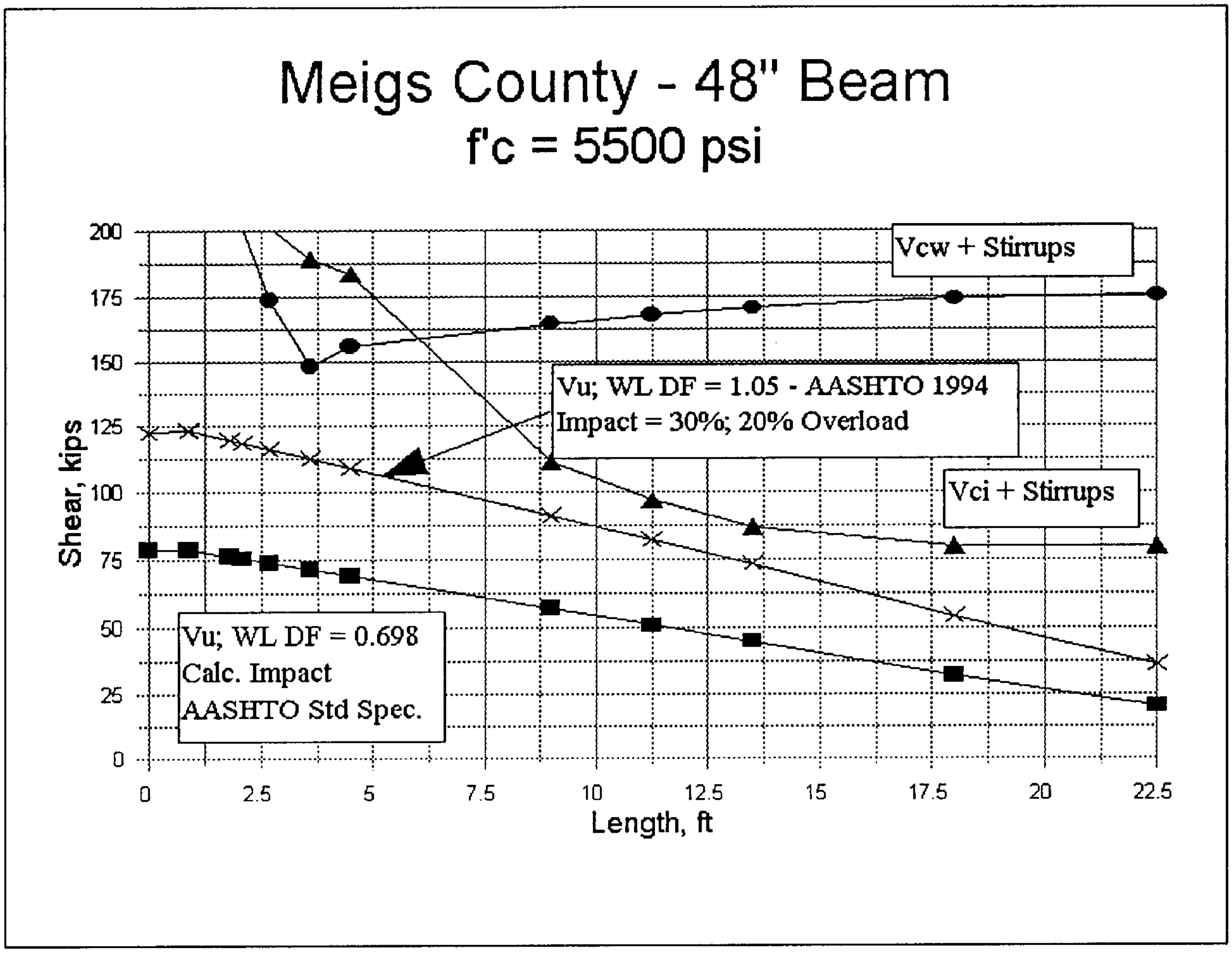


Figure 2.13 - Design Shear Forces

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CHAPTER 3

SUMMARY AND CONCLUSIONS

3.1 Summary

Bridge MEG 124-6.78 was a single span, composite, adjacent, prestressed box girder bridge. The bridge consisted of 9 beams and had a 45° right forward skew. Because it was on a curve, the bridge was superelevated. After being in service about 1 year, the three edge beams on the inside of the curve showed extensive cracking. Diagonal and vertical cracks were visible on the facia beam. Subsequent inspection revealed several construction errors, including loose bearing pads.

In an attempt to determine the cause of cracking, the bridge was subjected to both static and dynamic truck load testing. While this testing could not determine the cause of failure, it did eliminate several possibilities.

3.2 Conclusions

Based on the test results:

1) Under 10 different static load combinations, the bridge showed the expected load distributions between the beams. Uniform loads were equally shared while loads which were more toward one edge were distributed more to that end of the bridge, but the distribution was still reasonable. It did not appear that one or two beams were carrying the entire load. Therefore, the cause of failure was not overload in the edge beams due to improper load distribution.

2) Due to construction errors, the top of the abutments were not even and the bearing pads were not shimmed. Some of the bearing pads were loose and easily removed. Under load, the end of the beam with the loose pads twisted. Also, some twist was noted at the obtuse corners due to the skew of the bridge. However, the amount of twist was not enough to generate significant torsional shear stresses. Therefore, torsion due to loose bearing pads did not cause the failure.

3) Dynamic testing was done by running a full dump truck over the bridge at 15, 30 and 45 mph. The dynamic magnification factor (impact) was measured as the maximum dynamic midspan deflection in the beams divided by the maximum static deflection for the same beam. The maximum dynamic magnification factor was 30%. This is consistent with the value used in the *AASHTO Standard Specifications*.

4) The bridge was on a coal route. The effect of coal trucks running over the bridge was assessed. It was found that coal trucks had the effect of being about twice as heavy as the dump trucks. This is consistent with previous ODOT weigh-in-motion measurements and would also be consistent with the alternate military design loading. The coal trucks had a dynamic magnification factor of about 35%. The design impact factor is taken as 30%

maximum, but this higher impact factor is within the range of experimental accuracy. Therefore, it does not appear the bridge failed due to heavy coal trucks.

5) The original bridge design used the *AASHTO Standard Specifications for Highway Bridges*. These specifications do not have separate distribution factors for shear or account for skew in the member. A 1994 *AASHTO Guide Specification* has separate distribution factors for shear and accounts for skew. When the bridge is analyzed under the *Guide Specification*, the wheel load shear distribution factor rises from 0.698 to 1.05. However, if the beams are analyzed using the higher distribution factor, a 30% impact and allowing for 20% overload, the beams are still safe in shear.

The results of these tests show that beam failure did not occur due to normal traffic overloading the bridge. To find the cause of failure, additional work is needed.

3.3 Future Work

The bridge with the cracked beams was removed from service. During the demolition, several beam samples were saved for future testing. In addition, vibrating wire strain gages were embedded in some of the replacement beams for future testing. Future testing is to include:

1) Destructive testing of both cracked and uncracked beams. This will determine cause of failure, load capacity and beam behavior.

2) Testing of the new structure to test load distribution.

References

AASHTO Standard Specifications for Design of Highway Bridges, 16th Ed., American Association of State Highway and Transportation Officials, Washington, DC, 1992.

AASHTO Guide Specifications for Distribution of Loads for Highway Bridges, American Association of State Highway and Transportation Officials, Washington, DC, 1994.

AASHTO LRFD Bridge Design Specifications 1st Ed., American Association of State Highway and Transportation Officials, Washington, DC, 1994.

Miller, R., Weisgerber, F., Zhang, W., Dong, X., Greuel, A. and Long, T., *Cause of Cracking in Bridge #MEG 124-6.79,* Report to Sponsors, Ohio Department of Transportation and Federal Highway Admin., University of Cincinnati, March, 1999.