

INVESTIGATION OF THE DECEMBER 16, 1999 FATAL COLLAPSE OF A REINFORCING STEEL CAGE AT THE PIER WB-12 OF I-895 BRIDGE IN RICHMOND, VIRGINIA

U. S. Department of Labor
Occupational Safety and Health Administration

May 2000



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May 2000

TABLE OF CONTENTS

	PAGE
1.0 EXECUTIVE SUMMARY.....	1
2.0 CONDUCT OF THE INVESTIGATION.....	2
3.0 DESCRIPTION OF THE PROJECT, THE INCIDENT, AND THE COLLAPSE.....	4
4.0 ANALYSIS AND DISCUSSION.....	20
5.0 CONCLUSIONS.....	23
6.0 REFERENCES.....	24
APPENDIX A ENGINEERING ANALYSIS.....	25
APPENDIX B KEY REFERENCE MATERIALS.....	44

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1.0 EXECUTIVE SUMMARY

On December 16, 1999 at approximately 2:05 PM, a 44 ft tall reinforcing steel cage weighing 80 tons fell over and killed one of two workers working near the top of the cage. The wind at the time of the incident was reported to be about 20 to 25 mph. The deceased worker was caught between the collapsed reinforcing cage and the concrete footing surface. The other worker was thrown into an open space uninjured. At the time of the incident, the two workers were installing and tying the last portion of the horizontal reinforcements to the east face of the steel cage. The reinforcing steel cage was installed for the concrete Column 2 of Bridge Pier WB-12 on the west side of the James River. The accident occurred at the western most segment of the proposed Interstate Highway I-895 in the southeast suburb of Richmond, Virginia.

Personnel from the Virginia Occupational Safety and Health (VOSH) arrived at the scene within an hour of the incident. The Directorate of Construction, OSHA, Washington, DC, was requested by VOSH to provide assistance in the technical assessment of the collapse and in determining of the cause of the incident.

The OSHA investigation began soon after the incident and included: interviewing witnesses; inspecting the collapsed steel cage and observing the failures; taking photographs and related field measurements; reviewing construction drawings; and performing an engineering analysis. Based on the results of the investigation, OSHA concludes that:

1. The collapse of the reinforcing steel cage occurred because the Reinforcing Steel Erector had failed to provide adequate lateral support to the 44 ft high steel cage. The 20 mph wind triggered the collapse.
2. The existing guy wires were intended to plumb the reinforcing steel cage during its installation and were not capable of laterally supporting the steel cage.
3. Based on Items 1 and 2 above, the Reinforcing Steel Erector did not comply with the requirements of the OSHA Standard 1926.703 (d) (1).

2.0 CONDUCT OF THE INVESTIGATION

The following is a chronological list of major events during the investigation.

On December 16, 1999 within an hour of the incident, personnel from the Virginia Occupational Safety and Health (VOSH) arrived at the scene to start the investigation by interviewing witnesses and taking videos and photographs.

On December 17, 1999 the same personnel from the VOSH re-visited the incident site to continue interviewing witnesses and to take additional videos and photographs.

On December 22, 1999, at the request of the VOSH, two engineers from the Federal OSHA Directorate of Construction (DOC), accompanied by the Senior Compliance Safety and Health Officer of the VOSH, visited the incident site to observe the surroundings, inspect the collapsed steel cage, take photographs and field measurements, and attend a meeting with the General Contractor, Bridge Contractor, Reinforcing Steel Erector and others. In this visit, DOC received the following information:

- Contract Drawings of the Route 895 Connector, River Crossing -- WB Bridge & EB Bridge, Foundations & Columns, Piers WB-12 / EB-13 & WB-13 / EB-14, Submittal 16, Issued for Construction by FD/MK, 13 Sheets, Dated January 22, 1999.
- Sketch of the as-installed condition of the reinforcing steel cage prior to the collapse and the associated calculations for the weight of the steel cage, Prepared by RECCHI America, Inc., No date.
- Architectural Rendering of the Proposed I-895 and I-95 Interchange.
- 45 minutes video tape on the incident, taken by VOSH.

On December 28, 1999, after reviewing of the above materials, the same personnel of Federal OSHA and VOSH re-visited the incident site to inspect the collapsed steel cage again, take additional photographs and measurements, and conduct a meeting with the Reinforcing Steel Erector. In the second visit, DOC received the following additional information:

- Virginia Department of Transportation, Reinforcing Steel Tie Wires Requirement, January 1997.
- Mill Test Certification on a 3/8 inch diameter guy wire. However, the guy wires used to plumb the steel cage were 1/4 inch diameter in size.

- 36 photographs taken by VOSH, Employer's First Report of Accident, and a newspaper clip from Richmond Times on the incident.

On March 1, 1999 DOC received the wind speed records of December 12, 1999 through December 17, 1999 at the Richmond International Airport from the National Climatic Data Center. The wind speed at the incident site is assumed to be the same as that at the Richmond International Airport.

Throughout the course of the investigation, the Directorate of Construction worked together with the personnel from VOSH. The Senior Compliance Safety and Health Officer, Paul J. Trabosh, made significant contributions to this investigation.

Dinesh Shah, Structural Engineer of the Office of Engineering Services, took the field measurements, performed the computer analyses and checked the manual calculations of the draft report. Mohammad Ayub, Director of the Office of Engineering Services, reviewed the draft report and made valuable suggestions.

3.0 DESCRIPTION OF THE PROJECT, THE INCIDENT, AND THE COLLAPSE

3.1 Description of the Project

The proposed \$324 million, 8.8-mile Interstate Highway I-895, also known as Pocahontas Parkway, is owned by the Virginia Department of Transportation. It will link the existing Virginia State Route 150, also called the Chippenham Parkway, and Interstate Highway I-295 near the Richmond International Airport. As shown in Figure 3-1, the proposed I-895 is located in the southeast suburb of Richmond, Virginia. The project in question is located at the western most segment of the proposed highway, where it intersects the existing Interstate Highway I-95. Figure 3-2 presents the architectural rendering of the I-895 and I-95 interchange over the James River and the approximate location where the incident occurred.

The photograph on the cover of this report shows the collapsed steel cage. This photograph was taken from the east side of the James River. Figures 3-3 and 3-4 present the site condition around the west (land) side of the concrete footing. Based on the field identification, the collapsed steel cage is at the location of Column 2 of Bridge Pier WB-12 as shown in Figure 3-5. From this figure, the concrete footing is supported by twenty drilled shafts. The Bridge Pier WB-12 is supported on the north half of the footing. The two adjacent 90 ft tall concrete columns in construction, as shown in Figures 3-3 and 3-4, are not related to the incident. They are Bridge Pier EB-13 columns and are supported on the south half of the footing.

The design arrangement of the steel reinforcements for the concrete column of the bridge pier is presented Figure 3-6. As indicated in this figure, the bottom portion of the steel cage is embedded in the concrete footing. Based on the information from Reinforcing Steel Erector (RSE), when the incident occurred, the footing concrete was approximately two months old and had achieved full strength.

3.2 Description of the Incident

On December 16, 1999 at approximately 2:05 PM, under the high winds, a 44 ft tall reinforcing steel cage collapsed and killed one of the two workers working near the top of the cage. The deceased worker was caught between the collapsed steel reinforcements and the concrete footing surface. The other worker was thrown clear into the open space between the east side of the concrete footing and the steel sheet pile wall along the west edge of the river. At the time of the incident, the two workers were installing and tie wiring the last portion of the horizontal reinforcements, also called "candy canes", to the east face of the steel cage. The wind speed at the time was about 20 to 25 miles per hour, based on the readings from the adjacent tower crane.

Based on the discussion with the RSE, in the middle of October 1999, the bottom level of the reinforcements of the steel cage was erected on top of the drilled shaft foundations and was laterally supported by a HP steel frame. The concrete for the footing was cast around October 20, 1999. When the footing concrete had achieved most of its strength, the HP steel frame was removed (around November 4, 1999). At this stage, the height of the steel cage was approximately 30 ft above the concrete footing.

The vertical extension of the steel cage was started on or about December 9, 1999, seven weeks after the footing was cast. In this extension, 20 ft long vertical reinforcements were tie wired to the bottom reinforcements with a lapping length of approximately 6.5 ft. As a result, the height of the steel cage was increased to 43.5 ft. After the installation of the skeleton of the four circular cells, it was found that the partially extended steel cage was out of plumb. The top of the steel cage was reported to be leaning about 6 to 8 in. to the west and 3 to 4 in. to the north. To maintain the verticality of the steel cage, for the proper installation of the remaining vertical reinforcements, two guy wires to the east and one guy wire to the south were installed near the top of the steel cage. In addition, two additional guy wires were also installed to the west. As indicated by the RSE, the five installed guy wires were only intended to maintain the plumbness of the steel cage. These guy wires were not designed as a lateral support to the steel cage.

3.3 Description of the Collapse

The condition of the collapsed steel cage from its bottom to top is presented in Figures 3-7 through 3-10. As indicated in these figures, all reinforcements in the steel cage fell and collapsed as a group. In addition, no significant relative movements were observed between the vertical and horizontal reinforcements at their intersections. However, the horizontal reinforcements were severely rotated with respect to the bent vertical reinforcements at the intersections. This observed behavior was consistent with the information from the RSE, that the tie wires were only intended to maintain the reinforcements in position at the intersections. The tie wires were not intended to provide rigid connections between the horizontal and vertical reinforcements. In addition, as per the field observations, tie wires were generally installed at every third intersection, instead of every other intersection as was the normal practice. According to the RSE, the reduced tie wired intersection was approved by the project design engineer due to the congestion of the reinforcements.

Based on the field counts, at the time of the incident, there were 114 horizontal circular ties installed on the steel cage. Given the vertical spacing for the ties as 3.94 in. as defined in Figure 3-6, the ties were installed up to a height of 37.4 ft. Mechanical couplers were used for the bottom 60 ties of each cell to a height of 20 ft from the top of the concrete footing. These circular rings would provide some constraints on the direction of the lateral deflection of the vertical reinforcements during the collapse. A metal concrete form with the timber work platform was observed under the collapsed cage. In accordance with the SRE, the

concrete form was not connected to the steel cage. However, it did influence the collapsed shape of the steel cage, due to its presence.

The condition of the top of the steel cage at its south end is presented Figures 3-11 and 3-12. These photographs were taken from the open space between the east side of the concrete footing and the steel sheet pile wall which the second worker fell into. The upper end of the southwest, south and southeast guy wires were also shown in Figure 3-11. A close-up view of the upper end of the southeast guy wire is presented in Figure 3-12.

The resting condition of the steel cage after the collapse is presented Figures 3-13 through 3-15. As a result of the collapse, a significant amount of tie wires were broken and fell on top of the concrete footing as shown in Figure 3-15. The last section of the extension for the west face of the steel cage, as shown in Figure 3-16, was not installed prior to the collapse. This section consisted of 23 vertical reinforcements and 42 horizontal reinforcements. In addition, according to the RSE, the lower lapping end of the outer vertical reinforcements coincided with the upper end of the un-extended inner vertical reinforcements at 23.5 ft as shown in Figure 3-14. This value was consistent with the vertical space counts as indicated in Figure 3-16.

The condition of the northwest, southwest and south guy wires after the collapse of the steel cage is presented in Figures 3-17 through 3-22. All of the three wires were field measured as 1/4 inch in diameter. Figure 3-17 shows the lower connection point of the northwest guy wire at the north end of the steel cage for Column 1 of the same pier. Figure 3-18 presents the damaged condition and the size measurement of the northwest guy wire. Since the northwest guy wire was damaged during the collapse and was removed from the steel cage, the location of its upper end could not be field identified.

Figure 3-19 shows the lower connection point of the southwest guy wire at the south end of the steel cage for Column 1 of the same pier. The lower and upper ends of the southwest guy wire are presented in Figures 3-20 and 3-21, respectively. Since no damages were observed on the guy wire and no latches were present on any of the hooks, it is believed that the guy wire system was disengaged prior to the collapse of the steel cage.

The upper end of the south guy wire is already identified in Figures 3-10 and 3-11. Figure 3-22 presents the middle portion of the south guy wire, it had a hook without latch. The lower end of this guy wire was anchored to the concrete footing. Since the south guy wire was installed approximately along the major axis of the steel cage, perpendicular to the direction of the collapse, it was neither stretched nor damaged. The last two (northeast and southeast) guy wires on the collapsed (east) side of the steel cage were not tensioned, therefore, they were not damaged.

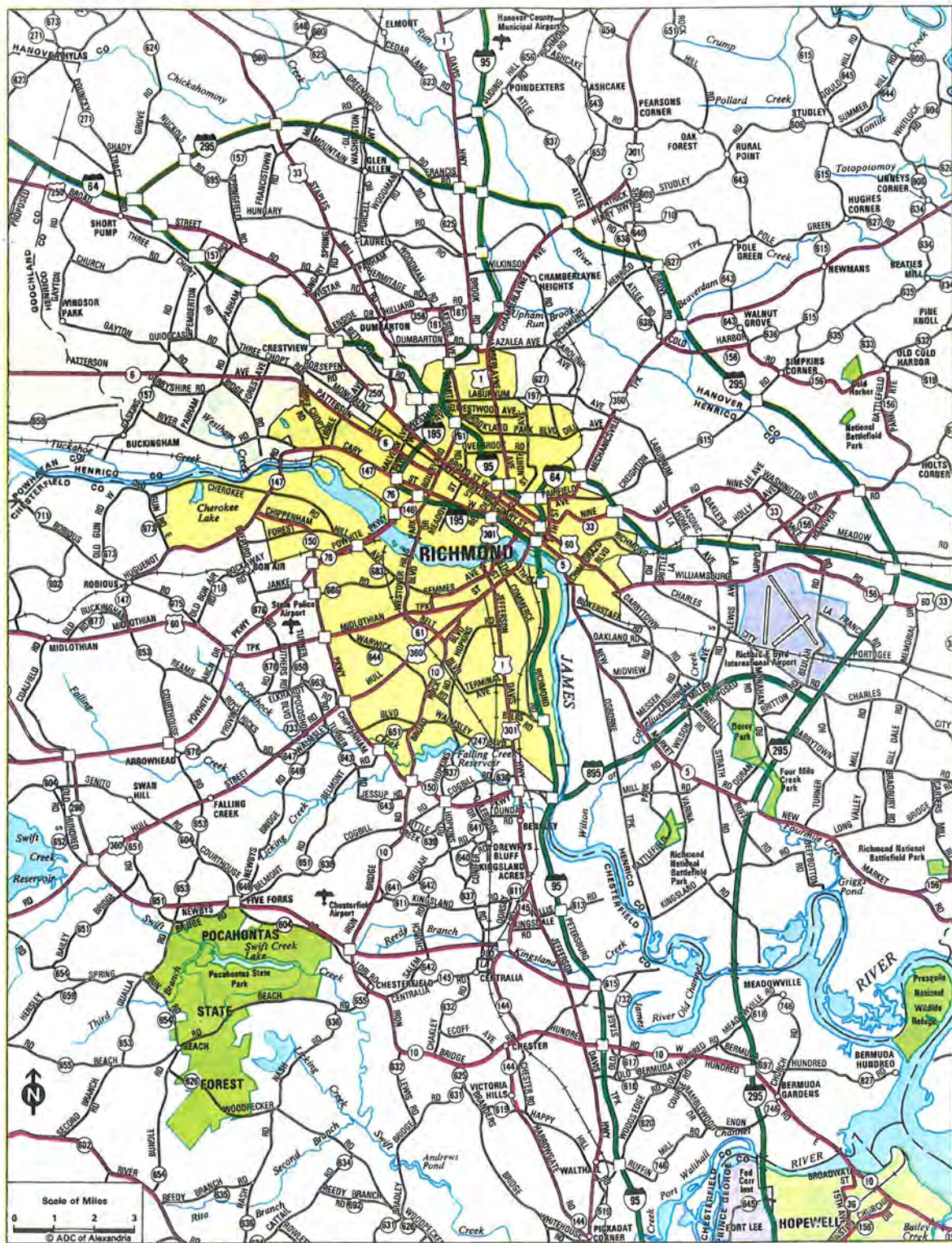


Figure 3-1 Project Location Plan in Richmond, Virginia (United States Road Atlas, by American Map Corporation, 1995).

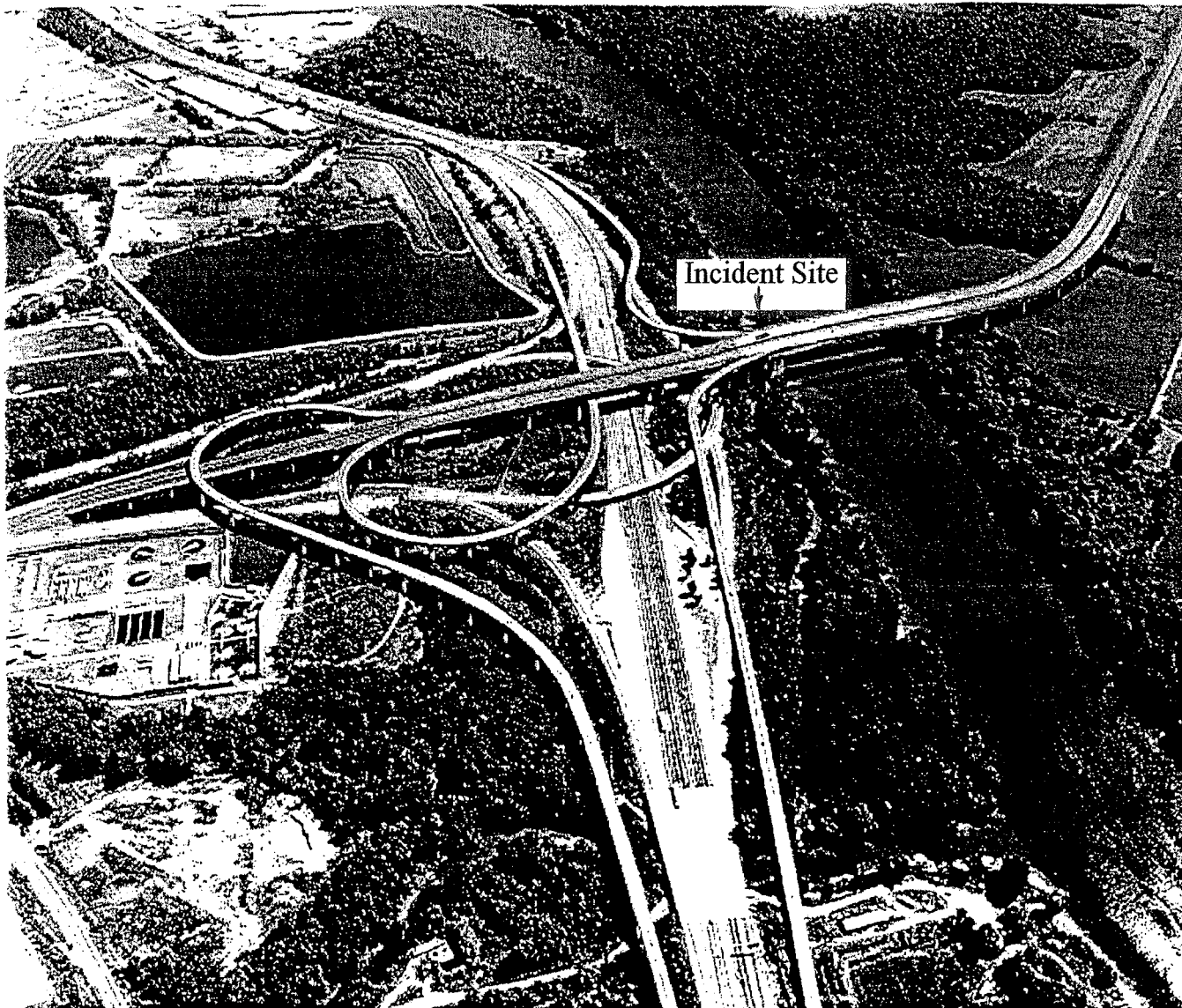


Figure 3-2 Architectural Rendering of the Proposed Interstate Highway I-895 at the Interstate Highway I-95 (Provided by the Steel Reinforcement Erector).



Figure 3-3 Site Condition around the Concrete Footing (Looking toward Northeast).



Figure 3-4 A Close-up View of Figure 3-3.

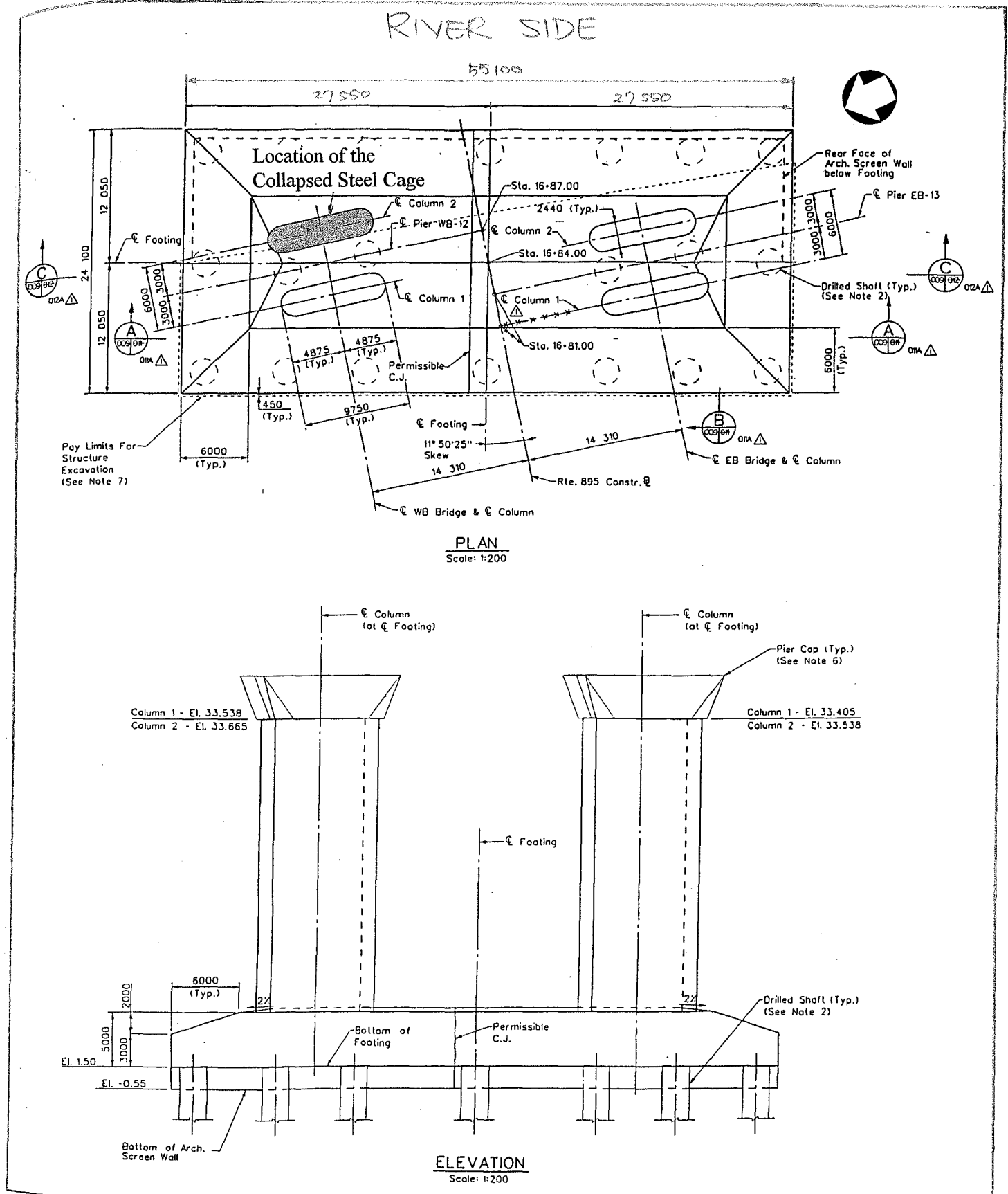


Figure 3-5 Plan and Elevation of the Concrete Footing, Bridge Pier WB-12 and Bridge Pier EB-13 (Contract Drawings, Sheet 5 of 13, 80% Reduction).

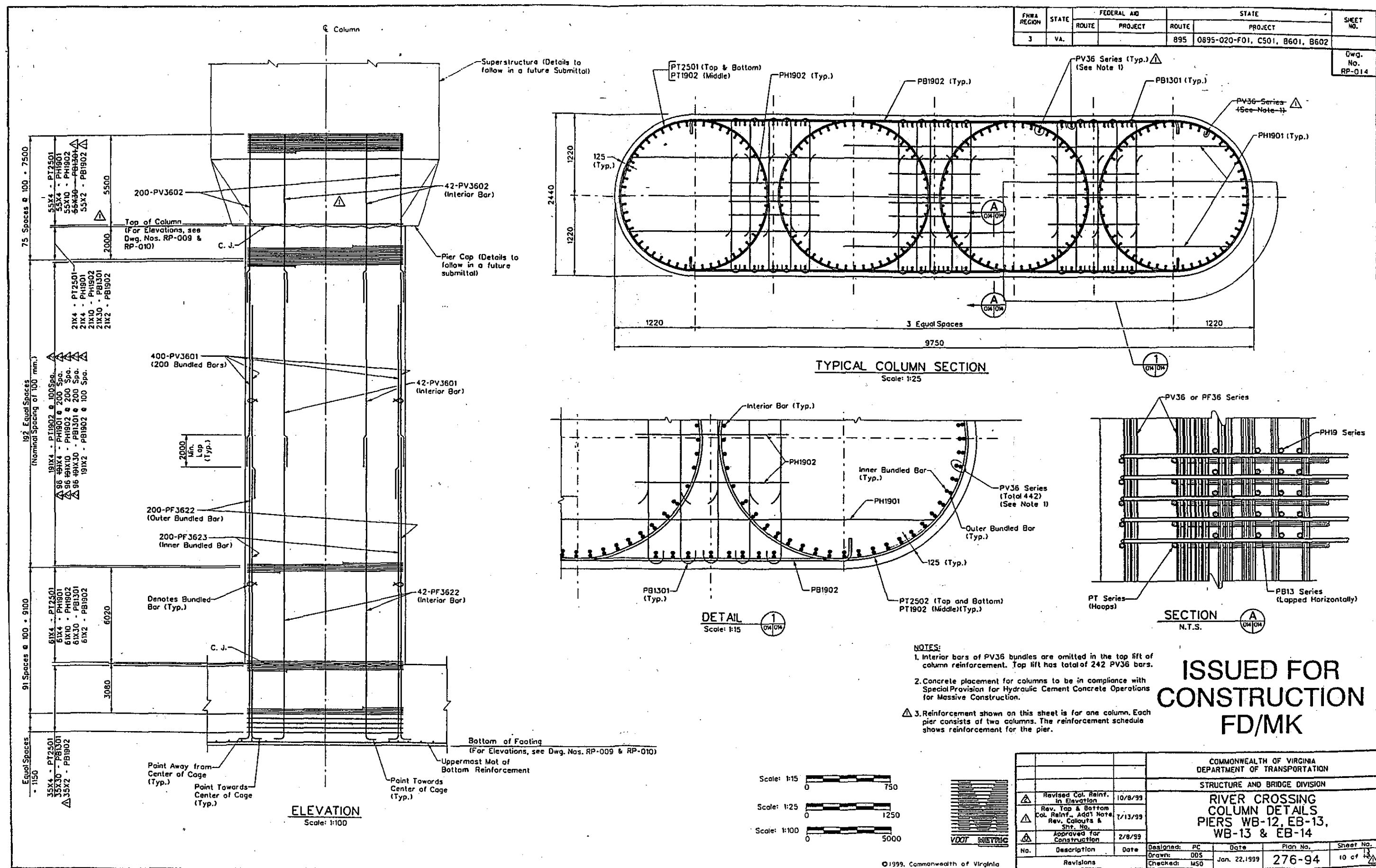


Figure 3-6 Steel Reinforcement Details for Column 2 of Bridge Pier WB-12 (Contract Drawing, Sheet 10 of 13, 90% Reduction).



Figure 3-7 The Bottom Fixed End of the Collapsed Steel Cage (Looking toward North from the Concrete Footing).



Figure 3-8 The Lower Portion of the Collapsed Steel Cage (Looking toward North from the Concrete Footing).



Figure 3-9 The Middle Portion of the Collapsed Steel Cage (Looking toward North from the Concrete Footing).



Figure 3-10 The Upper Free End of the Collapsed Steel Cage (Looking toward North from the Concrete Footing).

Please note that the red color hook around the middle of the photograph was the upper end of the south guy wire.



Figure 3-11 The Upper Free End of the Collapsed Steel Cage (Looking upward from the Alley between the Side of the Concrete Footing and the Steel Sheet Piling).



Figure 3-12 The Close-up View of Figure 3-11.

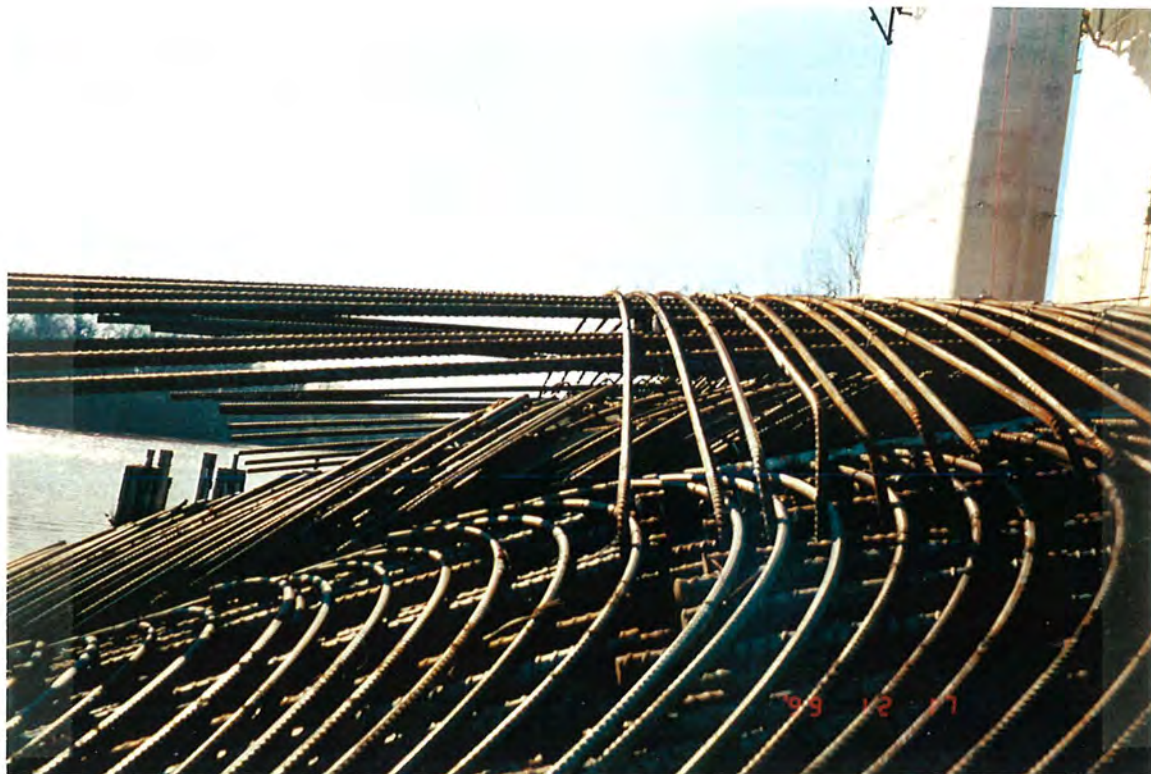


Figure 3-13 The Condition of the Collapsed Steel Cage at 23.5 Foot from the Bottom Fixed End (Looking toward South from the Concrete Footing).
Please note the separation of the vertical reinforcements from the west face of the steel cage.

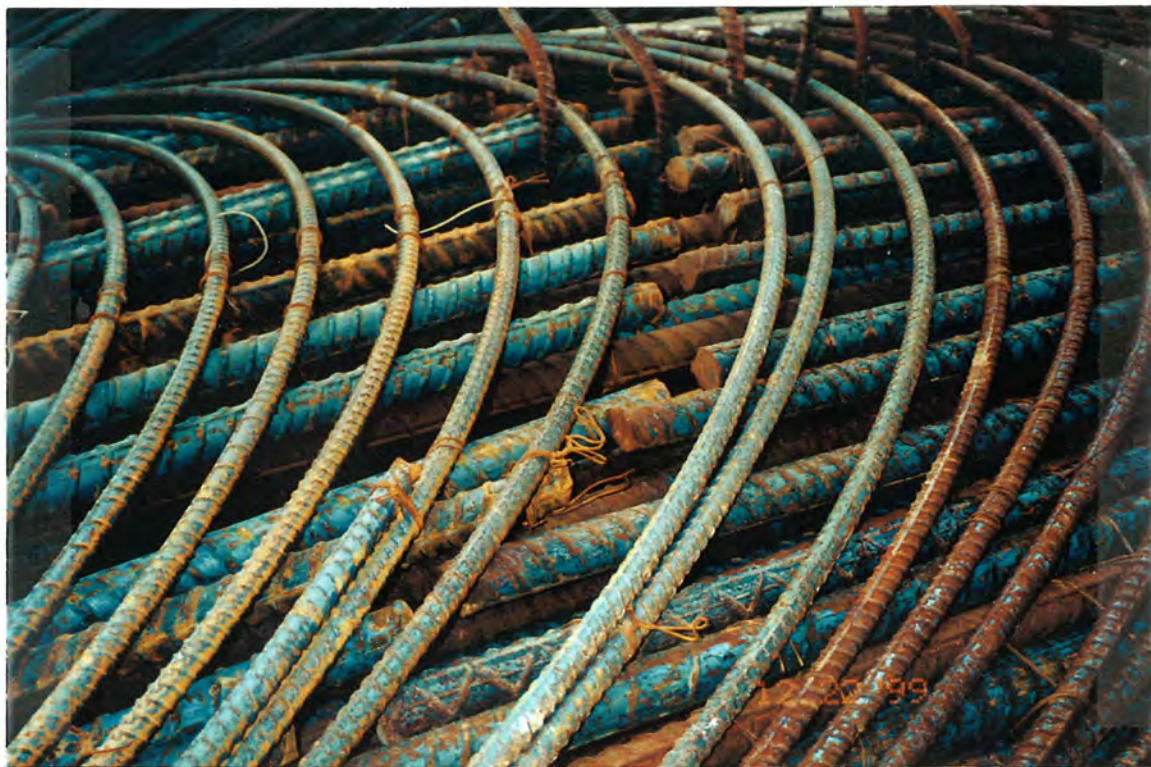


Figure 3-14 A Close-up View of Figure 3-13.



Figure 3-15 The Condition of the Collapsed Steel Cage at its North Fixed End (Looking from the Concrete Footing).

Please note the separation between the vertical reinforcements and the closed horizontal circular reinforcements on the right of the photograph, and the presence of the broken tie wires on top of the concrete footing.



Figure 3-16 The Last Section of West Face Extension Did Not Install to the Collapsed Steel Cage.

Please note that there were 23 vertical reinforcements and 42 horizontal reinforcements in this section.



Figure 3-17 The Lower Connection Point of the Northwest Guy Wire at the North End of the Steel Cage for Column 1 of Bridge Pier WB-12.

Please note that there was no latch on the hook near the bottom of this figure. However, scratching marks were observed on this hook.



Figure 3-18 The Damaged Northwest Guy Wire.

Please note that the size this guy wire was measured as 1/4 inch in diameter.

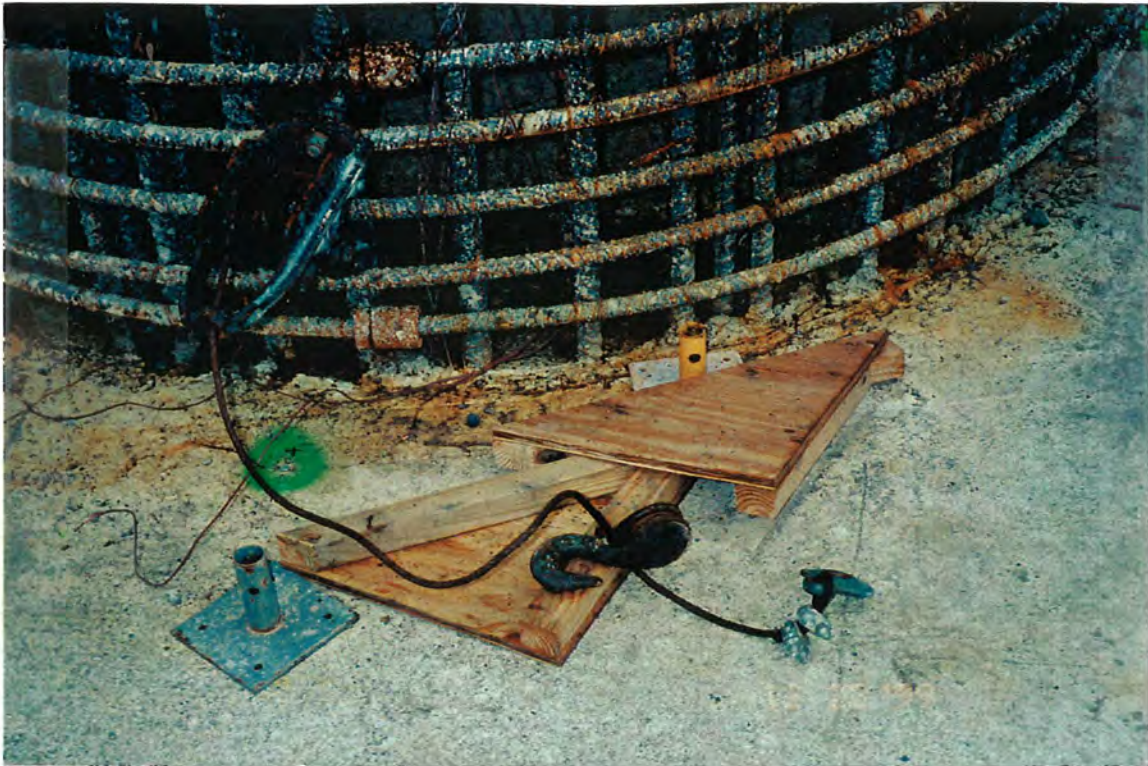


Figure 3-19 The Lower Connection Point of the Southwest Guy Wire at the South End of the Steel Cage for Column 1 of Bridge Pier WB-12.
Please note that there were no latches on neither hooks nor any scratching marks observed on these hooks.



Figure 3-20 The Lower End of the Southwest Guy Wire.
Please note that there were no damages observed on this end of the wire.

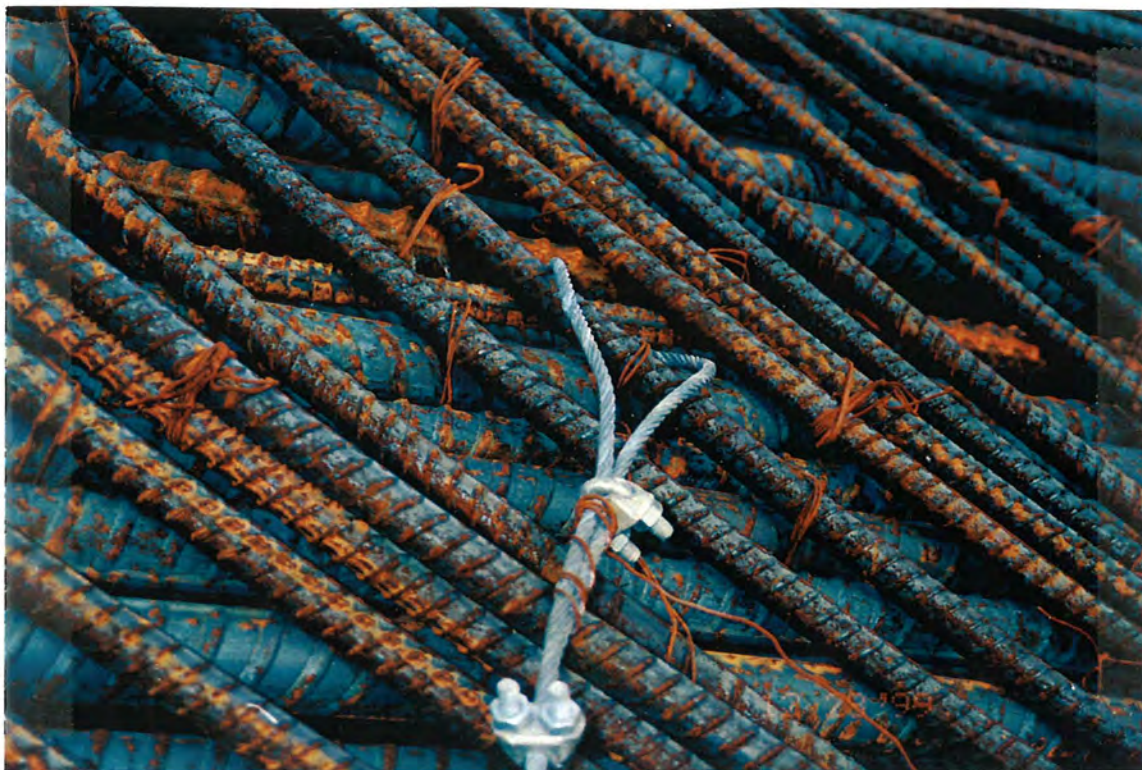


Figure 3-21 The Upper End and Connection Point of the Southwest Guy Wire.
Please note that there were no damages observed on this end of the wire neither.



Figure 3-22 The Middle Portion of the South Guy Wire.
Please note that there was no latch on the hook in this portion of the guy wire.

4.0 ANALYSIS AND DISCUSSION

To determine the cause of the collapse, the as-installed condition of the steel cage and its associated guy wires immediately prior to the collapse was re-constructed in Chapter 3.0. The actual wind speed on the day of the incident from the National Climatic Data Center (NCDC) was used in the engineering analysis. In this analysis, the yield strength of structural elements, instead of the allowable stress, was used. In addition, no load factors were applied in the failure analysis. All of this information is included in Appendix A.

The basic premise for the analysis was based on the following observations on the collapsed steel cage:

- All reinforcements in the steel cage fell as a group.
- The horizontal reinforcements rotated up to 45 degree with respect to the bent vertical reinforcements.
- No significant relative translations occurred between the horizontal and vertical reinforcements at their intersections.
- The vertical reinforcements were the only structural elements that supported the weight of the steel cage.

Considerations were given to examine whether the horizontal circular ties provided any rigidity to the vertical reinforcing bars to resist lateral loads. If it did, the steel cage would have a very high degree of resistance against any lateral load because of a many fold increase in its section modulus. On the other hand, if the ties did not provide any rigidity to the vertical reinforcing steel to form a closed rigid frame, the lateral loads will be resisted by individual reinforcing bars, highly susceptible to flexure and lateral displacements.

Our field inspections revealed that the circular ties were connected to the vertical reinforcements with nominal tie wires and were incapable of providing any stiffness to the intersection. The angle of 90 degrees between the intersecting reinforcements was easily compromised under the lateral load as shown in different photographs previously discussed. So, it was decided to consider the bending stiffness of individual reinforcements instead of the entire steel cage as one element. However, the lateral loads will be equally distributed to all the vertical reinforcements because they were interconnected and were of the same size. The presence of the circular ties resulted in equal deflection of all vertical reinforcements, as observed in the field.

Given the above, two main assumptions were made:

1. The section modulus of the collapsed steel cage is the sum of the section modulus of each individual vertical reinforcement.
2. The horizontal deflection of the steel cage can be simulated by a representative vertical reinforcement.

The above assumptions along with other detailed assumptions are listed in Section A-1 of Appendix A.

The total weight and the vertical location of the centroid of the steel cage were calculated in Tables A-1 and A-2, respectively. The weight of the steel cage was estimated as 80 tons. The centroid of the steel cage was estimated as 18.9 ft above the concrete footing. All of this information is included in Section A-2.

The height of the representative vertical reinforcement was calculated as 39.2 ft in Section A-3-1. It was the average height of all (442) installed vertical reinforcements. The wind area for the representative reinforcement was calculated from the projected area of all installed vertical reinforcements including their lapping areas and divided by the total number of the vertical reinforcements. This information is included in Section A-3-1. Due to the potential shielding effects, other projected areas of all exterior and interior horizontal reinforcements were not considered. The wind areas used in the analysis was conservatively assumed, since the eliminated wind areas of the horizontal ties were more than the shielded areas of the vertical bars. In addition, the wind exposed areas of the horizontal ties would increase as the steel cage started to bend.

The wind coefficients for the representative reinforcement was selected based on the requirements of Reference 1. However, the Important Factor (I) of 1.0 was used to simulate the actual wind condition in the analysis. All of this information is included in Section A-3-3. Since the steel cage was classified as a flexible structure, the Gust Response Factor (G) was calculated by a rational analysis that incorporates the dynamic properties of the steel cage. This information is presented in Section A-3-4. The typical wind load on the representative reinforcement was calculated as 0.620 plf in Section A-3-5.

Although the northwest and southwest guy wires were not intended to resist any wind load (from the west direction) on the steel cage, the strength contribution of these two wires should be considered in the failure analysis. However, since the southwest guy wire was disengaged at its lower end without any damages prior to the collapse of the steel cage as documented in Figures 3-19 and 3-20. Thus, its contribution was not considered in the analysis. As presented in Section A-4-1, the wind induced tension on the northwest guy wire was calculated as 10,544 pounds, which is exceeding its nominal strength (breaking load) of 7,000 pound, based on Reference 3, Page 95. Thus, the northwest guy wire would fail. This guy wire actually failed as shown in Figure 3-18.

The wind induced stress and horizontal deflection of the representative reinforcement after the failure of the guy wires is calculated in Section A-4-2. Please note that the calculated stress and deflection of the representative reinforcement were also representing those of the steel cage. Since the steel cage weighed 80 tons, the gravity ($P\Delta$) effect on its deflected shape shall be considered as per Reference 4, Page 6-41. The $P\Delta$ analysis was also performed on the representative reinforcement with the share weight (363 pounds) of the steel cage in Section A-4-3. The resulting bending stress of the representative reinforcement was 263.8 ksi, which was much greater than its yield strength of 60 ksi. Thus, the representative reinforcement yielded and the steel cage collapsed.

Based on the above analysis, under a 20 mph wind, the steel cage could not laterally support itself, with or without the installed guy wires. Thus, the Reinforcing Steel Erector (RSE) did not provide adequate lateral support to prevent overturning and to prevent collapse of the reinforcing steel cage for a bridge pier per the requirement of the OSHA Standard 1926.703 (d) (1). In addition, this requirement is further emphasized in ANSI/ASCE 7-88 Paragraph 6.1.1— Adequate temporary bracing shall be provided to resist wind loading on structural components and structural assemblages during the erection and construction phases.

In accordance with the records from the NCDC, the peak winds in the five days prior to the incident (from Sunday, December 12, 1999 through Thursday, December 16, 1999), were 9, 11, 17, 17 and 26 mph, respectively. Thus, the peak winds were in an increasing trend toward December 16, 1999 the day of the incident.

Based on the RSE, the installation of the upper extension of the steel cage started on or about December 9, 1999 with the addition of the skeleton of the four circular cells. The remaining vertical reinforcements would be installed to the skeleton cells on top of the steel cage. The weight and the projected wind area of the skeleton cell were about one-fifth of those for the fully installed cell. Thus, as the installation continued, both the weight of the steel cage and the wind loads on the steel cage would increase until it could not support itself and collapsed on December 16, 1999. In addition, from the NCDC records, the direction of the peak winds on December 16, was 260 degrees (from west), which almost coincides with the direction of the collapse of the steel cage. Therefore, the collapse of the unbraced steel cage was triggered by the effect of high winds.

5.0 CONCLUSIONS

Based on the result of the investigation, the Occupational Safety and Health Administration concludes that:

1. The collapse of the reinforcing steel cage occurred because the Reinforcing Steel Erector had failed to provide adequate lateral support to the 44 ft high steel cage. The 20 mph wind triggered the collapse.
2. The existing guy wires were intended to plumb the reinforcing steel cage during its installation and were not capable of laterally supporting the steel cage.
3. Based on Items 1 and 2 above, the Reinforcing Steel Erector did not comply with the requirements of the OSHA Standard 1926.703 (d) (1).

6.0 REFERENCES

1. ANSI/ASCE 7-88 Minimum Design Loads for Buildings and Other Structures, by American Society of Civil Engineers, November 27, 1990.
2. Wind Engineering, by Henry Liu, Prentice Hall, Englewood Cliffs, New Jersey, 1991.
3. Wire Rope Users Manual, by Wire Rope Technical Board, Second Edition—Second Printing, November 1985.
4. Manual of Steel Construction, Load & Resistance Factor Design, Volume I, by American Institute of Steel Construction, Inc., Second Edition, 1994.
5. Contract Drawings of the Route 895 Connector, River Crossing—WB Bridge & EB Bridge, Foundations & Columns, Piers WB-12 / EB-13 & WB-13 / EB-14, Submittal 16, Issued for Construction by FD/MK, 13 Sheets, Dated January 22, 1999.
6. STAAD-III, Structural Analysis and Design Software, by Research Engineers, Inc., Revision 22.0W, October 1996.

APPENDIX A

ENGINEERING ANALYSIS

APPENDIX A

ENGINEERING ANALYSIS

- A-1 Basic Assumptions
- A-2 Estimation of the Total Weight and Vertical Location of the Centroid of the Reinforcing Steel Cage at the Time of the Incident
- A-3 Estimation of the Wind Load on a Representative Vertical Reinforcement at the Time of the Incident
 - A-3-1 The average height of the representative reinforcement
 - A-3-2 The projected area normal to the wind for the representative reinforcement
 - A-3-3 The associated wind coefficients for the representative reinforcement
 - A-3-4 The gust response factor of the representative reinforcement
 - A-3-5 The typical wind load on the representative reinforcement
- A-4 Estimation of the Horizontal Deflection and Flexural Stresses on a Representative Vertical Reinforcement at the Time of the Incident
 - A-4-1 The tensile force on the guy wire due to the wind load
 - A-4-2 The wind induced stresses and horizontal deflection on the representative reinforcement after the failure of the guy wires
 - A-4-3 The second order ($P\Delta$) effects on the flexural stresses of the representative reinforcement

A-1 Basic Assumptions

Based on the field observation, the vertical reinforcements above the concrete footing were the only structural elements that support the weight of the steel cage. After the breaking of the guy wires, these reinforcements were the only structural elements to resist the wind load applied to the steel cage. Based on the deflected pattern of the steel cage after the collapse, the horizontal reinforcements did not remain in its original orientation with the vertical reinforcements, i.e., the tie wires were not capable to prevent the rotation of the horizontal reinforcements from the associated vertical reinforcements. In addition, the tie wires were only used in one-third of the reinforcement intersections, the remaining intersections were free. Thus, each vertical reinforcement of the steel cage bent individually, not as a structural frame (cage) with fixed joints. However, due to the numerous closed horizontal circular ties, all vertical reinforcements within the circular ties of the steel cage fell together as a group. The following assumptions are made for the engineering analysis:

1. The section modulus of the collapsed steel cage will be the sum of the section modulus for each individual vertical reinforcement, not the combined section modulus of the steel cage.
2. The height of the representative reinforcement will be the average height of all (442) vertical reinforcements installed prior to the incident.
3. The wind area for the representative reinforcement will be the projected area normal to the wind of the total length of the vertical reinforcements installed above the concrete footing, divided by the total number (442) of the vertical reinforcements. Please note that due to the potential shielding effects, other projected areas of all exterior and interior horizontal reinforcements (ties) are not considered.
4. The wind direction will be along the minor axis of the steel cage with a magnitude of 20 miles per hour as recorded by National Climatic Data Center.
5. The representative reinforcement under the wind load will be rigidly supported cantilever element with its lower end fixed in the concrete footing and the upper end free. This is due to the fact that the installed guy wires were intended only to maintain the vertically of the steel cage during its upward extension. These guy wires had neither the strength nor the rigidity to laterally brace the steel cage under the wind load.
6. The horizontal deflection of the steel cage under the wind load at the time of the collapse will be the same as that of the representative reinforcement.

A-2 Estimation of the Total Weight and Vertical Location of the Centroid of the Reinforcing Steel Cage at the Time of the Incident

The number, the size, the length and the location of the reinforcements in the steel cage are estimated based on the following documents:

1. Contract Drawing Sheet 10 of 13, River Crossing Column details (Figure 3-4 of the text).
2. Contract Drawing Sheet 11A of 13, River Crossing Reinforcing Steel Schedule.
3. Rebar Configuration Prior to Collapse, Pier WB-12, Column 2, Page 1B of Appendix B; and Rebar Weight Calculations, Page 2B; Prepared by RECCHI, December 1999.
4. OSHA Field Observation and Measurements.

Based on the above information, it was determined that there were 442, Number 11, vertical reinforcements installed at the time of the incident. For the lower portion of the reinforcements from the concrete footing, there were 242 outer reinforcements with a height of 30.0 ft, and 200 inner reinforcements with a height of 23.5 ft. In the upper extension of the reinforcements, 219 outer and 177 inner vertical reinforcements with a length of 20 ft were installed. The only missing part of the upper extension was the west face section as shown in Figure 3-16 of the report.

The total number of the exterior horizontal reinforcements (ties) installed at the time of the incident was field counted as 114 layers. This number was also confirmed with the actual height of the vertical reinforcements, based on the vertical spacing between horizontal ties for different elevation as given in Item 1 above.

The number, the size, the length and the location of the interior horizontal reinforcements for each layer were determined based on the Items 1 and 2 above. Please note that since the upper extension on the west face of the steel cage was not installed at the time of the incident, the associated interior reinforcements (PB1301) to be installed from this face were not considered in the weight calculation.

The total weight of the steel cage at the time of the incident is calculated and tabulated in Table A-1. In addition, the vertical location of the center of gravity (centroid) of the steel cage is also calculated in Table A-2.

16A

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*      Proprietary Program of
*      Research Engineers, Inc.
*      Date=      8: 9: 56000
*      Time=      8: 9:49
*      Build No.   1007.01.02
*      USER ID: U.S. Dept of Labor
*****

```

1. STAAD PLANE #11 REBAR ANALYSIS
2. UNIT FT KIPS
3. JOINT COORDINATES
4. 1 0 0
5. 2 0 39.2
6. MEMBER INCIDENCES
7. 1 1 2
8. UNIT INCHES
9. MEMBER PROPERTIES
10. 1 PRIS AX 1.56 IZ 0.194
11. CONSTANTS
12. E 29000. ALL
13. SUPPORT
14. 1 FIXED
15. UNIT FT POUNDS
16. LOADING 1 (DEAD WEIGHT)
17. MEMBER LOAD
18. 1 UNI GY -9.26
19. LOADING 2 (WIND LOAD)
20. MEMBER LOAD
21. 1 UNI GX 0.620
22. LOADING 3 (DEAD + WIND)
23. REPEAT LOAD
24. 1 1. 2 1.
25. PDELTA 2 ANALYSIS

P R O B L E M S T A T I S T I C S

```

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS =      2/      1/      1
ORIGINAL/FINAL BAND-WIDTH =      1/      1
TOTAL PRIMARY LOAD CASES =      3, TOTAL DEGREES OF FREEDOM =      3
SIZE OF STIFFNESS MATRIX =      9 DOUBLE PREC. WORDS
REQRD/AVAIL. DISK SPACE = 12.00/ 1509.5 MB, EXMEM = 1966.2 MB

```

```

++ Processing Element Stiffness Matrix.           8: 9:57
++ Processing Global Stiffness Matrix.            8: 9:57
++ Processing Triangular Factorization.           8: 9:57
++ Calculating Joint Displacements.                8: 9:57
++ Adjusting Displacements                         8: 9:57
++ Adjusting Displacements                         8: 9:57
++ Calculating Member Forces.                     8: 9:57

```

26. LOAD LIST ALL
27. SECTION 0.0 0.1 0.2 MEMBER 1
28. PRINT SECTION FORCES

STRUCTURE DATA

TYPE = PLANE

NJ = 2

NM = 1

NE = 0

NS = 0

NRJ = 1

NL = 3

XMAX = 0.0

YMAX = 39.2

ZMAX = 0.0

MN/ELEM

2

1

1

///

J=2,M=1

UNIT FT POU

STAAD POST - PLOT (REV: 23.0)

DATE: APR 29, 2000

TITLE: #11 REBAR ANALYSIS

17A

JOINT DISPLACEMENT (INCH RADIANS)

STRUCTURE TYPE = PLANE

JOINT	LOAD	X-TRANS	Y-TRANS	Z-TRANS	X-ROTAN	Y-ROTAN	Z-ROTAN
1	1	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
	2	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
	3	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
2	1	0.00000	-0.00189	0.00000	0.00000	0.00000	0.00000
	2	56.20695	0.00000	0.00000	0.00000	0.00000	-0.15932
	3	1596.64355	-0.00189	0.00000	0.00000	0.00000	-5.07142

SUPPORT REACTIONS -UNIT POUN FEET

STRUCTURE TYPE = PLANE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	0.00	362.99	0.00	0.00	0.00	0.00
	2	-24.30	0.00	0.00	0.00	0.00	476.36
	3	-24.30	362.99	0.00	0.00	0.00	10267.85

MEMBER END FORCES

STRUCTURE TYPE = PLANE

ALL UNITS ARE -- POUN FEET

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
1	1	1	362.99	0.00	0.00	0.00	0.00	0.00
		2	0.00	0.00	0.00	0.00	0.00	0.00
	2	1	0.00	24.30	0.00	0.00	0.00	476.36
		2	0.00	0.00	0.00	0.00	0.00	0.00
	3	1	362.99	24.30	0.00	0.00	0.00	10267.85
		2	0.00	0.00	0.00	0.00	0.00	0.00

***** END OF LATEST ANALYSIS RESULT *****

37. FINISH

***** END OF STAAD-III *****

**** DATE= MAY 1,2000 TIME= 8: 9:57 ****

* For questions on STAAD-III, contact: *

* Research Engineers, Inc at Build No. 1007.01.02 *

* West Coast: Ph- (714) 974-2500 Fax- (714) 921-2543 *

* East Coast: Ph- (978) 688-3626 Fax- (978) 685-7230 *

APPENDIX B

KEY REFERENCE MATERIALS

Unedited Surface Weather Observations (METAR/SPEC)						Latitude 37° 31'	Longitude -77° 19'	Station Elev. (ft) 165	Time Conversion	Day 16	Mon. 12	Year 1999	SID RIC	Station (Type, Name, State) ASOS RICHMOND, VA	
WIND						VISIBILITY									
TIME OF OBSERVATION (LST) TRUE P E	SPD KTS	GUST KTS	VARIABILITY TRUE	SURFACE STATUTE MILES	RUNWAY VISUAL RANGE (FT)	PRESENT WEATHER	SKY CONDITIONS		TEMP °C	DEW POINT °C	ALT METER INS.	REMARKS AND SUPPLEMENTAL CODED DATA			
1	2	3	4	5	6	7	8	9	10	11	12	13	14		
50054200	8			10			CLR			07	04	A2993	SLP139	T00720044 10089 20044 56010 (DAP)	
50154220	8			10			CLR			07	04	A2993	SLP139	T00670039 (DAP)	
50254210	7			10			CLR			06	03	A2993	SLP141	T00610033 (DAP)	
50354210	6			10			CLR			05	03	A2993	SLP141	T00500028 53002 (DAP)	
50454260	4			10			FEW055			06	04	A2995	SLP148	T00560039 (DAP)	
50554280	5			9			FEW055			05	03	A2997	SLP154	T00500033 (DAP)	
50654300	4			10			BKN060 BKN250			05	03	A2999	SLP162	T00500033 10572 20044 53020 (KB)	
50754260	3			8			FEW055 SCT250			05	01	A3002	SLP172	T00500011 (KB)	
50854250	5			10			FSW065 SCT250			07	01	A3005	SLP180	T00671011 (KB)	
50954260	12			10			FEW250			09	01	A3006	SLP186	T00891011 51024 (KB)	
51054300	14	19		10			FEW250			09	04	A3006	SLP185	T00841039 (KB)	
51154300	12			10			FEW250			10	06	A3004	SLP178	T01001056 (KB)	
51254270	13	19		10			FEW250			11	07	A3002	SLP170	T01061067 10106 20044 58014 (KB)	
51354240	14	20		10			FEW250			11	07	A3002	SLP170	SML SC W 01061067 (KB)	
51454260	16	20		10			FEW250			11	08	A3001	SLP167	T01061078 (CES)	
51554260	11			10			FEW250			11	08	A3002	SLP172	T01061078 (CES)	
51654250	10			10			FEW250			09	08	A3005	SLP181	T00941083 (CES)	
51754250	10			10			CLR			09	08	A3008	SLP193	T00891083 (CES)	
51854280	5			10			CLR			07	08	A3011	SLP202	T00721083 10106 20072 53030 (CES)	

S1954280	5			10			07 -08 A3011
S1954260	4			10		CLR	07 -08 A3013 SLP207 T00671078 (CES)
S2054260	6			10		CLR	06 -09 A3014 SLP211 T00561089 (CES)
S2154230	5			10		CLR	03 -07 A3016 SLP219 T00331072 S3016 (DAP)
S2254260	4			10		CLR	03 -08 A3016 SLP219 T00331083 (DAP)
S2354250	5			10		CLR	03 -08 A3016 SLP220 T00281078 401060306 (DAP)

58

SUMMARY OF THE DAY (MIDNIGHT TO MIDNIGHT)													
PEAK WINDS				FASTEST 2-MIN WIND				SKY COVER					
Speed (mph)	Direction	Time (lst)		Speed (mph)	Direction	Time (lst)		Sunrise to Sunset	Character	Character	Percent of PSBL	Total Sunshine (min)	Sunset Time (lst)
23	42	45		46	47	48		33	34	35	32	31	1652
26	270	1521		22	250	1521		36			N	N	
								37					
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REMARKS, NOTES AND MISCELLANEOUS PHENOMENA:													
Time Clock Check Correct to the Nearest Minute AL													