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Inspection Methods & Techniques to Determine Non Visible Corrosion of Prestressing Strands in Concrete Bridge Components

Task 3 – Forensic Evaluation and Rating Methodology

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Abstract

Catastrophic failures of non-composite prestressed precast concrete adjacent-box beam bridges have occurred in several states due to corrosion of the prestressing steel. These failures have highlighted the need to improve methods used to detect corrosion damage and subsequently load rate the damaged members. In light of this, PennDOT initiated a research program aimed at improving inspection techniques through evaluation of off-the-shelf non-destructive testing (NDT) technologies and correlation of surface conditions with non-visible strand corrosion. Funding for the project was provided by the departments of transportation of Pennsylvania (the lead agency), New York, and Illinois.

Currently, inspection of concrete box girder sections relies on visual methods which correlate longitudinal and transverse cracking, spalling, and exposed strands with the rated level of performance of the member. While the visual method provides a qualitative estimate of the amount of damage, the specific location along a strand and the amount of damage to the strands is not clearly defined. As a result, the assessment of the condition of the bridge could in some cases result in an un-conservative or overly-conservative estimate of remaining strength. Furthermore, without a high level of accuracy in locating damage to the strands, remediation and rehabilitation is difficult to accomplish. To improve on the current inspection techniques the visual inspection requirements are revisited through an extensive destructive evaluation study. In addition, NDT methods are evaluated and compared with actual damage present in a group of 40-50 year old box beams removed from service. The goal of this project is to determine if visual inspection techniques or currently available NDT technologies will allow for accurate identification of non-visible corrosion of prestressing strands.

This report presents the results of the visual inspection, material testing, half-cell potential mapping, and the destructive evaluation of the beams. The research results indicate that fabrication techniques used for box beam construction in the 1950-1960 time period allowed for large variations in construction tolerance. Half cell methods were shown to not provide an accurate or reliable method of identifying corrosion of prestressing strands. Longitudinal cracking was shown to provide an accurate and reliable means of identifying corrosion of prestressing strands. Probabilities of corrosion on strands adjacent to longitudinal cracks are determined and discussed. Additionally, a new recommendation for inspecting beams and its impact on operating and inventory rating is provided.

Table of Contents

1 Project Overview 12
1.1 Inspection Techniques 12
1.1.1 Visual Inspection 13
1.1.2 Half-Cell Potential Mapping 13
1.1.3 Destructive Evaluation 13
1.1.4 Material Testing 16
1.1.5 Total Chloride Evaluation 16
1.1.6 Petrographic Analysis 17
1.1.7 Air Void Analysis 17
1.1.8 Strength Assessment 17
2 Bridge Beam Acquisition, Geometries, and Details 18
2.1 Box Beam Acquisition 18
2.2 Clearfield Creek Bridge 22
2.2.1 Clearfield Creek Bridge Location & Layout 22
2.2.2 Clearfield Creek Bridge Span 1 Beam 3 23
2.2.3 Clearfield Creek Span 2 Beam 4 25
2.3 Lakeview Drive Bridge 25
2.3.1 Lakeview Drive Bridge Location & Layout 26
2.3.2 Lakeview Drive Span 1 Beam 7 27
2.3.3 Lakeview Drive Span 2 Beam 16 29
2.3.4 Lakeview Drive Span 3 Beam 19 30
2.4 Main Street Bridge 31
2.4.1 Main Street Bridge Location & Layout 31
2.4.2 Main Street Span 3 Beam 2 32
2.4.3 Main Street Span 3 Beam 3 34
2.5 Summary of Bridge Details 34
3 Investigation of Beam Condition 36
3.1 Forensic Evaluation of Clearfield Creek Beams 36
3.1.1 Clearfield Creek Span 1 Beam 3 Visual Observations 36
3.1.2 Clearfield Creek Span 1 Beam 3 As-Built 36
3.1.3 Clearfield Creek Span 2 Beam 4 Visual Observations 37
3.1.4 Clearfield Creek Span 2 Beam 4 As-Built 37
3.2 Forensic Evaluation of Lakeview Drive Beams 38
3.2.1 Lakeview Drive Span 1 Beam 7 Visual Observations 38
3.2.2 Lakeview Drive Span 1 Beam 7 As-Built 39
3.2.3 Lakeview Drive Span 2 Beam 16 Visual Observations 39
3.2.4 Lakeview Drive Span 2 Beam 16 As-Built 40
3.2.5 Lakeview Drive Span 3 Beam 19 Visual Observations 41
3.2.6 Lakeview Drive Span 3 Beam 19 As-Built 41
3.3 Forensic Evaluation of Main Street Beams 42
3.3.1 Main Street Span 3 Beam 2 Visual Observations 42
3.3.2 Main Street Span 3 Beam 2 As-Built 42
3.3.3 Main Street Span 3 Beam 3 Visual Observations 43

3.3.4 Main Street Span 3 Beam 3 As-Built 43
3.4 Trapped Water in Box Sections 44
3.5 Clear Cover Measurements 45
3.6 Sounding 45
3.7 PennDOT Inspection 46
3.8 Summary of As-Built Conditions 48
4 Examination of Concrete Core Samples 49
4.1 Beam Core Locations 49
4.2 TEC Core Evaluations 54
4.2.1 Core Summary 54
4.2.2 Petrographic Examinations 57
4.2.3 Chloride Analyses 67
4.3 Concrete Compressive Strength 70
4.4 Strand Extraction 71
4.5 Core Summary 73
5 Subsurface Investigation of Prestressed Strands 75
5.1 Strand Visual Inspection 75
5.2 Damage Profiles 76
5.2.1 Clearfield Creek Bridge Beam #3 76
5.2.2 Clearfield Creek Bridge Beam #4 77
5.2.3 Lakeview Drive Bridge Beam #7 77
5.2.4 Lakeview Drive Bridge Beam #16 78
5.2.5 Lakeview Drive Bridge Beam #19 78
5.2.6 Main Street Bridge Beam #2 78
5.2.7 Main Street Bridge Beam #3 79
5.3 Exposure of the 2nd Level of Strands 79
5.4 Summary of Subsurface Investigation of Prestressing Strands 83
6 Half-Cell Potential Case Study 84
6.1 Potential Mapping Procedure 84
6.2 Procedure for Wetting Concrete Surface 86
6.3 Interpretation of Results 87
6.4 Clearfield Creek Bridge Members 88
6.4.1 Clearfield Creek Span 1 Beam 3 Potential Mapping 88
6.4.2 Clearfield Creek Span 2 Beam 4 Potential Mapping 89
6.5 Lakeview Drive Bridge Members 89
6.5.1 Lakeview Drive Span 1 Beam 7 Potential Mapping 89
6.5.2 Lakeview Drive Span 2 Beam 16 Potential Mapping 89
6.5.3 Lakeview Drive Span 3 Beam 19 Potential Mapping 90
6.6 Main Street Bridge Members 90
6.6.1 Main Street Span 3 Beam 2 Potential Mapping 90
6.6.2 Main Street Span 3 Beam 3 Potential Mapping 91
7 Discussion of Results 92
7.1 Concrete Core Evaluations 92
7.1.1 Discussion of Concrete Core Strength Results 92
7.1.2 Discussion of Chloride Analyses 93

7.2 Half-Cell Profiles ... 94
7.2.1 Half-Cell and Damage Profiles ... 94
7.2.2 Statistical Analysis of Half-Cell Data ... 97
7.3 Chloride Half-Cell Correlation ... 98
7.4 Establishing Probabilities of Corrosion ... 100
7.4.1 Probability of Corrosion of 1st Level Strands under a Longitudinal Crack ... 100
7.4.2 Probability of Corrosion of 1nd Level Strands with no Longitudinal Crack ... 101
7.4.3 Probability of Corrosion of 2nd Level Strands under a Longitudinal Crack ... 101
7.4.4 Probability of Corrosion of 2nd Level Strands with no Longitudinal Crack ... 102
7.4.5 Probability of Corrosion of 1st Level Strands Adjacent to a Longitudinal Crack. 102
7.4.6 Probability of Corrosion Summary ... 103
7.5 Sounding ... 104
7.6 Relationship between 1st and 2nd Level of Strands ... 104
7.7 Effect of Strand Damage on Nominal Moment Capacity ... 105
7.7.1 Rating Recommendations ... 107
7.7.2 Rating Recommendation Example ... 108
7.7.3 Strength Reduction Based on Surface Damage ... 110
7.8 Field Application of Proposed Recommendation: ... 113
7.9 Summary of Discussion of Results ... 116
8 Comprehensive Summary and Conclusions ... 119
9 References ... 122
Appendix A ... 124
Bridge Shop Drawings and DOT Details ... 124
Appendix B ... 125
PennDOT Condition Assessment of Beam Sections ... 125
Appendix C ... 126
Recommended Pre-tensioned Concrete Box Beam Rating Procedure ... 126
Appendix D ... 127
Ash Street Bridge Rating Evaluation Example ... 127

List of Figures

Figure 1-1: Tools Used to Expose Strands 15
Figure 1-2: Exposure of Bottom Layer Strands 15
Figure 2-1: Pennsylvania DOT Districts [PennDOT] 18
Figure 2-2: Beam Procurement 22
Figure 2-3: Location Plan - Clearfield Creek Bridge (Google Maps) 23
Figure 2-4: Plan View of Clearfield Creek Bridge 23
Figure 2-5: Section A-A of Clearfield Creek Bridge 23
Figure 2-6: Plan View of Span 1 Beam 3 of Clearfield Creek Bridge 24
Figure 2-7: Span 1 Beam 3 of Clearfield Creek Bridge as per Structural Drawings 24
Figure 2-8: Section B-B of Clearfield Creek Bridge 25
Figure 2-9: Plan View of Span 2 Beam 4 of Clearfield Creek Bridge 25
Figure 2-10: Span 2 Beam 4 of Clearfield Creek Bridge as per Structural Drawings 25
Figure 2-11: Location of Lakeview Drive Bridge (Google Maps) 26
Figure 2-12: Plan View of Lakeview Drive Bridge 27
Figure 2-13: Cross-Section of the Lakeview Drive Bridge 27
Figure 2-14: Beam 7 Location in Section of Lakeview Drive 27
Figure 2-15: Plan View of Span 1 Beam 7 of Lakeview Drive Bridge 28
Figure 2-16: Span 1 Beam 7 of Lakeview Drive Bridge as per Structural Drawings 28
Figure 2-17: Location of Beam 16 in Section of Lakeview Drive Bridge 29
Figure 2-18: Plan View of Span 2 Beam 16 of Lakeview Drive Bridge 29
Figure 2-19: Span 2 Beam 16 of Lakeview Drive Bridge as per Structural Drawings 30
Figure 2-20: Location of Beam 19 in Section of Lakeview Drive Bridge 30
Figure 2-21: Plan View of Span 3 Beam 19 of Lakeview Drive Bridge 30
Figure 2-22: Span 3 Beam 19 of Lakeview Drive Bridge as per Structural Drawings 31
Figure 2-23: Location of Main Street Bridge 32
Figure 2-24: Plan View of Main Street Bridge 32
Figure 2-25: Location of Beams 2 & 3 in Section of Main Street Bridge 32
Figure 2-26: Plan View of Span 3 Beam 2 of Main Street Bridge 33
Figure 2-27: Span 3 Beam 2 of Main Street Bridge as per Structural Drawings 33
Figure 2-28: Plan View of Span 3 Beam 3 of Main Street Bridge 34
Figure 2-29: Span 3 Beam 3 of Main Street Bridge as per Structural Drawings 34

Figure 3-1: Bottom Flange Crack and Spall Condition Span 1 Beam 3 36
Figure 3-2: As-Built Section Geometry of Span 1 Beam 3 of Clearfield Creek Bridge 37
Figure 3-3: Bottom Flange Crack and Spall Condition Span 2 Beam 4 37
Figure 3-4: As-Built Section Geometry of Span 2 Beam 4 of Clearfield Creek Bridge 38
Figure 3-5: Bottom Flange Crack and Spall Condition Span 1 Beam 7 39
Figure 3-6: As-Built Section Geometry of Span 1 Beam 7 of Lakeview Drive Bridge 39
Figure 3-7: Bottom Flange Crack and Spall Condition Span 2 Beam 16 40
Figure 3-8: As-Built Section Geometry of Span 2 Beam 16 of Lakeview Drive Bridge 40
Figure 3-9: Bottom Flange Crack and Spall Condition Span 3 Beam 19 41
Figure 3-10: As-Built Section Geometry of Span 3 Beam 19 of Lakeview Drive Bridge 41
Figure 3-11: Bottom Flange Crack and Spall Condition Span 3 Beam 2 42
Figure 3-12: As-Built Section Geometry of Span 3 Beam 2 of Main Street Bridge 43
Figure 3-13: Bottom Flange Crack and Spall Condition Span 3 Beam 3 43
Figure 3-14: As-Built Section Geometry of Span 3 Beam 3 of Main Street Bridge 44
Figure 3-15: Main Street Beam 2 - Regions of delamination found by the sounding method 46
Figure 3-16: Main Street Beam 3 - Regions of delamination found by the sounding method 46
Figure 3-17: Lakeview Drive Bm.19 - Regions of delamination found by the sounding method 46
Figure 4-1: Core Locations for Beam CC3 52
Figure 4-2: Core Locations for Beam CC4 52
Figure 4-3: Core Locations for Beam LV7 52
Figure 4-4: Core Locations for Beam LV16 53
Figure 4-5: Core Locations for Beam LV19 53
Figure 4-6: Core Locations for Beam MS2 53
Figure 4-7: Core Locations for Beam MS3 54
Figure 4-8: Sample of 3 ¾ inch Core Submitted for Analysis 56
Figure 4-9: 5/8 inch Diameter Plugs Submitted for Chloride Analyses 56
Figure 4-10: Petrographic Examination of Core 3D from Beam MS3 57
Figure 4-11: Petrographic Examination of Core 10D from Beam MS2 58
Figure 4-12: Petrographic Examination of Core 14D from Beam CC3 59
Figure 4-13: Petrographic Examination of Core 17D from Beam LV19 60
Figure 4-14: Petrographic Examination of Core 23D in Beam LV16 61
Figure 4-15: Petrographic Examination of Core 27D in Beam LV7 62

Figure 4-16: Petrographic Examination of Core 34D in Beam CC4 63
Figure 4-17: Degrees of Corrosion by TEC 66
Figure 4-18: Removal of Strands from 4 inch Cores 72
Figure 5-1: Various Strand Damage Conditions 76
Figure 5-2: Legend to Identify the Different Types of Corrosive Damage 76
Figure 5-3: Damage Profile for Clearfield Creek Bridge Beam 3 77
Figure 5-4: Damage Profile for Clearfield Creek Bridge Beam 4 77
Figure 5-5: Damage Profile for Lakeview Drive Bridge Beam 7 77
Figure 5-6: Damage Profile for Lakeview Drive Bridge Beam 16 78
Figure 5-7: Damage Profile for Lakeview Drive Bridge Beam 19 78
Figure 5-8: Damage Profile for Main Street Bridge Beam 2 79
Figure 5-9: Damage Profile for Main Street Bridge Beam 3 79
Figure 5-10: Before and After Deconstruction 80
Figure 5-11: Location of 2nd Layer Punch outs on Beam CC3 81
Figure 5-12: Location of 2nd Layer Punch outs on Beam CC4 81
Figure 5-13: Location of 2nd Layer Punchouts on Beam LV7 81
Figure 5-14: Location of 2nd Layer Punchouts on Beam LV16 82
Figure 5-15: Location of 2nd Layer Punch outs on Beam LV19 82
Figure 5-16: Location of 2nd Layer Punch outs on Beam MS2 82
Figure 5-17: Location of 2nd Layer Punch outs on Beam MS3 83
Figure 6-1: Half-Cell Electrode with Multimeter 84
Figure 6-2: Exposed Strands at Sectioned Face of Beam 85
Figure 6-3: Electrical Connection between Multi-meter and Strand. 85
Figure 6-4: Burlap Soaking the Bottom of the Beam 86
Figure 6-5: Plastic Tarp Covering the Beam During the Moistening Procedure 87
Figure 6-6: Half-Cell Potential Map of Lakeview Drive Span 3 Beam 19 88
Figure 6-7: Bottom Flange Half-Cell Potential Map - Clearfield Creek Beam 3 89
Figure 6-8: Bottom Flange Half-Cell Potential Map - Clearfield Creek Beam 4 89
Figure 6-9: Bottom Flange Half-Cell Potential Map – Lakeview Drive Beam 7 89
Figure 6-10: Bottom Flange Half-Cell Potential Map -- Lakeview Drive Beam 16 90
Figure 6-11: Bottom Flange Half-Cell Potential Map – Lakeview Drive Beam 19 90
Figure 6-12: Bottom Flange Half-Cell Potential Map – Main Street Beam 2 91
Figure 6-13: Bottom Flange Half-Cell Potential Map - Main Street Beam 3 91

Figure 7-1: Chloride content relative to strand corrosion level 94
Figure 7-2: Legend of damage index and Half Cell profile.. 94
Figure 7-3: Overlay of Damage Profile on Half Cell Potential Map for Beam CC3.................... 95
Figure 7-4: Overlay of Damage Profile on Half Cell Potential Map for Beam CC4.................... 95
Figure 7-5: Overlay of Damage Profile on Half Cell Potential Map for Beam LV7.................... 95
Figure 7-6: Overlay of Damage Profile on Half Cell Potential Map for Beam LV16................. 96
Figure 7-7: Overlay of Damage Profile on Half Cell Potential Map for Beam LV19................. 96
Figure 7-8: Overlay of Damage Profile on Half Cell Potential Map for Beam MS2 96
Figure 7-9: Overlay of Damage Profile on Half Cell Potential Map for Beam MS3 97
Figure 7-10: Voltage & Chloride Values along the Length of Beam LV19................................ 99
Figure 7-11: Voltage & Chloride Values along the Length of Beam CC4.................................. 99
Figure 7-12: Correlation of Half-Cell Potential and Chloride Levels 100
Figure 7-13: Sample Damaged Section Geometry 109
Figure 7-14: Sections evaluated.. 112
Figure 7-15: Ash Street Bridge Cross-Section.. 114
Figure 7-16: Field Sketch of Flaws from Visual Inspection... 114
Figure 7-17: Strand Area Reductions by the Proposed Rating Method..................................... 115
Figure 7-18: Strand Area Reductions by the Current PennDOT Rating Method 116

List of Tables

Table 1-1: Category Code for Beam Cores Samples 14

Table 2-1: Acquired Beam Data 19

Table 2-2: In-Situ Beam Condition........ 20

Table 3-1: Clearfield Creek Bridge Beams........ 36

Table 3-2: Lakeview Drive Bridge Beams 38

Table 3-3: Main Street Bridge Beams 42

Table 3-4: Void condition........ 44

Table 3-5: Beam Clear Cover 45

Table 3-6: Superstructure Condition Rating Guidelines for Prestressed Box-Beams [PennDOT] 47

Table 3-7: PennDOT Beam Rating........ 48

Table 4-1: Core Locations on Beam Surface........ 49

Table 4-2: Plug Locations on Beam Surface 51

Table 4-3: Summary of Cores Submitted to TEC........ 55

Table 4-4: Coarse Aggregate Types 64

Table 4-5: Air Void Analyses Data Summary........ 65

Table 4-6: Summary of Carbonation Evaluation for Petrography Cores 66

Table 4-7: Seven Wire Strand Corrosion Evaluation 67

Table 4-8: Chloride Analyses for All 3 ¾ inch Cores 68

Table 4-9: Chloride Analyses for 5/8 inch Cores 70

Table 4-10: Concrete Minimum Required Compression Strength 70

Table 4-11: Strength Core Results........ 71

Table 4-12: Recorded Data for Extracted Strands 72

Table 5-1: 2nd Level Concrete Inspection Region Locations........ 80

Table 6-1: Probability of Corrosion for Regular Reinforcing Steel (per ASTM C876)........ 88

Table 7-1: Average Chlorides (% by mass of concrete) Based on Strand Damage 93

Table 7-2: Statistical Analysis of Half Cell Voltage Readings in Relation to Extent of Damage 97

Table 7-3: Probability of Corrosion for Prestressing Steels Based on Half-Cell Potential........ 98

Table 7-4: Probabilities of Corrosion of 1st Level Strands under a Longitudinal Crack........ 100

Table 7-5: Probabilities of Corrosion of 1st Level Strands with no Longitudinal Crack........ 101

Table 7-6: Probabilities of Corrosion of 2nd Level Strands under a Longitudinal Crack 101

Table 7-7: Probabilities of Corrosion of 2nd Level Strands with No Longitudinal Crack........ 102

Table 7-8: Probabilities of Corrosion of 1st Level Strands Adjacent to a Longitudinal Crack... 102
Table 7-9: Probability of Finding Corrosion of Steel Strands Under a Longitudinal Crack...... 103
Table 7-10: Probability of Finding Corrosion of Steel Strands with No Longitudinal Crack.... 103
Table 7-11: Average Wire Strength Due to Corrosion 104
Table 7-12: Relationship between 1st and 2nd Layer of Strands... 104
Table 7-13: Box Beam Flexural Capacity 105
Table 7-14: Strand Strength Reductions Based on Probabilities for Longitudinal Cracking..... 106
Table 7-15: Strand Strength Reductions Based on Probabilities w/ No Longitudinal Cracking 106
Table 7-16: Inspection Window Size for Beams Based on Strand Diameter 107
Table 7-17: Strand Area Reduction Calculations 109
Table 7-18: Reduction in Flexural Capacity (fps Formulation)... 112
Table 7-19: Reduction in Flexural Capacity (ε Compatibility Formulation) 112
Table 7-20: Comparison of Actual Box-Beam Strength and Proposed Strength Reduction...... 113
Table 7-21: Inventory Rating [Tons] 116
Table 7-22: Operating Rating [Tons]... 116

1 Project Overview

The overall objectives of the project are to:

1) Identify inspection methods, techniques and equipment to detect and evaluate corrosion that is otherwise undetectable by visual inspection methods
2) Further refine visual inspection methods so as to better correlate external observations and simple materials testing (e.g., chloride content, depth of carbonation, etc.) with the extent and severity of corrosion.

To achieve these objectives, the project was organized into three primary tasks.

In Task 1, an extensive literature search was conducted to identify the literature that describes non-destructive inspection techniques and equipment available in the US and abroad for quantifying the conditions of prestressing strands not visible during current inspection procedures. An electronic compilation/database of all literature reviewed has been developed and has been published in the following report.

- Naito, C., Warncke, J., "Inspection Methods and Techniques to Determine Non Visible Corrosion of Prestressing Strands in Concrete Bridge Components Task 1 – Literature Review," ATLSS Report No. 08-06, Sept. 2008, pp.71.

The purpose of Task 2 was to identify the most promising Non-Destructive Testing (NDT) technologies available, and then assess the accuracy of these methods in a controlled laboratory environment. A total of six vendors of NDT equipment were identified and were invited to the laboratory to demonstrate their technologies on beam mockups. The methodology and results of the second task is summarized in the following report.

- Jones, L., Naito, C., Hodgson, I., Pessiki, S., "Inspection Methods and Techniques to Determine Non Visible Corrosion of Prestressing Strands in Concrete Bridge Components Task 2 – Assessment of Candidate NDT Methods," ATLSS Report No. 09-09, June 2010.

Task 3 consisted of an assessment of the in situ conditions of the beams and development of a recommended approach for rating of box beams with corrosion damage. The results of these tasks are included in this report. The report includes (1) the inspections of the beams (including visual inspection, half-cell potential mapping, and the destructive evaluation); (2) material testing of the concrete; (3) correlation of the surface conditions with the in-situ corrosion observed; and (4) development of a rating procedure and an example application of the methodology.

1.1 Inspection Techniques

The beams were examined using three levels of inspection. The first level of inspection consisted of a surface condition evaluation of the beams. The second level of inspection consisted of potential mapping of the beams using the half-cell methodology per ASTM C876. The third level of inspection was a destructive evaluation which included beam chipping, coring, skinning, and strand exposure/removal. The following sections of the report present the details of the inspection methods employed.

1.1.1 Visual Inspection

The beams were visually examined to establish their in-situ condition. In addition, the results of the visual evaluation were used together with the results of the half-cell mapping described below to identify locations for core samples, strand exposure, and strand removal.

The visual inspection included a complete documentation of the existing condition of each beam. This included the development of crack maps which provide description of all cracks (crack length and width) as well as the extent and severity of all spalls and delaminations. This type of data is commonly recorded during standard bridge inspections. The results of the visual inspection serve as a baseline of comparison with other inspection methods that go beyond what is currently performed on standard bridge inspections. An enhancement on traditional visual inspection was conducted through detailed photography. Each beam was accurately photographed to create an exact map of the crack locations along the length and width of each beam segment. These photographs were later overlaid on the actual damage condition of the strand to assess correlation between surface and subsurface conditions.

1.1.2 Half-Cell Potential Mapping

The half-cell potential method was used to develop corrosion potential maps of each beam. This process was developed to detect corrosion of steel reinforcement within concrete structures, primarily in the marine industry. The half-cell potential system comprises an external half-cell electrode and a voltmeter used to detect the voltage differential within embedded steel reinforcement. The magnitude of the voltage differential has been found to be an indicator of corrosion potential. The process was performed in accordance with ASTM C876, and is discussed in detail in section 6.

1.1.3 Destructive Evaluation

The destructive evaluation phase of this project comprised (1) the measurement of overall section and cover dimensions; (2) removal and evaluation of core samples from the bottom flange of each beam; (3) exposure and assessment of prestressing strands; and (4) an assessment of the extent of delamination.

1.1.3.1 Cover and Section Measurement

All strand locations and the box cross-sections were measured at the cut face of each beam segment. The measurements were taken to the nearest 1/16th of an inch and plotted with respect to the strand pattern and cross-sectional dimensions specified in the original design drawings. These drawings were used to assess if variations in the sectional dimensions (e.g., cover) or other fabrication errors contributed to development of corrosion. Cover along the length of each strand was not directly measured but instead was assumed to vary linearly from one end of the beam to the other.

1.1.3.2 Beam Cores

A number of core samples were removed from each beam specimen. Three different classes of core samples were extracted for subsequent testing.

Four inch nominal diameter cores were extracted from the bottom flange of each of the seven beams. The cores were used to conduct petrographic analysis, depth of carbonation, chloride content, and air void analysis. In each beam, cores were taken at various locations to examine a range of surface conditions and measured half-cell potential levels. A category system was established based on the measured half-cell potential and the presence of cracking at the core location, as shown in Table 1-1. These cores are designated with an A, B, C, or D depending on the aforementioned criteria. In some locations, two 4 in. nominal diameter cores were removed in close proximity to one another. Where one core would be sent for forensic evaluation and the second would be destructively evaluated at the ATLSS laboratory to examine corrosion damage of the prestressing steel. This allowed for a comparison between half-cell potential readings and strand damage due to corrosion.

Table 1-1: Category Code for Beam Cores Samples

Core Category	Surface Crack?	Half-Cell Potential
A	Yes	Low
B	Yes	High
C	No	High
D	No	Low
S	No; used for compressive strength evaluation	Low

Strength cores, having a 2 in. nominal diameter, were taken from each beam; these are designated with an "S" according to the established category code. These cores were only taken in areas where a low half-cell potential reading was obtained, with the objective of determining a representative estimate of compressive strength for each beam. The cores were tested in accordance with ASTM C39.

Additionally, a series of 0.5 in. nominal diameter plugs – approximately 1 in. in length – were taken along the width of two of the beams; Clearfield Creek Beam #4 and Lakeview Drive Beam #19. These were taken to assist in correlating half cell potential, chloride level, and corrosion.

1.1.3.3 Exposure of Strands

After all core samples were removed, the bottom layer of prestressing strands was exposed through a cutting and chipping process to assess the level of corrosion present. To accomplish this task, a pneumatic chipping gun, a concrete saw, and a track to guide the saw was used as illustrated in Figure 1-1:

(a) pneumatic chipping gun (b) electric concrete saw

Figure 1-1: Tools Used to Expose Strands

With the saw, a cut was made between each strand in the bottom layer. Extreme care was taken so as not to damage the strands with the saw. A steel angle guide was used to ensure a straight cut along each beams length. Once all of the cuts along the beam were made, the chipping process could be started. The concrete cover was then removed with a pneumatic chipping tool. A visual of a beam after cutting with the saw and after chipping with the jackhammer is illustrated in Figure 1-2. Once all of the strands were exposed, photographs were taken and each strand was systematically examined to document the corrosion damage along its length. This data was then used to produce a damage profile for each beam. The results of this survey are discussed in detail in section 5.

(a) After Cutting (b) After Chipping

Figure 1-2: Exposure of Bottom Layer Strands

1.1.3.4 Delamination Assessment

Areas where the bottom flange concrete had delaminated were located using a sounding rod in accordance with ASTM D4580. After sounding the concrete, minimal amounts of delamination were found within the seven beam specimens. It should be noted however that a large delamination was found in Lakeview Drive Beam #7 through visual inspection. Utilizing the holes produced from coring – with each hole giving a view through the bottom flange – it has been verified that delaminations were not present in the bottom flange of the six other specimens at the core locations. This data is used concurrently with the visual inspection to aide in detecting corrosion of the prestressing strands. More discussion on the results of this evaluation is presented in section 3.6.

1.1.4 Material Testing

A series of material characterization tests were conducted to assess the in situ condition of the box beams.

1.1.4.1 Carbonation Evaluation

The depth of carbonation tests were conducted on cores removed from the beams. Carbonation in concrete is characterized by the infusion of carbon dioxide, which reacts with alkaline components in the cement paste; mainly $Ca(OH)_2$. This process leads to a reduction in the pH level of the pore solution to less than 9.0. The reduction of the pH value can be readily assessed by the color change of a suitable indicator. A solution of 1% phenolphthalein in 70% ethyl alcohol was used to determine the depth of carbonation. Phenolphthalein turns non-carbonated concrete red, and remains colorless in carbonated concrete. In order to measure the depth of carbonation in concrete specimens, a slice is broken off and tested with phenolphthalein. The slice must be thick enough to avoid the possibility of carbon dioxide penetration from the end surface affecting the observed measurement from the side surfaces.

The indicator method does not make it possible, however, to determine whether the reduction of pH value may have resulted from influences other than the absorption of CO_2 (e.g., SO_2, HCl or other acidic gases). The color change observed after spraying with phenolphthalein may be due to reactions observed on the newly created surfaces; therefore, proper handling of the pieces was maintained prior to testing.

1.1.5 Total Chloride Evaluation

The total acid soluble chloride profiles of the beams were examined at the various core sample locations throughout each beam. Two common methods of measuring the total chloride are used in the United States. They are as follows:

- ASTM C1152 Standard Test Method for Acid-Soluble Chloride in Mortar and Concrete
- AASHTO T-260 Sampling and Testing for Chloride Ion in Concrete and Concrete Raw Materials

The methods measure the total soluble chloride in the concrete by pulverizing a 10g sample such that it is able to pass through a fine sieve (No.20 sieve for ASTM, No.50 sieve for AASHTO). Both methods allow the use of potentiometric titration to determine the chloride content. The chloride content is measured and reported as either a percent chloride by mass of cement or by mass of concrete. The amount can also be quantified as pounds of chloride per cubic yard of concrete. The AASHTO method will be followed for this study. The total chloride testing was performed by The Erlin Company.

Alternatively, in lieu of ASTM C1152, ASTM C1218 Water-Soluble Chloride in Mortar and Concrete could have been used to calculate the chloride profiles. However, in ASTM C1152 it states that the amount of acid-soluble chloride in most hydraulic-cement systems is equal to the total amount of chloride in the system. ASTM C1218 also warns that water-soluble chloride determined at some particular time in the life of a cement system is capable of being substantially different than that at another time. This could have led to a non-representative chloride measurement. Consequently, acid-soluble chloride (ASTM C1152) was used in the calculation of chloride percent by mass of concrete.

1.1.6 Petrographic Analysis

Petrographic examinations of a limited number of cores taken from the beams were conducted. The evaluation followed the practice defined by ASTM. The test method and scope is presented below. One core sample (a "D" core) from each beam was analyzed.

- ASTM C 856- 04 Standard Practice for Petrographic Examination of Hardened Concrete (Active)

"Scope: This practice outlines procedures for the petrographic examination of samples of hardened concrete. The samples examined may be taken from concrete constructions, they may be concrete products or portions thereof, or they may be concrete or mortar specimens that have been exposed in natural environments, or to simulated service conditions, or subjected to laboratory tests. The phrase "concrete constructions" is intended to include all sorts of objects, units, or structures that have been built of hydraulic cement concrete."

The quality of the concrete used in each beam was assessed in accordance with ASTM C856 to identify if corrosion damage was related to poor concrete quality. The results of this investigation are presented in section 4.2.2.

1.1.7 Air Void Analysis

Hardened air void analyses were conducted on cores removed from the beams. The air content is used to assess if there is a correlation between entrapped and/or entrained air and the occurrence of steel corrosion. The air void spacing and quality was evaluated using ASTM procedures.

- ASTM C457 Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete

"Scope: This test method describes procedures for microscopical determinations of the air content of hardened concrete and of the specific surface, void frequency, spacing factor, and paste-air ratio of the air-void system in hardened concrete (1). Two procedures are described: Procedure A, the linear-traverse method and Procedure B, the modified point-count method."

The modified point count method was utilized for the study. The results are presented in section 4.2.2.4.

1.1.8 Strength Assessment

The strength of the concrete used in the beams was assessed by compressive strength testing of cores taken from the beams. The cores were tested in accordance with ASTM C39. A minimum of three cores were taken from each beam, so that an average strength could be calculated. The cores were extracted according to ASTM C42, "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete." The results of the examination are summarized in section 4.3.

2 Bridge Beam Acquisition, Geometries, and Details

Seven non-composite adjacent prestressed concrete box beams were acquired for this project. The beams were recovered from three decommissioned bridges in the state of Pennsylvania. A detailed summary of the bridge/beam location, the system and section geometries, and structural details is presented in this chapter.

2.1 Box Beam Acquisition

The twelve PennDOT district offices were contacted to determine if any prestressed concrete bridges were in the process of being decommissioned or replaced during the schedule of the research (2008). Of the twelve districts only District 9 and District 12 had beams coming out of service in the timeframe needed. A map of the eleven PennDOT districts is presented in Figure 2-1. A number of other districts had replacement projects scheduled for 2009 and 2010 but due to the proposed project schedule these systems could not be included in the study. As a consequence, the research program is limited to three bridges built in 1956, 1960, and 1961. It is expected that newer bridges are built with improved quality control procedures, thus the damage conditions observed in the beams examined should represent a worse-case scenario.

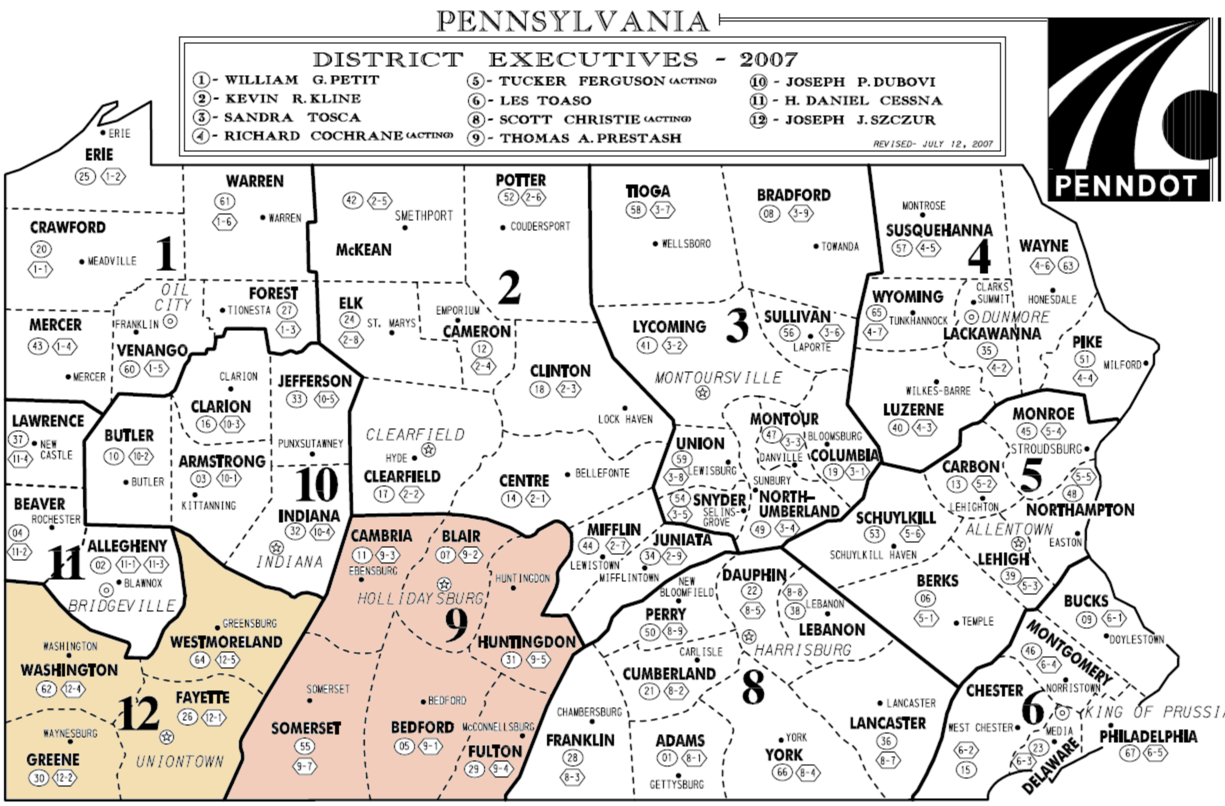


Figure 2-1: Pennsylvania DOT Districts [PennDOT]

Details of the bridges and shop drawings for each bridge type were provided by the district offices. The bridge details are included as an appendix. The beams were chosen to represent different fabricators, different ages, different details and a variety of damage conditions. A beam summary is listed in Table 2-1. The section size, length, and general condition are included.

- Bridge 1: Clearfield Creek Bridge
  Location: Flinton, Cambria County PA

Type: Three Span Prestressed Adjacent Box Beam Bridge
Feature Intersected: Clearfield Creek (One span over creek and two spans over flood plain)
Bridge ID: 11102101801351
Year Built: 1956
Beam Manufacturer: New Enterprise Stone and Lime Company

- Bridge 2: Lakeview Drive Bridge
  Location: Washington County, PA
  Type: Four Span Prestressed Adjacent Box Beam Bridge
  Feature Intersected: Interstate 70 (two Spans over traffic and two approaches)
  Bridge ID: 62101400500000
  Year Built: 1960
  Beam Manufacturer: Spancrete
- Bridge 3: Main Street Bridge
  Location: Strabane Township, Washington County, PA
  Type: Four Span Prestressed Adjacent Box Beam Bridge
  Feature Intersected: Interstate 70 (two Spans over traffic and two approaches)
  Bridge ID: 62404900301265
  Year Built: 1961
  Beam Manufacturer: Spancrete

Table 2-1: Acquired Beam Data

Bridge:	Beam:	Span:	Section Length:	Cross Section:	Condition Description:
Clearfield Creek	3	1	15ft	42x36 Box	Longitudinal cracking with rust staining.
Clearfield Creek	4	2	15ft	42x36 Box	Large longitudinal crack with spalling visible.
Lakeview Drive	7	1	15ft	48x27 Box	Heavily damaged section with spalls and cracks. The section was full of water. Examine potential for delamination
Lakeview Drive	16	2	12ft	48x42 Box	No cracking or corrosion visible on section however other areas of beam have significant corrosion.
Lakeview Drive	19	3	12ft	48x42 Box	Longitudinal crack with heavier corrosion. Hairline and larger distributed cracks Use for visual assessment.
Main Street	2	3	15ft	48x42 Box	Heavy corrosion on bottom flange without longitudinal cracking. Large patches. Determine if corrosion adjacent to patch exists using NDE methods.
Main Street	3	3	15ft	48x42 Box	Longitudinal crack with heavy splitting. Examine damage formation and NDE study.

Table 2-2 presents photographs of the soffit of each beam specimen. As previously noted the condition of these beams varies from average (i.e., minimal cracking and exterior damage) to poor (i.e., concrete spalling, large cracks, and rust staining).

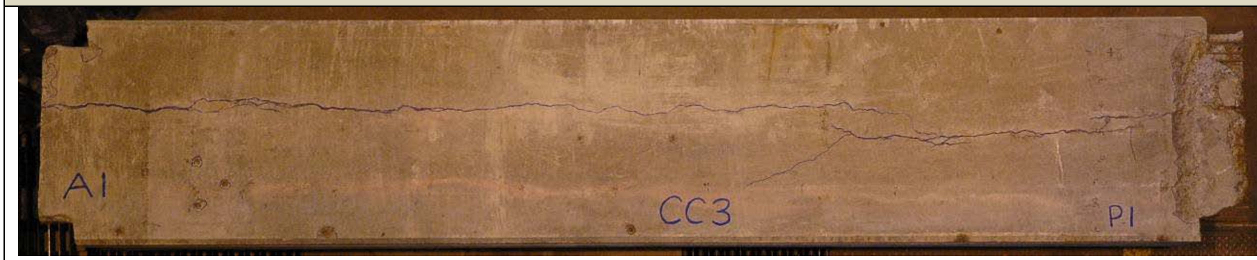
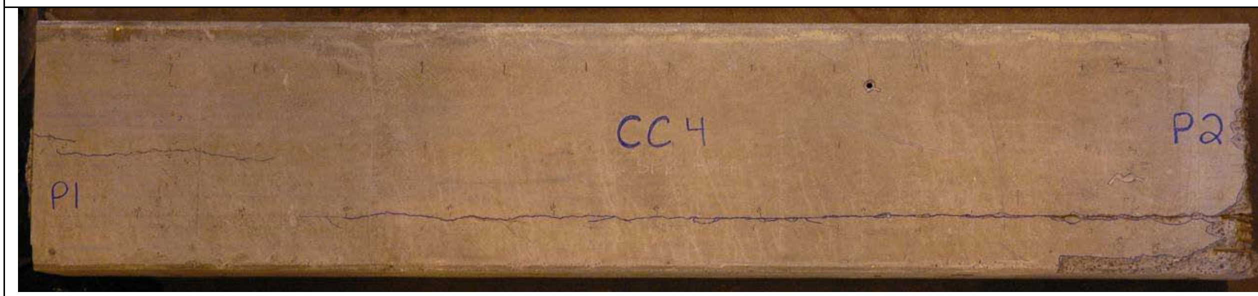
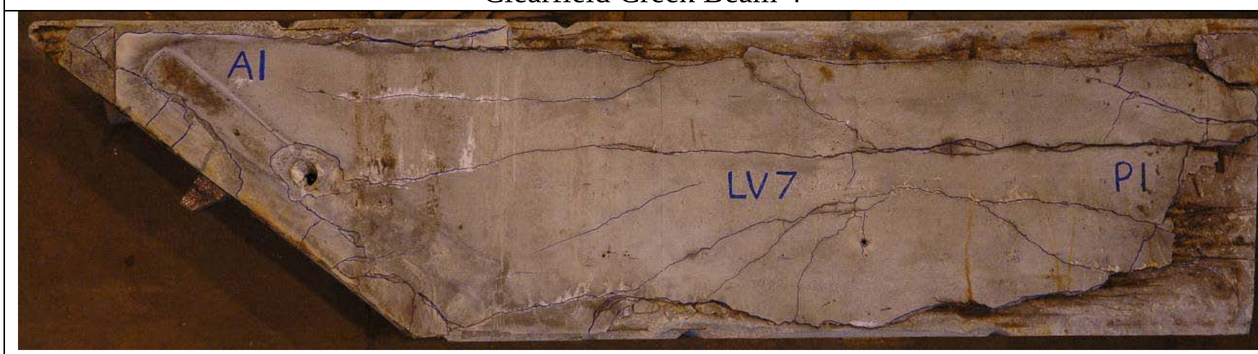
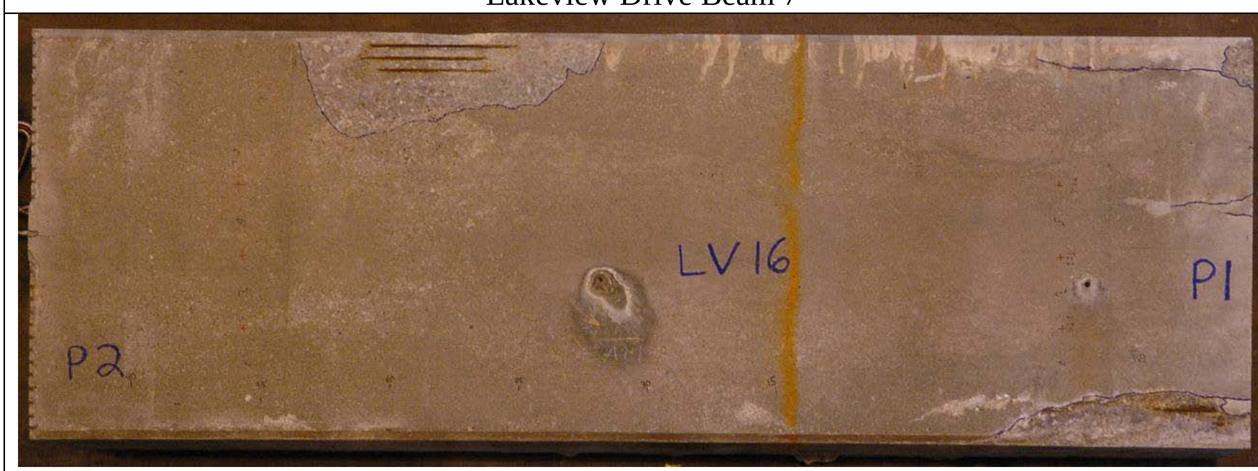
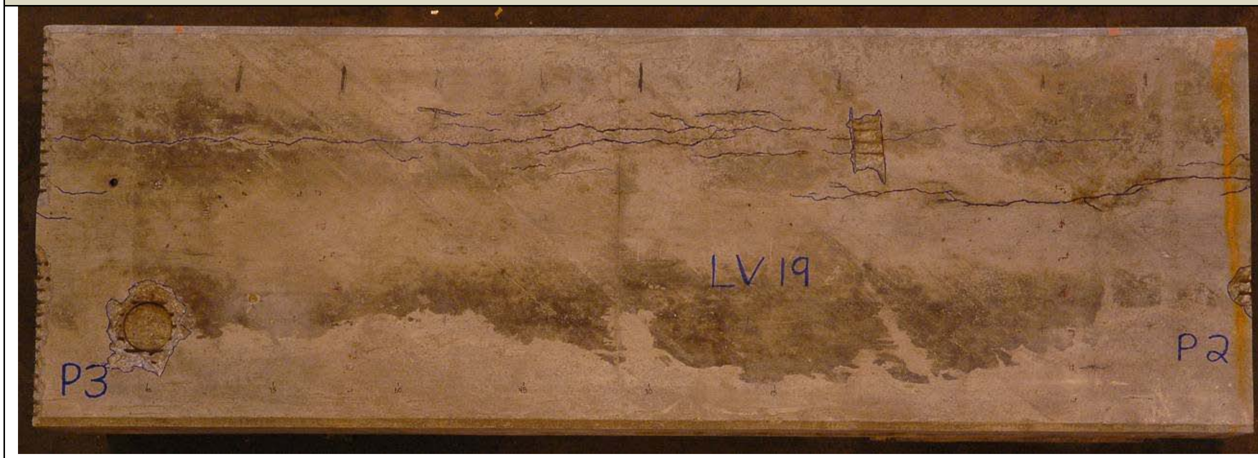
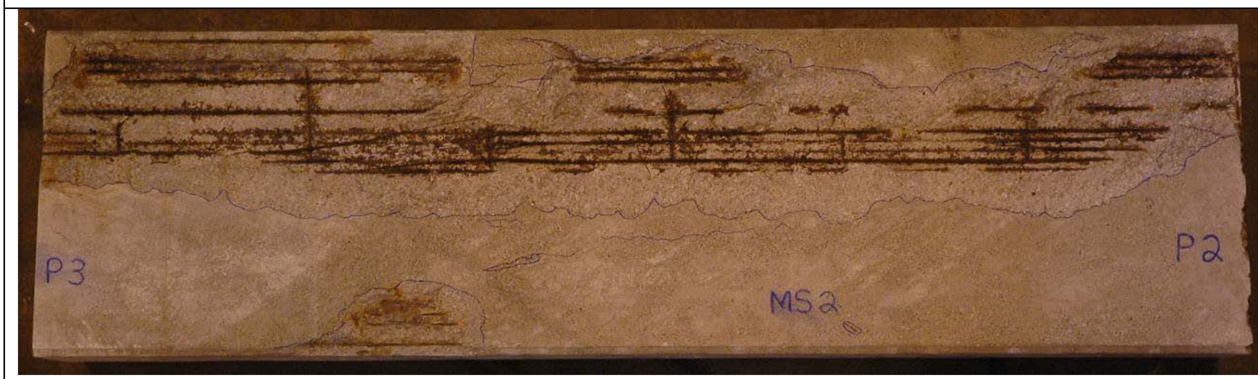
Table 2-2: In-Situ Beam Condition
 Clearfield Creek Beam 3
 Clearfield Creek Beam 4
 Lakeview Drive Beam 7
 Lakeview Drive Beam 16

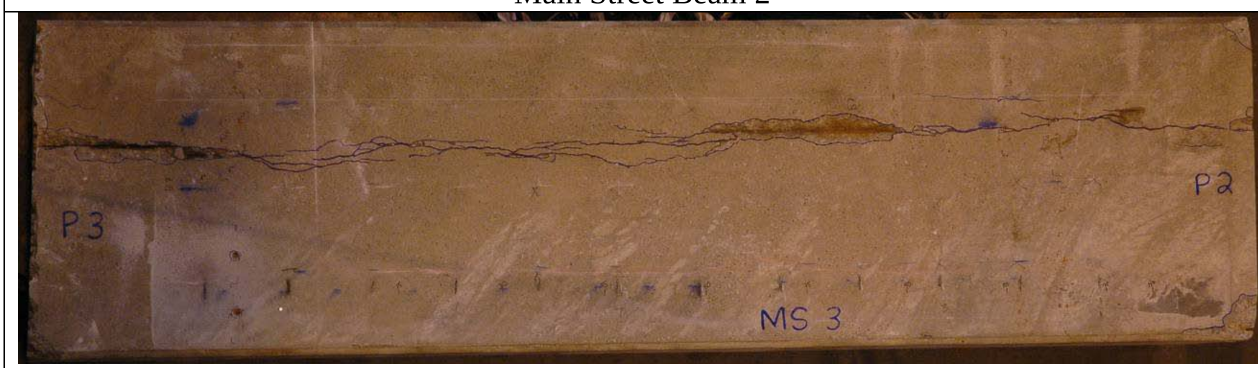
Table 2-2: In-Situ Beam Condition



Lakeview Drive Beam 19



Main Street Beam 2



Main Street Beam 3

These seven beam segments, as shown above, were acquired from both PennDOT Districts 9 and 12. The beams were staged on site and sectioned into approximately 15 ft. long segments. They were then transported to the ATLSS Center at Lehigh University. The beams were carefully handled so as to minimize further deterioration or damage to the sections. The cutting and shipping process is illustrated in Figure 2-2. A detailed description of each bridge and beam is presented in the following sections.

a) Beam Support and Staging

b) Beam Sectioning

c) Field Handling of Cut Sections

d) Trucking of Sections

Figure 2-2: Beam Procurement

2.2 Clearfield Creek Bridge

Clearfield Creek Bridge (ID: 11-1021-0180-1351) carried Bear Valley Road (State Route 1021) over the Clearfield Creek in Flinton, Cambria County, Pennsylvania. The bridge was located in PennDOT District 9. The bridge was a three-span prestressed concrete adjacent-box beam bridge with Span 1 over the creek and Spans 2 and 3 over a flood plain. There were twelve beams in the cross-section. The bridge had an asphalt wearing surface placed on top of the box girders. The beams were manufactured by New Enterprise Stone and Lime Company in 1956.

An inspection conducted in June 2006 identified damage to the superstructure. The damage included longitudinal cracks along the bottom of all beams ranging from hairline to 3/16ths of an inch in width. Lateral movement of the fascia beams was noticeable resulting in a sizeable separation between the fascia and first interior beams. Numerous instances of spalling and rust staining were observed. A vertical (flexure) crack was observed near mid-span on a fascia beam. Replacement of all 36 beams was recommended and structural monitoring was given a priority.

2.2.1 Clearfield Creek Bridge Location & Layout

The Clearfield Creek Bridge was located in Flinton, PA as illustrated in Figure 2-3.

Figure 2-3: Location Plan - Clearfield Creek Bridge (Google Maps)

Two beams from this bridge were acquired for further investigation at Lehigh University. The bridge comprises three spans of approximately 73 ft. One beam is from Span 1, situated over the creek. The other beam was located in Span 2, situated over the flood plain. Both are interior beams as identified in the plan view drawing Figure 2-4. No beams were taken for study from Span 3. The beams were inspected on-site and were chosen to have a moderate to low level of damage associated with hairline cracking and discoloration. The shop drawings indicated that all beams had a 28-day compressive strength requirement of 5000 psi.

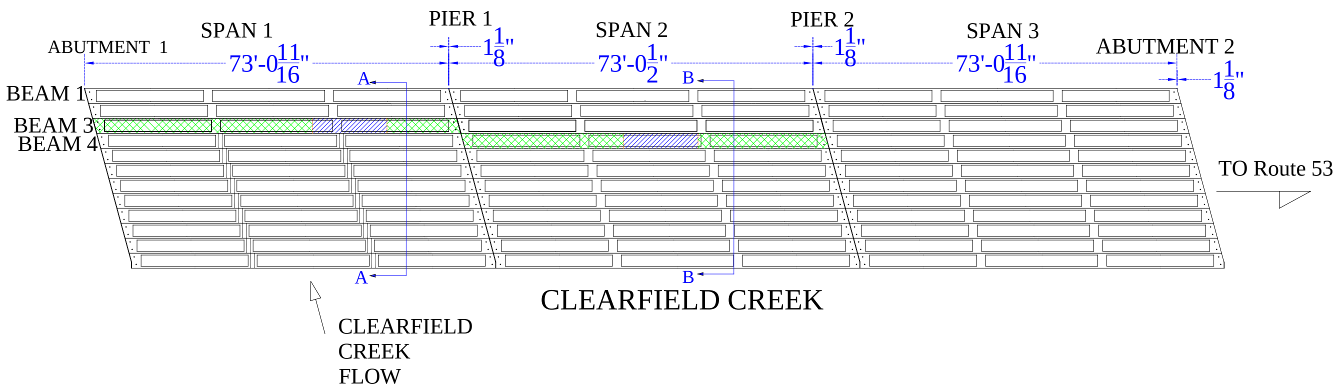


Figure 2-4: Plan View of Clearfield Creek Bridge

2.2.2 Clearfield Creek Bridge Span 1 Beam 3

As shown in Figure 2-5, each span is comprised of 12 prestressed concrete box-beam girders. A ¼ in. slope per foot starts at the center and goes towards the edge of the bridge. Beam 3 was cut and transported to the ATLSS Center at Lehigh University for further studies. It is shown in Figure 2-6 that the overall beam length is approximately 73 ft., and that the length of the cut section to be studied is approximately 15 ft. in length.

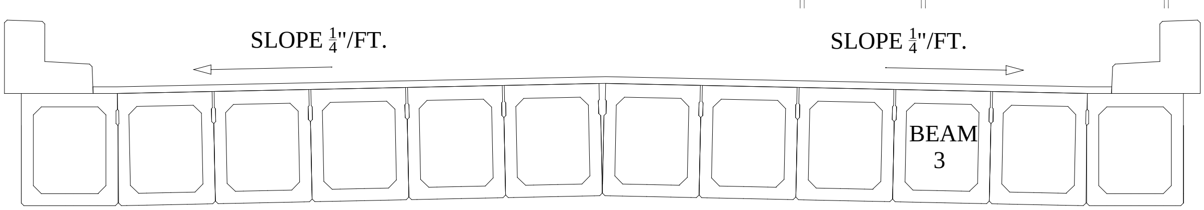


Figure 2-5: Section A-A of Clearfield Creek Bridge

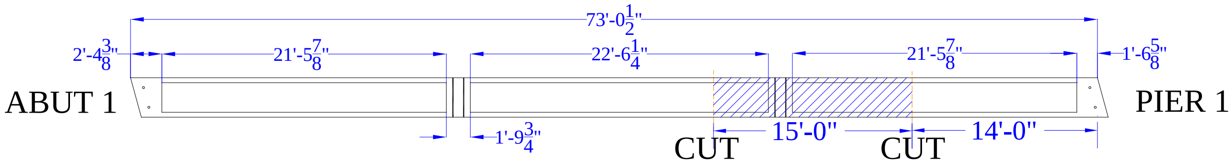


Figure 2-6: Plan View of Span 1 Beam 3 of Clearfield Creek Bridge

The structural drawings were supplied by PennDOT. This beam has a specified depth of 42 inches and a width of 36 inches. The prestressed steel reinforcement comprises thirty-seven (37) 3/8 in. diameter seven-wire strand with a minimum strand tensile strength of 250 ksi. The strands are pretensioned to an initial stress of 175 ksi with specified locations as shown in Figure 2-7. The strands have a nominal area of 0.0799 in^2 per strand. A minimum clear cover of 1 ½ inches was specified in the structural drawings. This requirement satisfies Section 1.6.16A of the 1965 Standard Specification for Highway Bridges wherein it states that 1 ½ inches should be used as clear cover for prestressing strands [AASHO 1965].

The beam is reinforced with #4 U-stirrups placed from the top of the beam. Matching U-stirrups from the bottom of the beam are not included. Furthermore, no horizontal transverse reinforcement is used in the bottom flange. The vertical legs of the stirrups are not properly developed with hooks and may not provide adequate development of the reinforcement if shear cracking were to occur. The beams were built in accordance with the design specifications of the time. Current bridge specifications have increased the requirements for U-stirrups in bridge sections as noted below:

> *AASHTO [2008] 5.11.2.6.4 Pairs of U-stirrups or ties that are placed to form a closed unit shall be considered properly anchored and spliced where length of laps are not less than 1.7 ℓ_d, where ℓ_d in this case is the development length for bars in tension.*

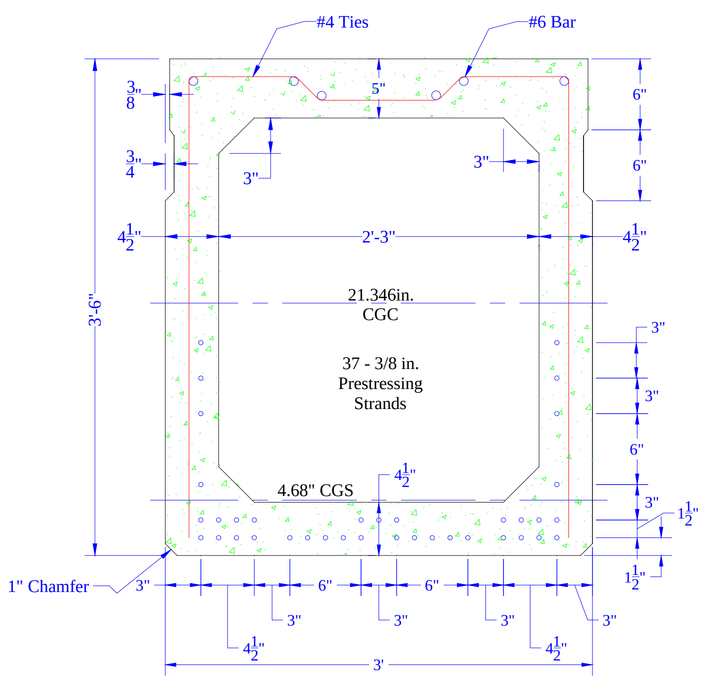


Figure 2-7: Span 1 Beam 3 of Clearfield Creek Bridge as per Structural Drawings

2.2.3 Clearfield Creek Span 2 Beam 4

As shown in Figure 2-8, the cross-section of Span 2 is identical to that of Span 1 described above. Beam 4 was cut and transported to the ATLSS Center at Lehigh University for further studies. The length of the cut section selected for this study, shown in Figure 2-9, is approximately 15 feet.

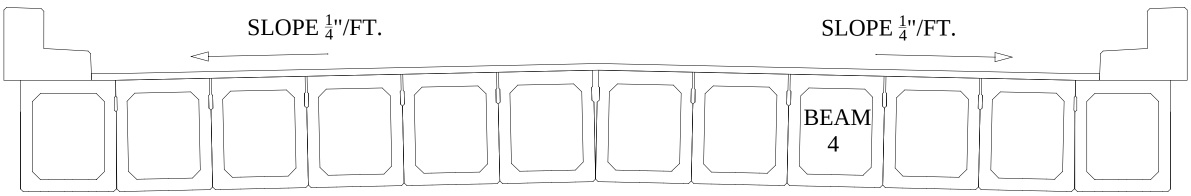


Figure 2-8: Section B-B of Clearfield Creek Bridge

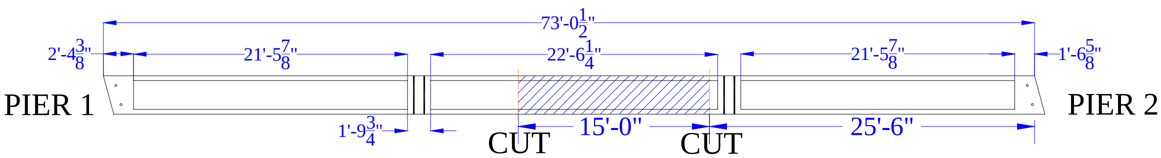


Figure 2-9: Plan View of Span 2 Beam 4 of Clearfield Creek Bridge

The details of Beam 4, shown in Figure 2-10, are identical to those of Beam 3.

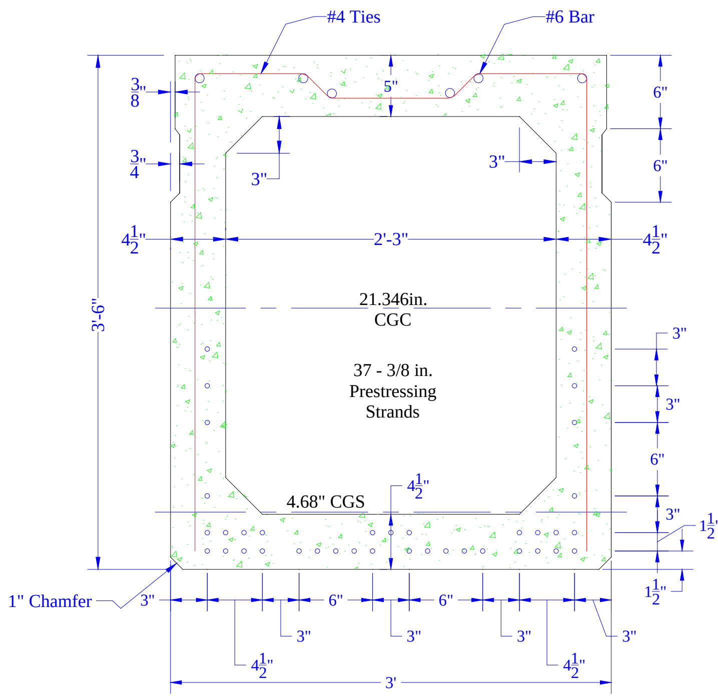


Figure 2-10: Span 2 Beam 4 of Clearfield Creek Bridge as per Structural Drawings

2.3 Lakeview Drive Bridge

The Lakeview Drive Bridge (SR-ID: 62-1014-0050-0000) carried Lakeview Drive (State Route 1014) over Interstate 70 in South Strabane Township, Washington County, Pennsylvania. The bridge was a four-span prestressed concrete adjacent box beam bridge with two approach spans and two spans over traffic. The beams were manufactured by Spancrete Dickerson Structural Concrete Corporation in 1960.

An inspection conducted in March 25 of 2004 identified heavy spalling on the bottom flanges of the beams with exposed and corroded strands. In addition, longitudinal cracks with efflorescence, scale, and heavy leaching were identified. Delaminations and significant strand damage were noted. Priority beam replacement and repair was recommended and structural monitoring was added to the bridge work plan.

On December 27, 2005 the east-side fascia beam of Span 3 failed near midspan and fell to the highway below. No impact from traffic on the highway below or overload of the bridge itself was reported. The bridge superstructure was subsequently removed and replaced with a new system. A select number of beams from the bridge were saved and stored in the field.

2.3.1 Lakeview Drive Bridge Location & Layout

This bridge was located in South Strabane Township, PA as illustrated in Figure 2-11,

Figure 2-11: Location of Lakeview Drive Bridge (Google Maps)

Three beams from the Lakeview Drive Bridge were acquired for further investigation at Lehigh University. Beams from Spans 1, 2, and 3 were chosen. The location of the beams relative to their original location on the bridge is shown in Figure 2-12. A general cross-sectional drawing of the bridge is given in Figure 2-13. Spans 2 and 3 consist of 42 inch deep beams which both have a span length of 91 feet 2 inches. Span 1 comprises 42 inch deep fascia beams and 27 inch deep interior beams, each with a length of 56 feet 3 inches. Span 4 comprises 42 inch deep fascia beams and 21 inch deep interior beams, each having a length of 44 feet 6 inches. Deeper fascia beams are used on spans 1 and 4 to provide a consistent bridge profile when viewed from the side. The required concrete compressive strength at 28-days varied based on span. Beam 7 had a required compressive strength of 5000 psi. Beams 16 and 19 had a required compressive strength of 5900 psi.

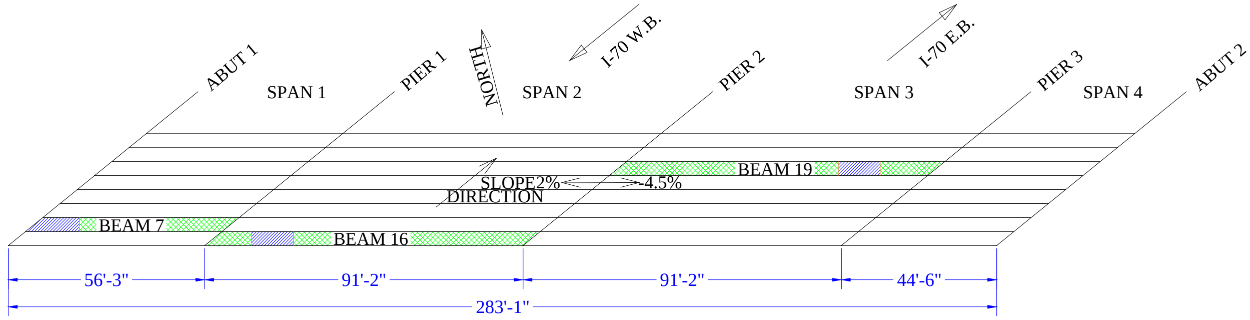


Figure 2-12: Plan View of Lakeview Drive Bridge

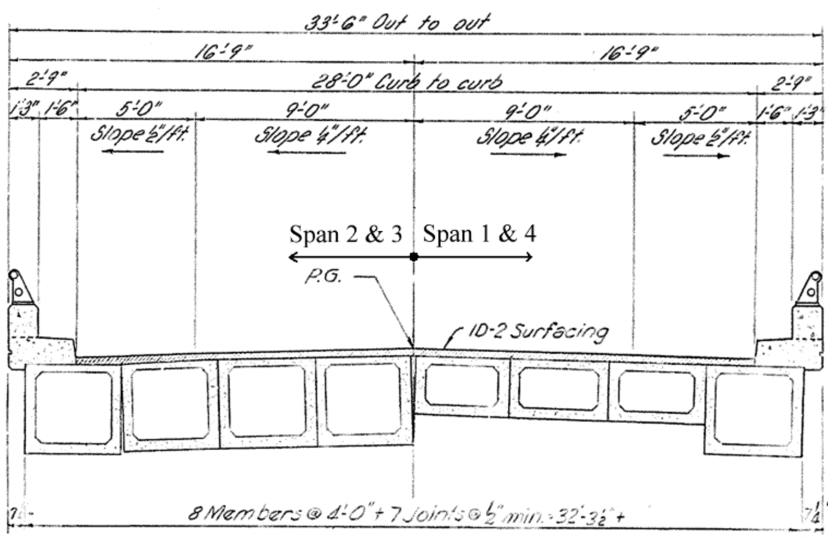


Figure 2-13: Cross-Section of the Lakeview Drive Bridge

2.3.2 Lakeview Drive Span 1 Beam 7

As shown in Figure 2-14, each span is comprised of 8 prestressed concrete box-beam girders. A ¼ inch slope per foot starts at the center and goes ¼ of the length towards the edge of the bridge. This slope then changes to ½ inch per foot for the remainder of the length of the bridge. A segment of Beam 7 was cut from Span 1 and transported to the ATLSS Center at Lehigh University for further studies. It is shown in Figure 2-15 that the overall beam length is 56 feet 3 inches, and that the length of the cut section to be studied is approximately 12 feet 10 inches in length.

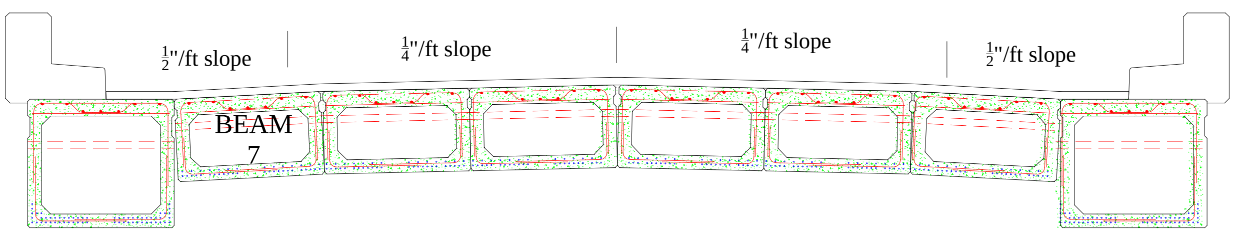


Figure 2-14: Beam 7 Location in Section of Lakeview Drive

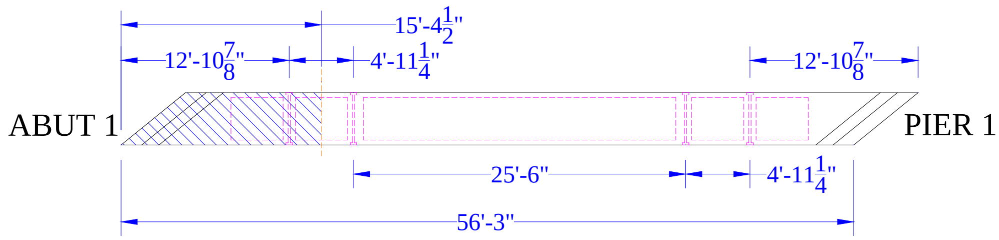


Figure 2-15: Plan View of Span 1 Beam 7 of Lakeview Drive Bridge

The structural drawings were supplied by PennDOT. The beam has a specified depth of 27 inches and a width of 48 inches. The prestressed steel reinforcement comprised thirty-eight (38) 3/8 in. diameter seven-wire strands with a minimum tensile strength of 250 ksi. The strands were pre-tensioned to an initial prestress of 14 kips per strand (175 ksi). Each strand had a nominal area of 0.0799 in^2. The specified locations of the strand are shown in Figure 2-16. The beam has groupings of two #5 hairpin stirrups at the top and two #4 L shaped bars at the bottom. The bottom shear stirrups are located between the top and bottom layers of prestressing strand; common practice during 1960-era construction. This detail is no longer common practice. Current practice [AASHTO LRFD 2008, Sect. 5.10.3.1.5] dictates that longitudinal reinforcement shall be enclosed within stirrups and ties.

The spacing of the top and bottom transverse reinforcement was not the same along the length of the beam. The top flange shear reinforcement was to be spaced at 15 inches and the bottom shear reinforcement was to be spaced at 24 inches. As a result, the top and bottom stirrups are typically not in contact in the web. Since the stirrups are not adequately lapped the shear reinforcement may not be fully developed. Clear cover is called out as 1 ¾ inches in the structural drawings. This satisfies Section 1.6.16A of the 1965 Standard Specification for Highway Bridges wherein it states that 1 ½ inches should be used as clear cover for prestressing strands [AASHTO 1965].

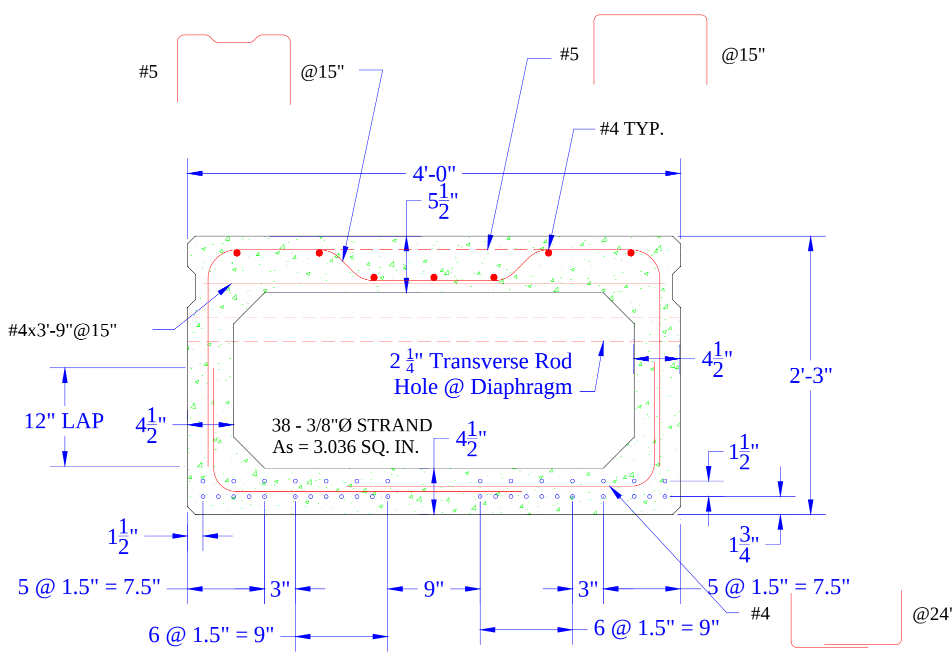


Figure 2-16: Span 1 Beam 7 of Lakeview Drive Bridge as per Structural Drawings

2.3.3 Lakeview Drive Span 2 Beam 16

As shown in Figure 2-17, this span is also comprised of eight prestressed concrete adjacent-box girders; each of the same dimensions. A ¼ inch slope per foot starts at the center and goes ¼ of the length towards the edge of the bridge. This slope then changes to ½ inch per foot for the remainder of the length of the bridge. A segment of Beam 16 was cut and transported to the ATLSS Center at Lehigh University for further studies. It is shown in Figure 2-18 that the overall beam length is 91 feet 2 inches, and that the length of the beam segment to be studied is approximately 12 feet in length.

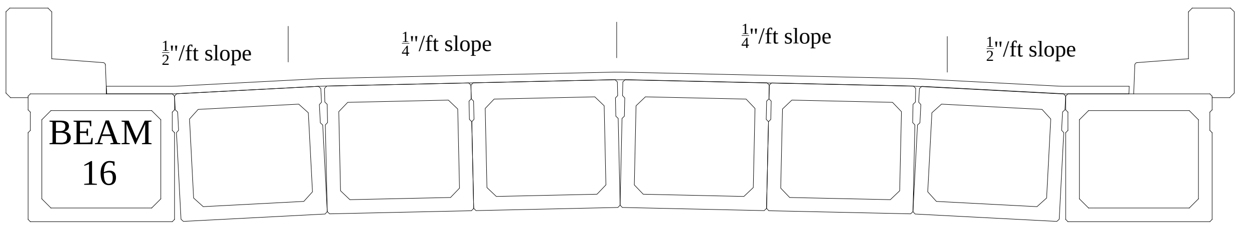


Figure 2-17: Location of Beam 16 in Section of Lakeview Drive Bridge

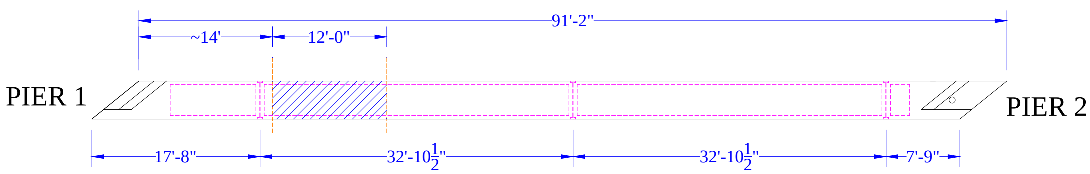


Figure 2-18: Plan View of Span 2 Beam 16 of Lakeview Drive Bridge

The structural drawings were supplied by PennDOT. The beam has a specified depth of 40 inches and a width of 48 inches. The prestressed steel reinforcement comprises sixty (60) 3/8 in. diameter wound seven-wire strand with a minimum tensile strength of 250 ksi with specified locations as shown in Figure 2-19. The clear cover and stirrup spacing/placement are identical to those found in Beam 7.

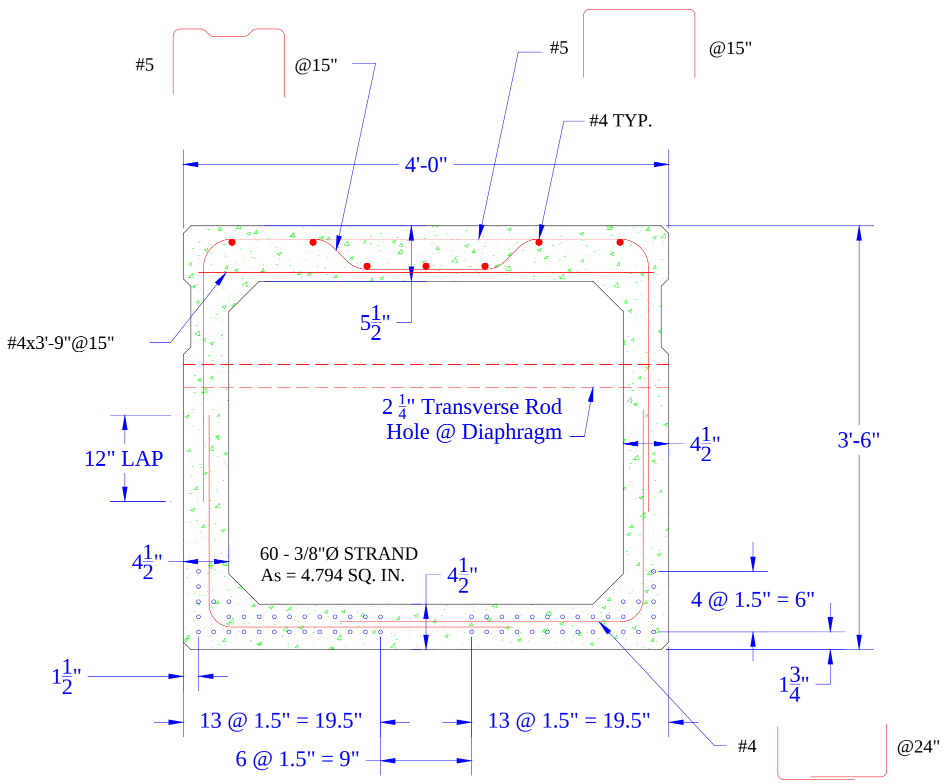


Figure 2-19: Span 2 Beam 16 of Lakeview Drive Bridge as per Structural Drawings

2.3.4 Lakeview Drive Span 3 Beam 19

As shown in Figure 2-20, the cross-section of this Span is identical to that of Span 2. A segment of Beam 19 was cut and transported to the ATLSS Center at Lehigh University for further studies. It is shown in Figure 2-21 that the overall beam length was approximately 91 feet 2 inches, and that the length of the cut section to be studied is approximately 12 feet in length.

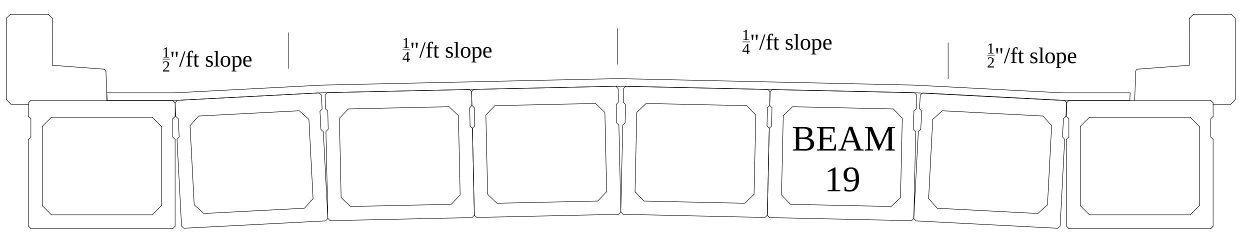


Figure 2-20: Location of Beam 19 in Section of Lakeview Drive Bridge

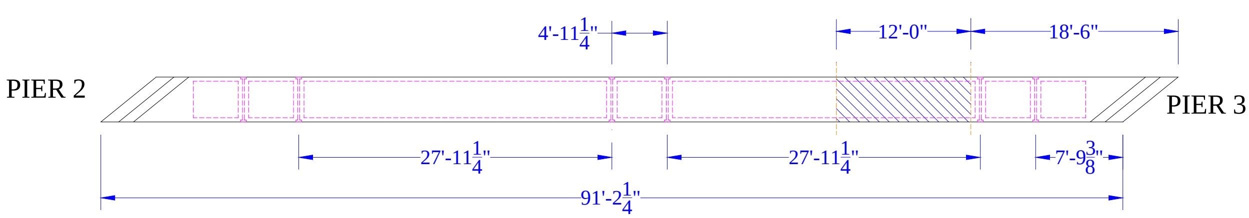


Figure 2-21: Plan View of Span 3 Beam 19 of Lakeview Drive Bridge

The details of Beam 19, shown in Figure 2-22, are identical to those of Beam 16.

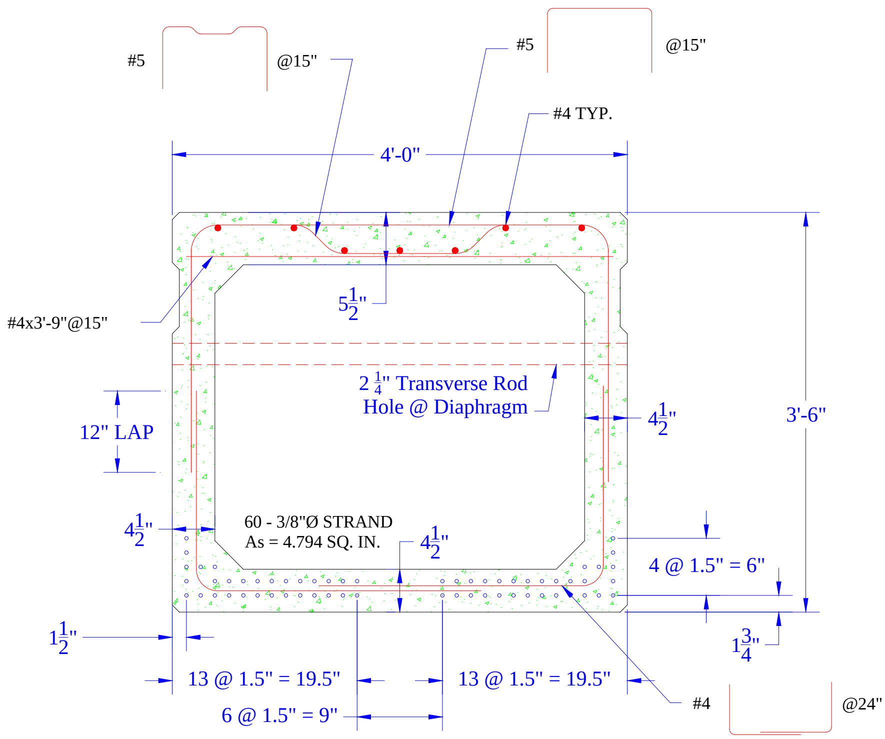


Figure 2-22: Span 3 Beam 19 of Lakeview Drive Bridge as per Structural Drawings

2.4 Main Street Bridge

Main Street Bridge (ID: 62-4049-0030-1265) carried Main Street (State Route 4049) over Interstate 70 in South Strabane Township, Washington County, Pennsylvania. The bridge consisted of a four-span prestressed concrete adjacent-box beam superstructure with two approach spans and two spans over traffic. The beams were manufactured by Spancrete Dickerson Structural Concrete Corporation in 1961.

An inspection conducted in January of 2006 identified heavy spalling on the bottom flanges of the beams with exposed and corroded strands. In addition longitudinal cracks with efflorescence, scale and heavy leaching were identified. All beams were removed and the superstructure was replaced.

2.4.1 Main Street Bridge Location & Layout

This bridge was located in South Strabane Township, PA as illustrated in Figure 2-23.

Figure 2-23: Location of Main Street Bridge

Beams from the Main Street Bridge were acquired for further investigation at Lehigh University. Two beams from Span 3 were chosen. The location of the beams relative to their original location on the bridge is shown in Figure 2-24. The bridge cross-sections are shown below. The required concrete compressive strength at 28-days varied. For span 4 the compressive strength of 5600 psi was required. For all other beams a compressive strength of 5440 psi was required.

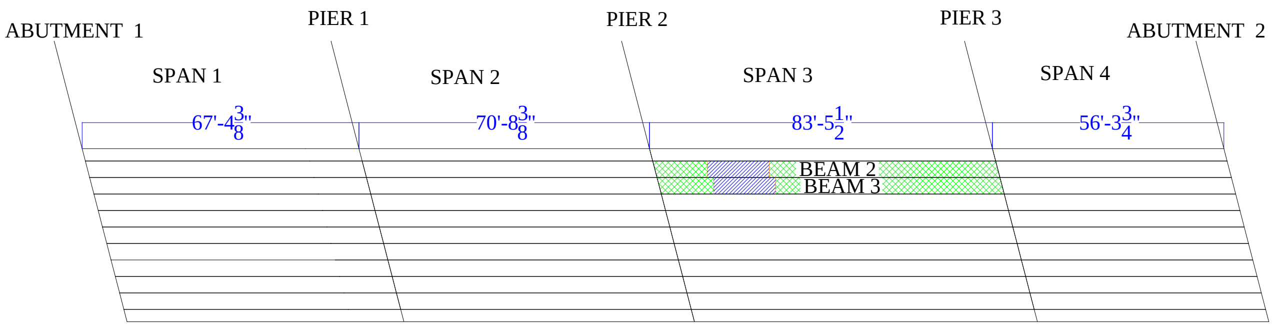


Figure 2-24: Plan View of Main Street Bridge

2.4.2 Main Street Span 3 Beam 2

As shown in Figure 2-25, Span 3 is comprised of eleven prestressed concrete box-beam girders, with the fascia girders being of different width (but the same depth) than the interior girders. A ½ inch slope per foot starts at one side of the roadway and goes towards the other edge of the bridge. An increased cross slope of ½ inch per foot was detailed over Beams 2 and 3. Beam 2 was cut and transported to the ATLSS Center at Lehigh University for further studies. It is shown in Figure 2-26 that the overall beam length was 83 feet 5 ½ inches, and that the length of the cut section to be studied is approximately 15 feet in length.

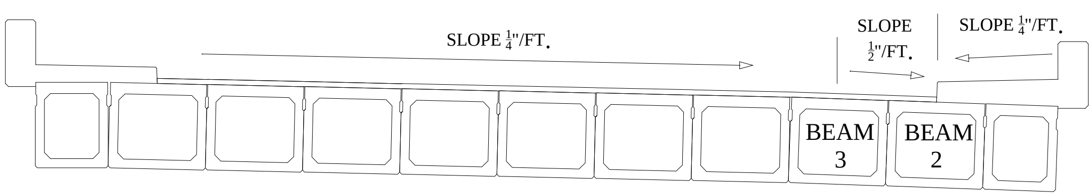


Figure 2-25: Location of Beams 2 & 3 in Section of Main Street Bridge

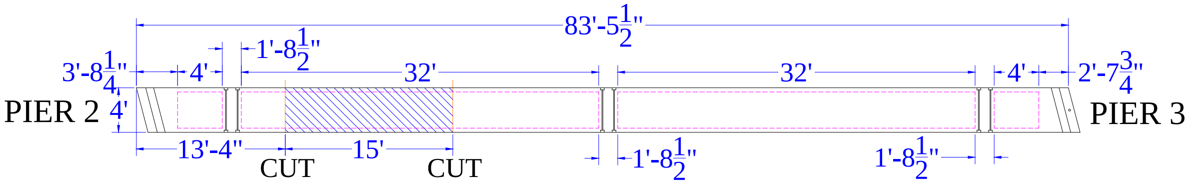


Figure 2-26: Plan View of Span 3 Beam 2 of Main Street Bridge

The structural drawings were supplied by PennDOT. The beam has a specified depth of 42 inches and a width of 48 inches The prestressed steel reinforcement comprises fifty-four (54) 3/8 in. diameter seven-wire strands with a minimum tensile strength of 250 ksi. The strands were pre-tensioned to an initial prestress of 14 kips per strand (175 ksi). Each strand had a nominal area of 0.0799 in^2. The strand locations are illustrated in Figure 2-27. Stirrup placement consists of #4 hairpin stirrups up top and #4 U-bars on the bottom. The bottom shear stirrups are located between the top and bottom layers of prestressing strand; common practice during 1960-era construction. This detail is no longer common practice. Current practice dictates that the bottom layers of stirrups are placed below the lowest level of strands. Clear cover is called out as 1 ¾ in. as per structural drawings; this satisfies Section 1.6.16A of the 1965 Standard Specification for Highway Bridges wherein it states that 1 ½ in. should be used as clear cover for prestressing strands [AASHTO 1965].

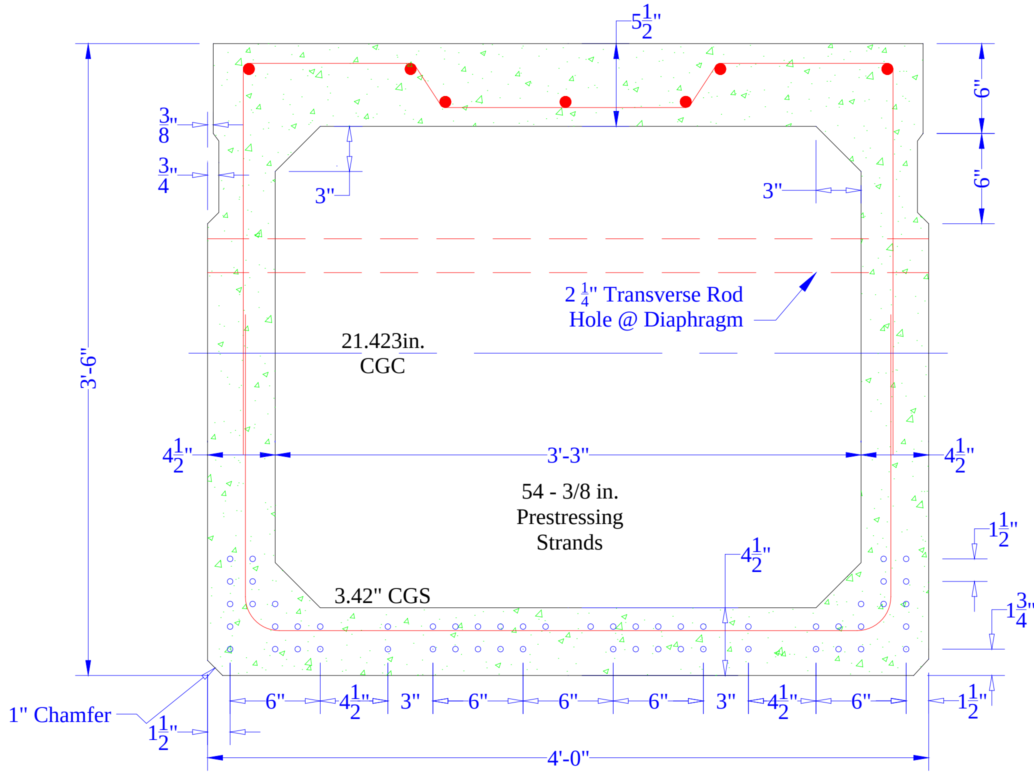


Figure 2-27: Span 3 Beam 2 of Main Street Bridge as per Structural Drawings

2.4.3 Main Street Span 3 Beam 3

As shown in Figure 2-25, Beam 3 was also taken from Span 3. Beam 3 was cut and transported to the ATLSS Center at Lehigh University for further studies. It is shown in Figure 2-28 that the length of the beam segment to be studied is approximately 15 feet in length.

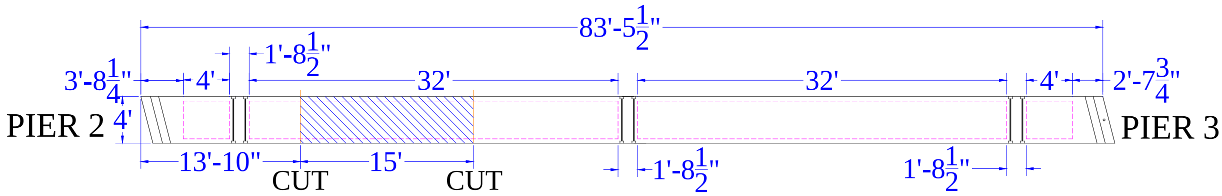


Figure 2-28: Plan View of Span 3 Beam 3 of Main Street Bridge

The details of Beam 3, shown in Figure 2-29, are identical to those of Beam 2.

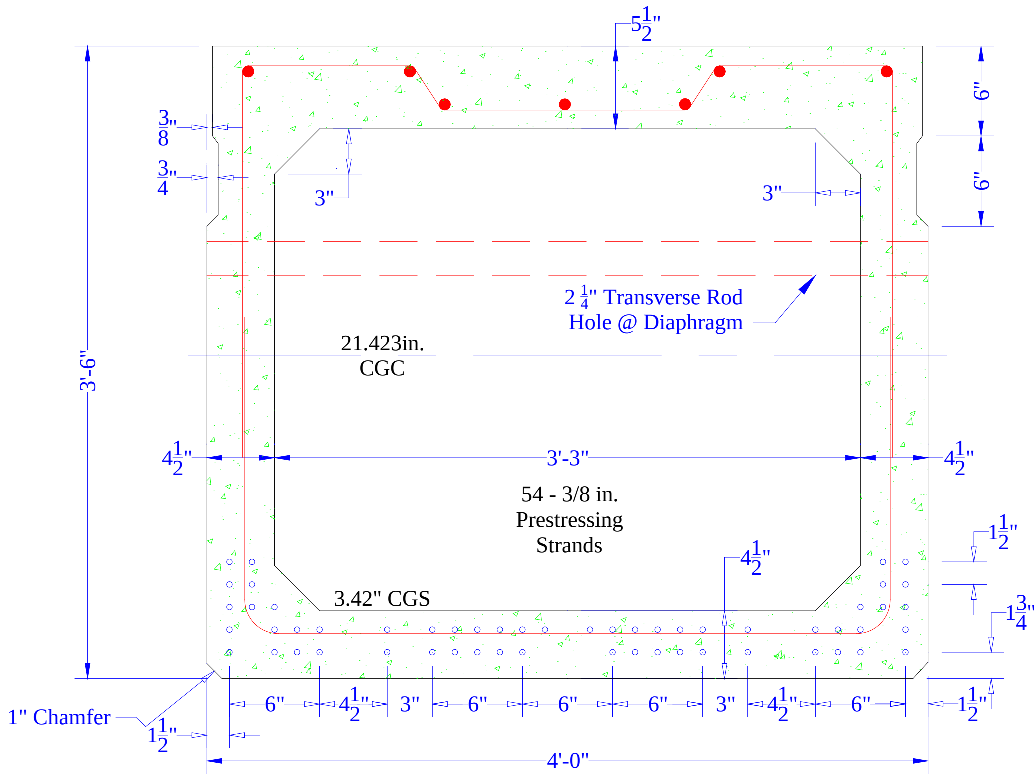


Figure 2-29: Span 3 Beam 3 of Main Street Bridge as per Structural Drawings

2.5 Summary of Bridge Details

The following is a highlight of the above section on bridge geometries and details:

- Seven beam sections were procured from three decommissioned bridges and transported to the ATLSS Center at Lehigh University for further studies; these beams had varying degrees of wear.
- All beams were recovered from non-composite adjacent prestressed box beam bridges. The depth, length, width, concrete, and reinforcement properties varied between beams.

- Prestressing reinforcement consists of 3/8 in. diameter seven wire bonded prestressing strand. All strands were pre-tensioned to 70% of ultimate. A minimum strand ultimate strength of 250 ksi was specified.
- According to the structural design drawings, all bridges are supposed to have a minimum clear cover of 1 ½ inches or greater in accordance with the prevailing AASHO cover requirement.
- The beams all utilized U-stirrups and/or L-stirrups. In all cases the stirrups were installed in accordance with the prevailing design code. The stirrup arrangement used does not meet current [AASHTO 2008] requirements. Either the stirrups are not hooked appropriately, lapped adequately or are not aligned with the bottom U-stirrup to form a closed stirrup. While this may have met the prevailing design code [1965 AASHO], the detailing used would not meet current specifications for bridge beams.
- The stirrups in Lakeview Drive and Main Street Bridges were placed between the layers of reinforcement in the bottom flange of the box beam; this was common practice at the time. However, current practice is to place stirrups outside all layers of longitudinal reinforcement.

3 Investigation of Beam Condition

An in-depth forensic evaluation of each beam section was performed. A visual inspection performed on each specimen documented any staining, spalls, cracks, etc. located along the bottom flange. At the cut ends of each beam, dimensions and clear cover were measured to evaluate if these sections were manufactured in accordance with structural drawings. The condition of the void on each beam was inspected to determine if there was any water trapped inside. Finally, Specialty Engineering, Inc. (SEI) conducted a thorough visual inspection as per PennDOT guidelines and rated each structure. The aforementioned data is presented more thoroughly below, grouped by bridge.

3.1 Forensic Evaluation of Clearfield Creek Beams

The beams were cut in the field to produce manageable sections. The section size and description is summarized in Table 3-1.

Table 3-1: Clearfield Creek Bridge Beams

Beam:	Span:	Section Length:	Cross Section:	Condition Description:
3	1	15ft	42x36 Box	Longitudinal cracking with rust staining.
4	2	15ft	42x36 Box	Large longitudinal crack with spalling visible.

3.1.1 Clearfield Creek Span 1 Beam 3 Visual Observations

The visual inspection for this beam, Figure 3-1, revealed the presence of long longitudinal cracks along the center of the beam. Approximately 2% of the concrete surface was spalled off. The crack is at its widest towards the Abutment (A1) end of the beam. The cracks along the beam varied from hairline to 0.08 in. wide. Light rust staining and mild efflorescence was present.

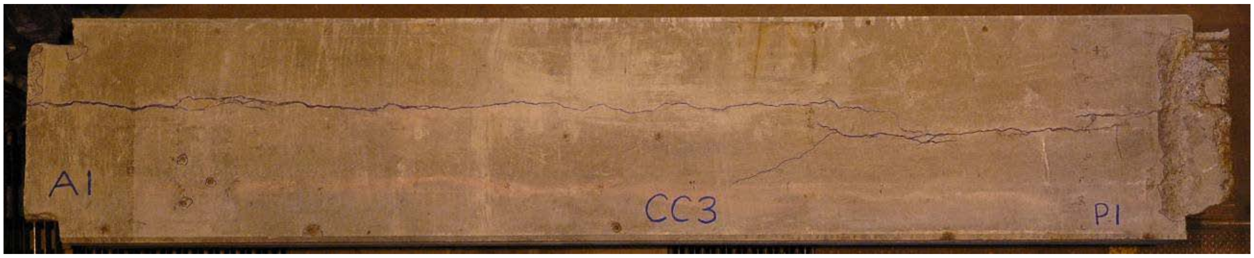
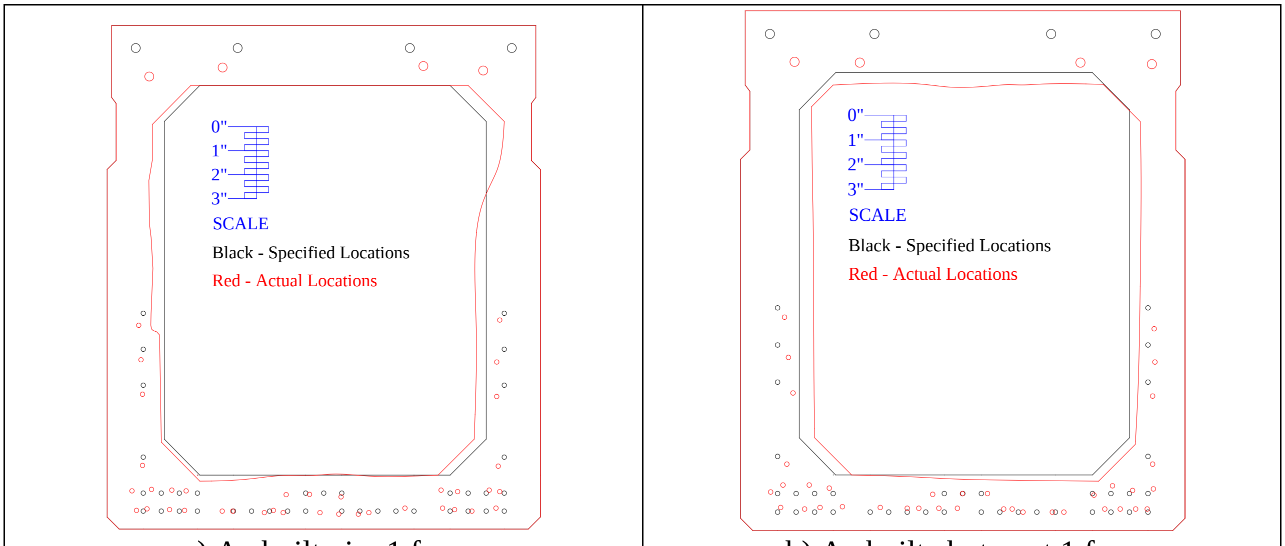


Figure 3-1: Bottom Flange Crack and Spall Condition Span 1 Beam 3

3.1.2 Clearfield Creek Span 1 Beam 3 As-Built

The as-built drawings in Figure 3-2 indicate that the cardboard forms shifted during the concrete placement. As a result, it appears that the thickness of the box girder webs and flanges change along the length; this can lead to detrimental behavior of the section. A reduced web thickness will reduce the shear strength of the member, whereas a reduced bottom flange thickness will allow the top layer of strands to corrode more easily due to a resulting decrease in the concrete cover (when water is present in the box void). It is also noted that reinforcing steel placement deviates from the specified locations. The bottom strand locations varied primarily along the horizontal axis, thus cover on the bottom strands was not significantly reduced. The average clear cover to the bottom layer of strands was 1.45 in.



a) As-built pier 1 face b) As-built abutment 1 face

Figure 3-2: As-Built Section Geometry of Span 1 Beam 3 of Clearfield Creek Bridge

3.1.3 Clearfield Creek Span 2 Beam 4 Visual Observations

The visual inspection for this beam, Figure 3-3, reveals a longitudinal crack starting from the P2 end and running nearly the full length of the beam segment. Approximately 3% of the concrete surface was spalled off. The cracks along the beam varied from hairline to 0.05 in. in width with the majority of cracks less than 0.003 in. wide. Minimal rust staining was present.

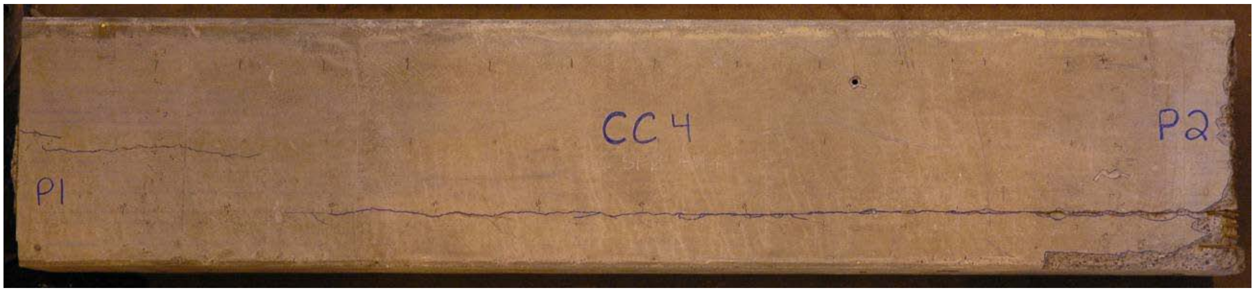
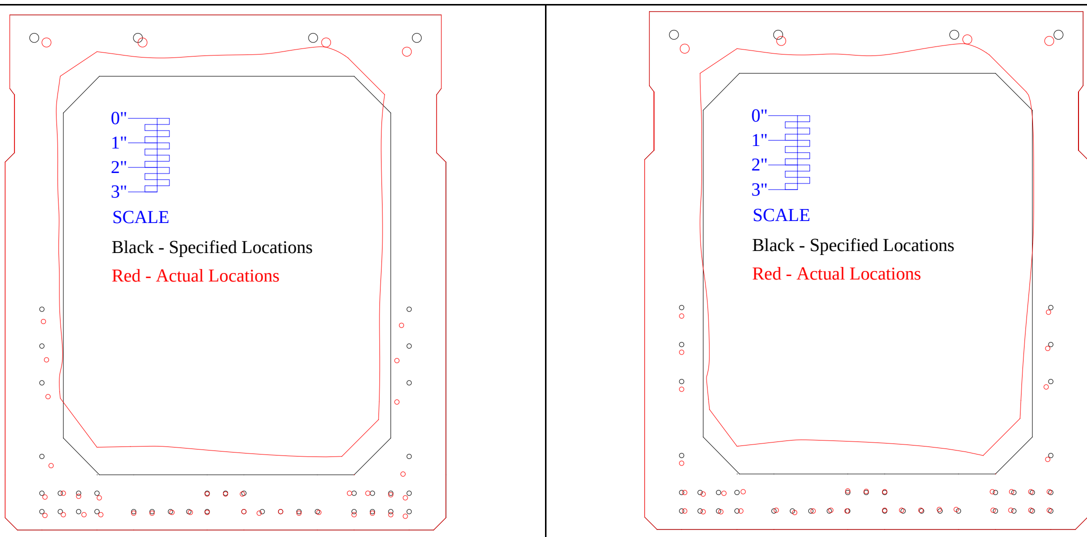


Figure 3-3: Bottom Flange Crack and Spall Condition Span 2 Beam 4

3.1.4 Clearfield Creek Span 2 Beam 4 As-Built

The as-built drawings in Figure 3-4 indicate that the cardboard forms shifted during the concrete placement. As a result, the thickness of the box-girders webs and flanges change along the length. A reduced web thickness will reduce the shear strength of the member, whereas a reduced bottom flange thickness will allow the top layer of strands to corrode more easily. It is also noted that reinforcing steel placement exhibited only minor deviations from the specified locations, most noticeably in the prestressing strands within the webs of the box-girders. The bottom strand locations varied primarily along the horizontal axis, thus cover on the bottom strands was not significantly reduced. The average clear cover to the bottom layer of strands was 1.25 in.



a) As-built pier 2 face b) As-built pier 1 face

Figure 3-4: As-Built Section Geometry of Span 2 Beam 4 of Clearfield Creek Bridge

3.2 Forensic Evaluation of Lakeview Drive Beams

The beams were chosen to represent a range of conditions from good to poor. The visible damage ranges from no visible damage on beam 16 to heavy delamination and corrosion on Beam 7. A detailed summary of each beam is presented in Table 3-2.

Table 3-2: Lakeview Drive Bridge Beams

Beam:	Span:	Section Length:	Cross Section:	Condition Description:
7	1	15ft	48x27 Box	Heavily damaged section with spalls and cracks. The section was full of water. Examine potential for delamination
16	2	12ft	48x42 Box	No cracking or corrosion visible on section however other areas of beam have significant corrosion.
19	3	12ft	48x42 Box	Longitudinal crack with heavier corrosion. Hairline and larger distributed cracks. Use for visual assessment.

3.2.1 Lakeview Drive Span 1 Beam 7 Visual Observations

The visual inspection for this beam, Figure 3-5, indicated numerous cracks along the member’s length. The cracks along the beam varied from hairline to greater than 0.06 in. wide. Severe concrete spalling occurred along the edges of the member, with approximately 23% of the overall concrete surface being spalled off. The strands are completely exposed and missing in spots. A large delamination was detected in this beam. Overall, the beam was in very poor condition.

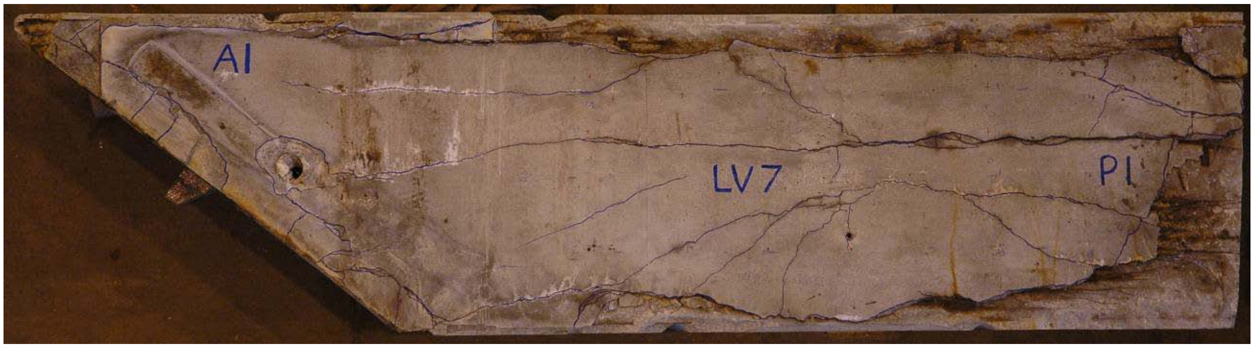


Figure 3-5: Bottom Flange Crack and Spall Condition Span 1 Beam 7

3.2.2 Lakeview Drive Span 1 Beam 7 As-Built

The as-built drawings in Figure 3-6 indicate that the cardboard forms shifted during the concrete placement. As a result, it appears that the thickness of the box-girders webs and flanges change along the length of the beam segment. The bottom flange thickness was increased. It is also noted that reinforcing steel placement deviates from the specified locations. Fifteen of the 38 strands were missing at the cut section due to corrosion and spalling. The remaining strands located in the bottom layer of the section varied laterally and vertically in the section. The strands remaining had reduced cover.

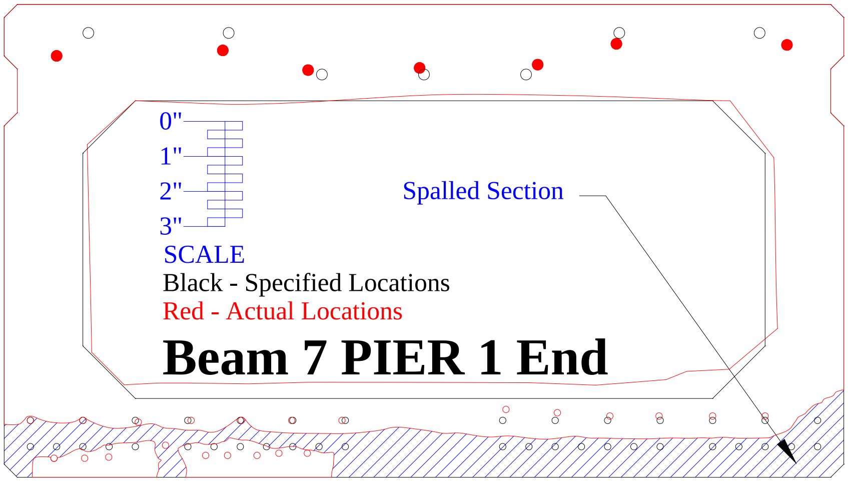


Figure 3-6: As-Built Section Geometry of Span 1 Beam 7 of Lakeview Drive Bridge

3.2.3 Lakeview Drive Span 2 Beam 16 Visual Observations

The visual inspection for this beam, Figure 3-7, shows no noteworthy cracks along the bottom flange of the member. The cracks were limited to hairline widths. However, there are three distinct areas where concrete has spalled off of the beam, displaying the corroded prestressing steel underneath; this totaled to approximately 7% of the overall concrete surface. Some rust staining and mild efflorescence is also noted.

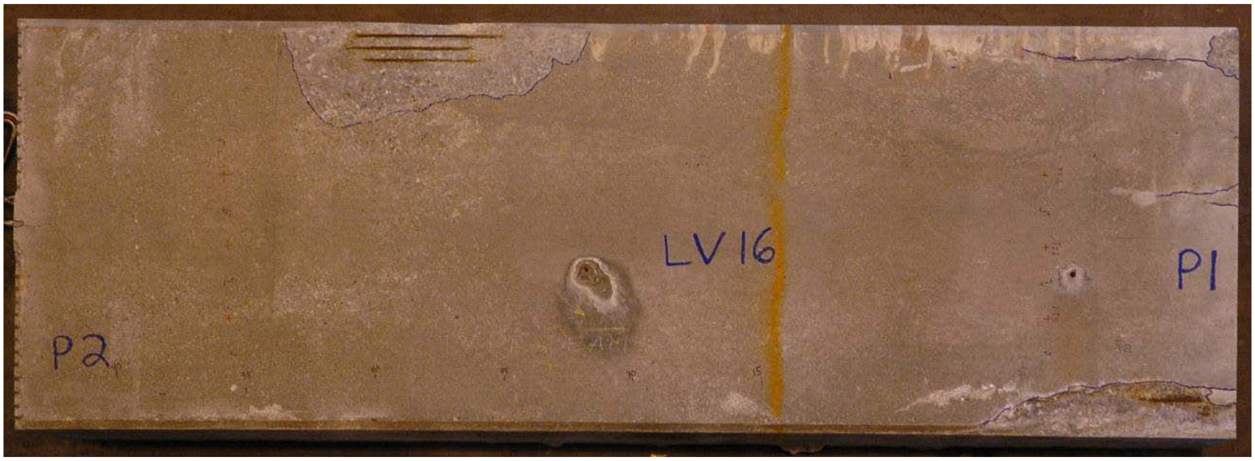
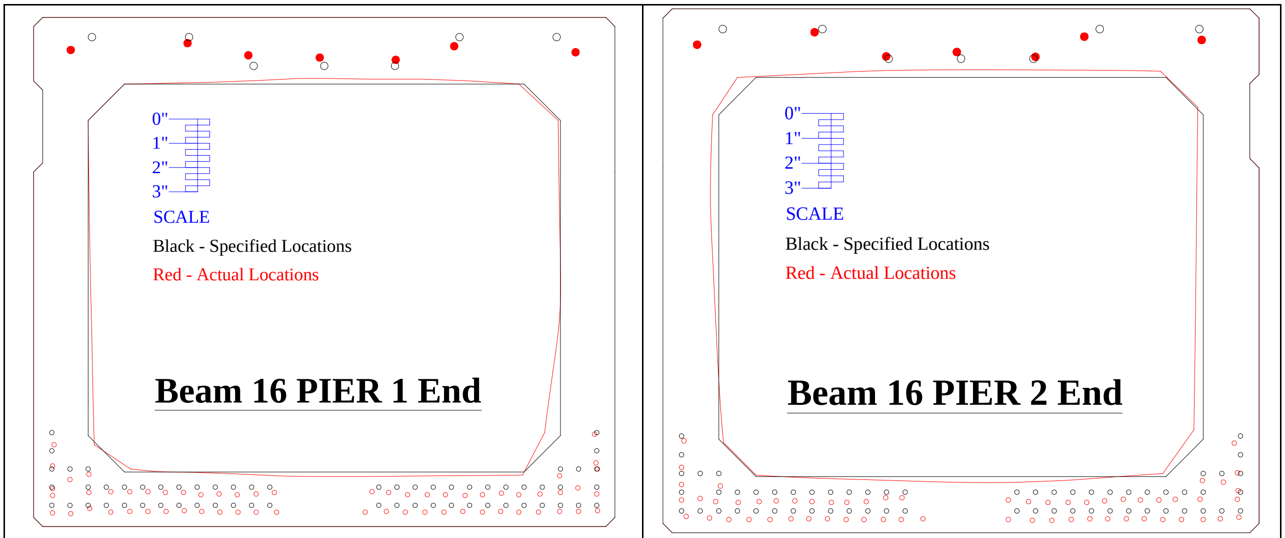


Figure 3-7: Bottom Flange Crack and Spall Condition Span 2 Beam 16

3.2.4 Lakeview Drive Span 2 Beam 16 As-Built

The as-built drawings in Figure 3-8 indicate that the cardboard forms shifted slightly during the concrete placement. The web width was larger for the beams and the top and bottom flange thickness was reduced. It is also noted that reinforcing steel placement deviates from the specified locations such that there is less clear cover than specified. The average clear cover to the bottom layer of strands was 0.975 in. This is of important due to the fact that clear cover has a large effect on protecting the reinforcing steel from corrosion due to chlorides. This reduction in cover could be attributed to the fact that the beam transverse reinforcement was placed on top of the bottom layer of prestressing strand. Without proper support of the transverse reinforcement the added weight would have easily deflected the strands. This effect would have been amplified if a long strand length was used; for example, cases where multiple beams are fabricated in one line. Current PCI construction methods do not allow for support of the transverse reinforcement on strands.



a) As-built pier 1 face b) As-built pier 2 face

Figure 3-8: As-Built Section Geometry of Span 2 Beam 16 of Lakeview Drive Bridge

3.2.5 Lakeview Drive Span 3 Beam 19 Visual Observations

The visual inspection for this beam, Figure 3-9, reveals two prominent cracks along the length of the member. The cracks along the beam varied from hairline to 0.05 in. wide. One crack extending from each pier end. The crack path bifurcates at many points along its length. There is heavy rust-staining present on the member, as well as one minor area of spalling at the pier 2 end of the beam; thus the beam had less than 1% of its concrete surface spalled off. The circular damaged area near the P3 end on the beam was created during previous inspections.

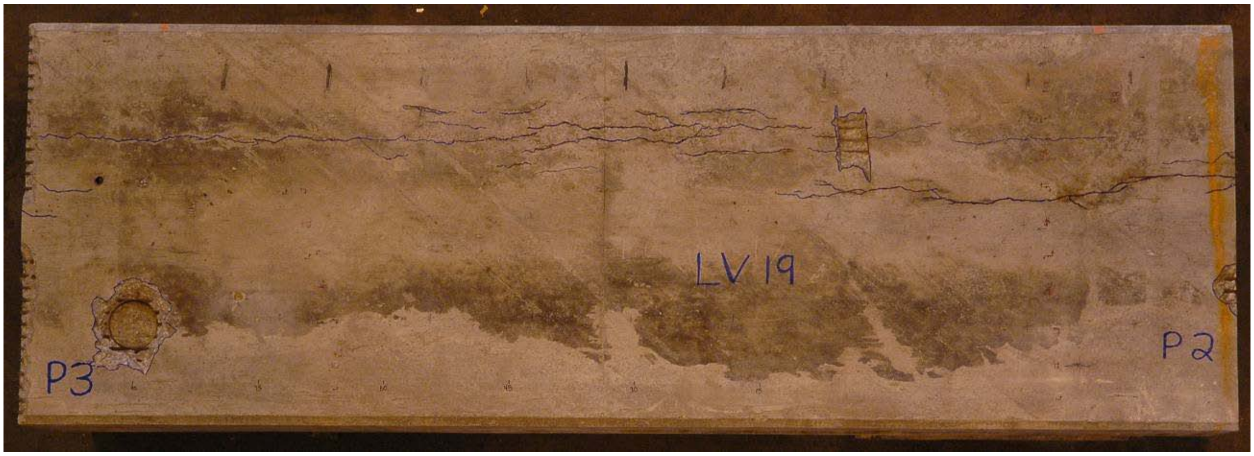
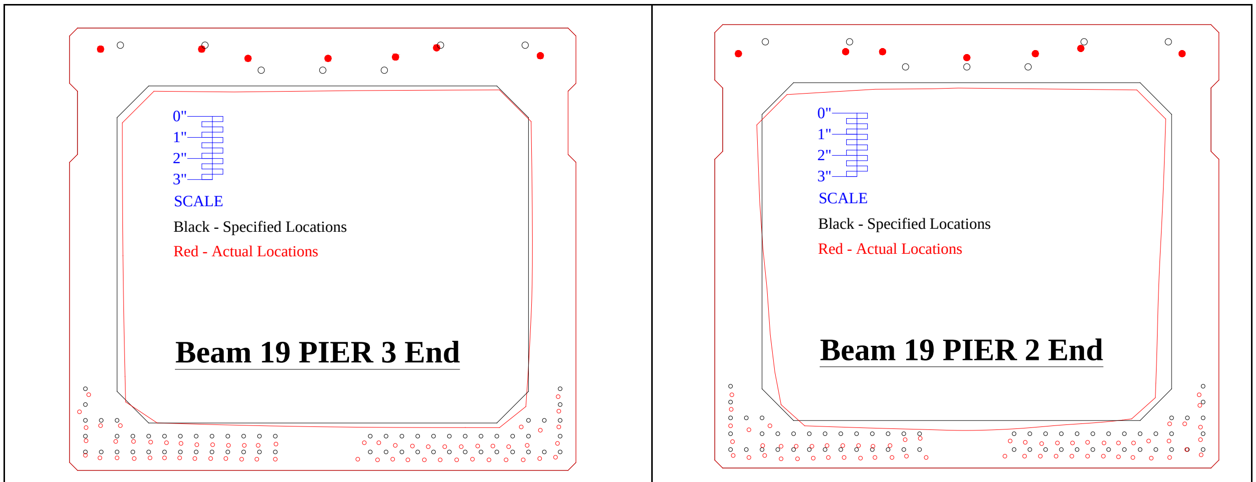


Figure 3-9: Bottom Flange Crack and Spall Condition Span 3 Beam 19

3.2.6 Lakeview Drive Span 3 Beam 19 As-Built

The as-built drawings in Figure 3-10 indicate that the cardboard forms shifted slightly during the concrete placement. The web thickness was increased at the bottom of the beam and the bottom flange thickness was reduced. It is also noted that reinforcing steel placement deviates from the specified locations such that there is less clear cover than called for. The average clear cover to the bottom layer of strands was 0.90 in. This is important due to the fact that clear cover has a large effect on protecting the reinforcing steel from corrosion due to chlorides.



a) As-built reinforcement location b) As-built reinforcement location

Figure 3-10: As-Built Section Geometry of Span 3 Beam 19 of Lakeview Drive Bridge

3.3 Forensic Evaluation of Main Street Beams

Two beams were recovered for further investigation. Sections of the beams were cut and delivered to Lehigh University. Details on the beam section are summarized in Table 3-3.

Table 3-3: Main Street Bridge Beams

Beam	Span	Section Length	Cross Section	Condition Description
2	3	15ft	48x42 Box	Heavy corrosion on bottom flange without longitudinal cracking. Large patches. Determine if corrosion adjacent to patch exists using NDE methods.
3	3	15ft	48x42 Box	Longitudinal crack with heavy splitting. Examine damage formation and NDE study.

3.3.1 Main Street Span 3 Beam 2 Visual Observations

The visual inspection for this beam, Figure 3-11, revealed severe spalling along the length. Strands are completely exposed over a large area of the bottom flange. The strands in the spalled area were heavily corroded and in some cases completely deteriorated. Minor hairline cracking occurred between the spalled region. Due to the significant spalling – approximately 49% of the concrete surface – and corrosion damage the beam is in very poor condition.

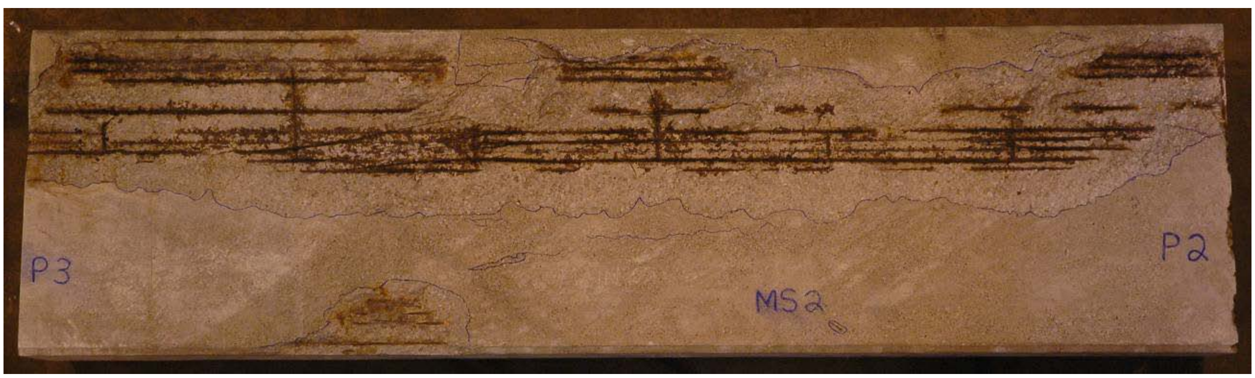
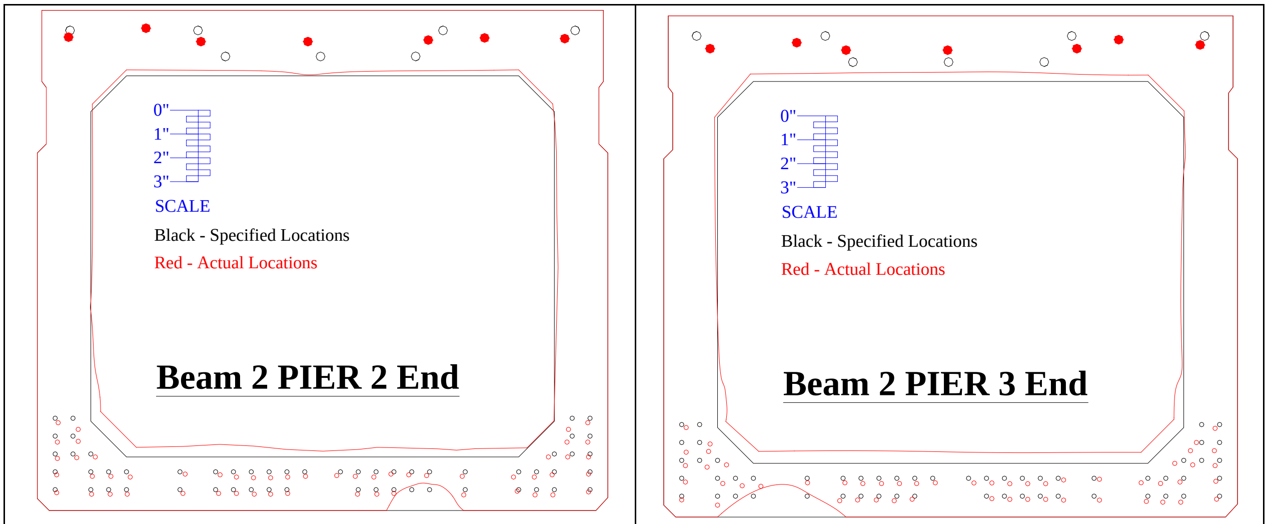


Figure 3-11: Bottom Flange Crack and Spall Condition Span 3 Beam 2

3.3.2 Main Street Span 3 Beam 2 As-Built

The as-built drawings in Figure 3-12 indicate that the cardboard forms shifted slightly during the concrete placement. The web thickness was essentially maintained, however the bottom flange thickness was increased and the top flange thickness was reduced. It is also noted that reinforcing steel placement deviates from the specified locations such that there is less clear cover than called for. The average clear cover to the bottom layer of strands was 1.18 in. In addition two strands were lost at the pier 2 end of the beam and 3 were lost from the pier 3 end.



a) As-built pier 2 face b) As-built pier 3 face

Figure 3-12: As-Built Section Geometry of Span 3 Beam 2 of Main Street Bridge

3.3.3 Main Street Span 3 Beam 3 Visual Observations

The visual inspection for this beam, Figure 3-13, revealed a longitudinal crack along the entire beam length. In some portions of the crack, light spalling had occurred; totaling approximately 2% of the concrete surface. The crack width varied. Efflorescence and rust staining have also been detected.

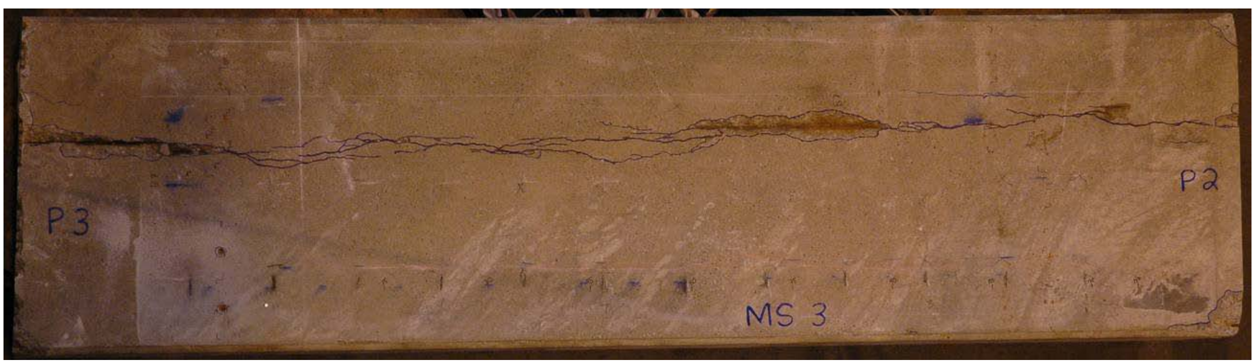
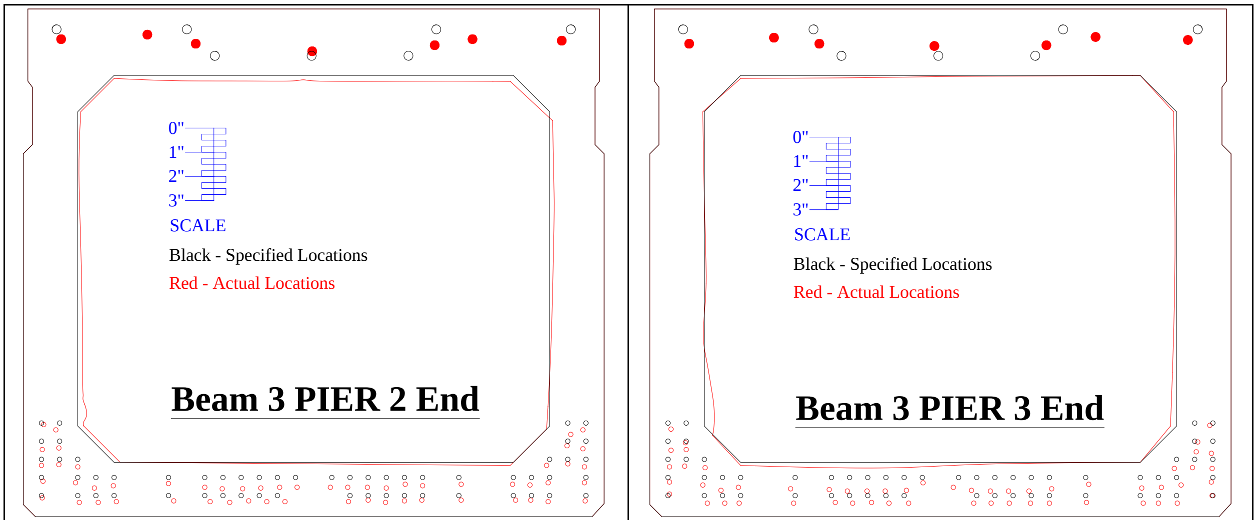


Figure 3-13: Bottom Flange Crack and Spall Condition Span 3 Beam 3

3.3.4 Main Street Span 3 Beam 3 As-Built

The as-built drawings in Figure 3-14 indicate that the cardboard forms shifted slightly during the concrete placement. As a result, it appears that the thickness of the box-girders web and flanges change slightly along the sections length. It is also noted that reinforcing steel placement is off such that there is less clear cover than called for. The average clear cover to the bottom layer of strands was 1.06 in.



a) As-built pier 2 face

b) As-built pier 3 face

Figure 3-14: As-Built Section Geometry of Span 3 Beam 3 of Main Street Bridge

3.4 Trapped Water in Box Sections

Investigation of the box beam voids was conducted prior to the forensic investigation to determine if standing water was present within the voids during service. As previously identified in Naito et. al. 2007, vent holes were installed through the top flange and drain holes were installed in the bottom flange of the box beams during fabrication. The vent holes are used to allow heat to escape during concrete curing. These vent holes were left open after the curing, providing an entry point for water runoff from the bridge deck to enter the void. Over time the water degraded the cast-in-place cardboard forms, which in turn clogged the drain holes in the bottom flange. Once the drains were clogged, water could not exit the beam resulting in the collection of water within the void.

The presence of water within the void has numerous negative effects on the longevity of the bridge. The water creates significant additional dead load on the beams. Since the water comes from deck runoff high chloride levels are present in the water. This could result in corrosion initiating from within the section. In cold regions the trapped water could freeze. The expansion of the water as it turns to ice could result in bursting stresses within the beam thus exacerbating longitudinal spitting cracks generated from corrosion. To alleviate these problems the vent holes should be closed after the curing process and drain holes should be maintained during each inspection.

The presence of water was evaluated in the seven beams through an examination of the cardboard condition within each beam. A summary of the in-situ conditions is presented in Table 3-4.

Table 3-4: Void condition

Beam:	Void Condition:
Clearfield Creek Beam 3 (CC3)	Wet
Clearfield Creek Beam 4 (CC4)	Wet
Main Street Beam 2 (MS2)	Dry

Table 3-4: Void condition	
Main Street Beam 3 (MS3)	Wet
Lakeview Drive Beam 7 (LV7)	Wet
Lakeview Drive Beam 16 (LV16)	Wet
Lakeview Drive Beam 19 (LV19)	Dry

3.5 Clear Cover Measurements

Measurements were taken at the cut ends of each girder. Clear cover was measured to the nearest 1/16th of an inch. The cover was measured at each end of the cut beam sections with the exception of Lake View Drive beam #7, where the cover could not be measured due to significant corrosion damage leading to severe spalling and strand deterioration. The clear cover was measured on a total of 128 strands for a total of 256 cover measurements. The collected data revealed that clear cover was much less than required in most areas. The cover measured for each beam is summarized in Table 3-5. The average clear cover for each end of the cut beam sections is summarized. As noted the strand cover varied from one end of the 15 ft. section to the other.

Table 3-5: Beam Clear Cover				
Beam ID:	Pier End:	Avg Clear Cover (in.):	Pier End:	Avg Clear Cover (in):
CC3	A1	1.57	P1	1.33
CC4	P1	1.35	P2	1.15
LV16	P2	0.91	P1	1.04
LV19	P3	0.91	P2	0.89
MS2	P3	1.21	P2	1.16
MS3	P3	1.02	P2	1.11

Measurements taken on as-built clear cover revealed that only one beam cross-section had an average above the required 1.5 inches. In a statistical analysis of the strand cover it was determined that 92% of all strands at the sectioned ends had less cover than required. The clear cover varied from a maximum of 1.75 in. to a minimum of 0.6875 in. The average clear cover of all measurements taken was 1.12 in. with a standard deviation of 0.23 in. The reduced cover may have contributed to premature corrosion of the strands.

3.6 Sounding

Each of the seven beam specimens were sounded to detect areas where delamination had occurred. This procedure was performed in accordance with ASTM D4580, “Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding”. The bottom flange of each beam was sounded using a metal hammer. Changes in pitch of sound of the hammer strikes indicate areas where delamination exists. A “hollow” sound is heard when the delaminated region is struck with a hammer. The difficulty in performing sounding for this case lies in the construction methods used on these beams. As noted previously, the flange thickness varied due to the cardboard forms used in the beams.

Only three of the seven beams were found to have delaminations according to the sounding method described. These beams are Main Street Beams 2 and 3, and Lakeview Drive Beam 19. Figure 3-15, Figure 3-16, and Figure 3-17 show the locations of delaminations found on the bottom flange of Main Street Beam 2, Main Street Beam 3, and Lakeview Drive Beam 19, respectively. It can be seen that the delaminated regions occurred over or adjacent to the primary longitudinal cracks. It should be noted that during the destructive evaluation, it was found that Lakeview Drive Beam 7 was almost fully delaminated. However, due to the fact that the delamination was fairly deep into the flange (approximately 2 to 3 in.) it was not detected by sounding. A comparison of the sounding results with in-situ damage is discussed in section 7.5.

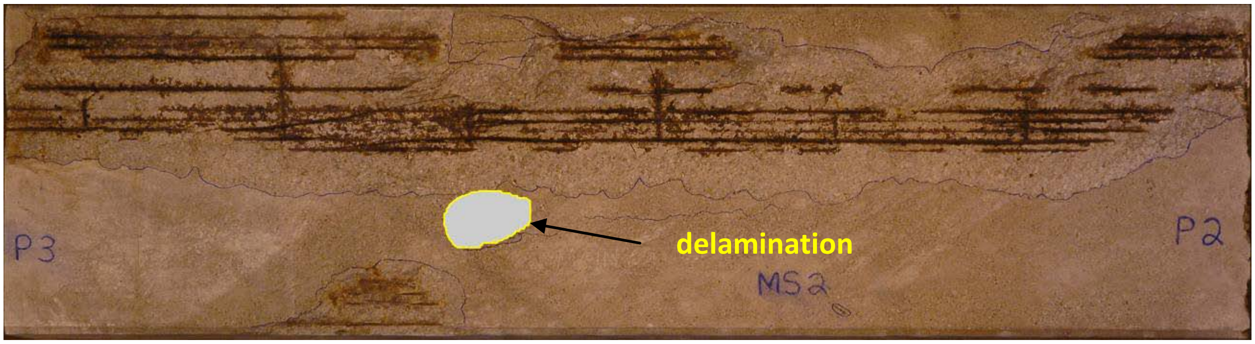


Figure 3-15: Main Street Beam 2 - Regions of delamination found by the sounding method

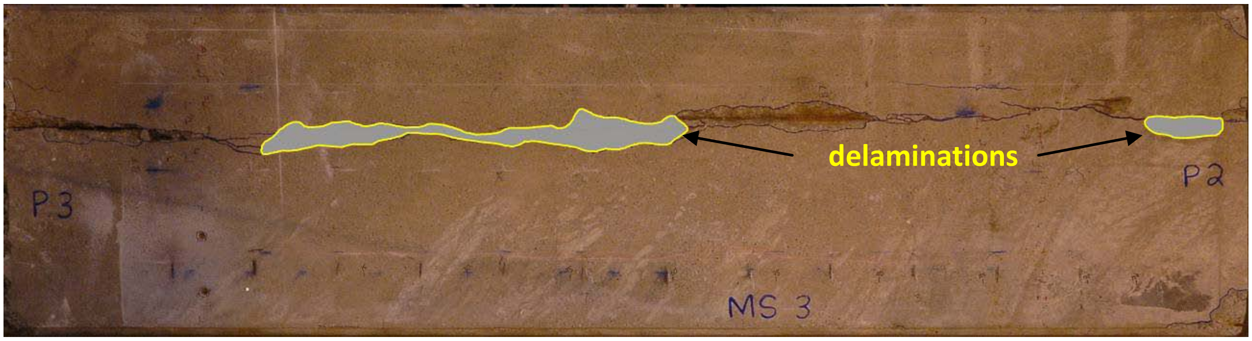


Figure 3-16: Main Street Beam 3 - Regions of delamination found by the sounding method

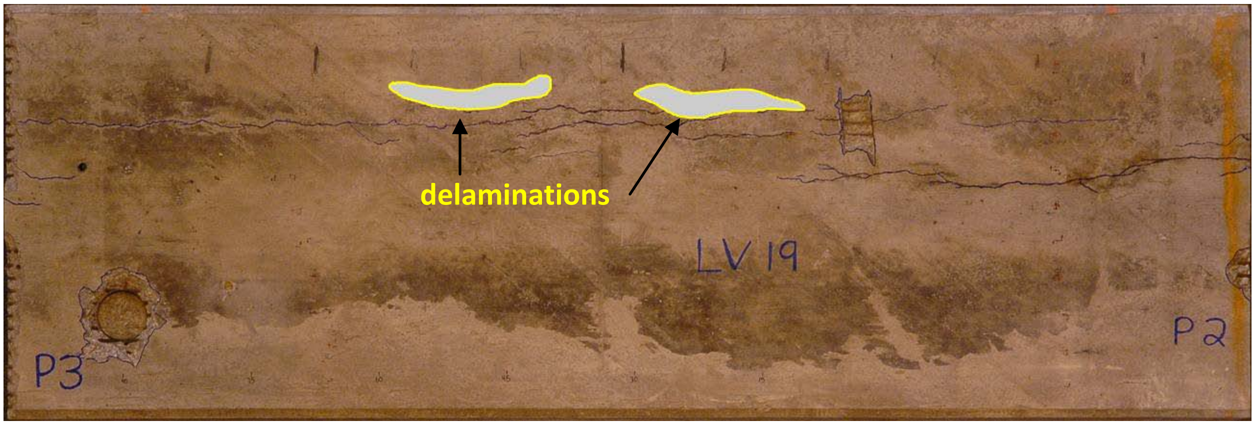


Figure 3-17: Lakeview Drive Bm.19 - Regions of delamination found by the sounding method

3.7 PennDOT Inspection

These beams were evaluated in accordance with the PennDOT inspection methodology. Specialty Engineering, Inc. (SEI) inspected the seven (7) prestressed concrete box beam samples at Lehigh University's ATLSS Laboratory on November 19, 2008. The inspection was performed based on a visual inspection method outlined in PennDOT publication 100A. Some possible defects could not be identified because the beams were cut short from the main section and were placed upside down in the lab. Those defects include: loss of camber, differential deflection between the adjacent beams, and the condition of the transverse tie rods. For all interior beam samples, the condition of the sides of the beams was not factored into the condition rating, since this examination is based on their in-situ inspection conditions. Table 3-6 discusses the various condition ratings; the rating consists of a numeral scale from 0-9 and is based on factors such as spalling, cracks, stains, etc present in the beam. Refer to Table 2-1 and Table 2-2 for information on each beam that led to its condition rating.

Table 3-6: Superstructure Condition Rating Guidelines for Prestressed Box-Beams [PennDOT]

Condition Rating	Percent # strands exposed (single beam)	Other Deterioration of P/S Concrete Beams	
9 - Excellent	0%		No cracks, stains or spalls
8 - Very Good	0%		No cracks, stains or spalls
7 - Good	0%		Map cracks and miscellaneous hairline cracks
6 - Satisfactory	0%	Spalls	Minor spalls/delaminations, <5%
		Cracks	Map cracks and misc. hairline cracks
5 - Fair	1-5%	Spalls	Spalls/delaminations, <15%
		Transverse cracks	None
		Longitudinal cracks	Hairline longitudinal cracks in bottom flange
		Longitudinal Joints	Leakage at joints with light efflorescence
4 - Poor	6-15%	Spalls	Spalls/delaminations, 15-25%
		Transverse cracks	Hairline flexure cracks across bot. flange
		Longitudinal cracks	Minor efflorescence and/or minor rust stains
		Longitudinal Joints	Heavy efflorescence and/or minor rust stains
		Transverse Tendons	Loose or heavily rusted
		Web cracks	Initiation of vert. or diag. cracks in P/S beam near open jts. in barrier (< 3" length)
3 - Serious	15-20%	Spalls	Spalls/delaminations, >25%
		Transverse cracks	Open flexure cracks in bot. Flange
		Web cracks	Vert. or diag. cracks in P/S beam near open jts. in barrier
		Camber	Sagging/Loss of camber
		Transverse Tendons	Broken or missing
2 - Critical	>20%	All	Any cond. worse than detailed above

The visual inspection report was completed and submitted to Lehigh University. The beams were situated in the North Bay of the ATLSS Center in an inverted position. As a consequence of the orientation and the location of the beams, only the beam soffit was examined. The beams were rated in accordance with the PennDOT strike-off letter issued in September of 2007. Three of the beams were rated at 2; a critical condition. The inspection report is included as an appendix to this report.

To supplement the inspection of the procured beam sections, the final In-Service State inspections were examined. The final in-service ratings of the superstructures are summarized in Table 3-7. Inspections on these bridges prior to the 2007 recommendation were based on the overall condition of the bridge beams and not on an individual beam. As a consequence the Lakeview Drive and Main Street Bridges had a less severe rating during their final in-service inspection. Clearfield Creek was inspected six months after the Lakeview Drive collapse. Longitudinal cracking of a majority of the Clearfield bridge beams and the separation of the beams due to the corrosion of the tie rods resulted in the lower in-service rating.

Table 3-7: PennDOT Beam Rating							
Bridge	Beam	Span	Section Length	Cross Section	Lehigh PennDOT Rating	In-Service Inspection Date	In-Service Rating
CC	3	1	15ft	36x42	5	06/13/06	2
CC	4	2	15ft	36x42	4	06/13/06	2
LV	7	1	15ft	48x27	2	03/25/04	4
LV	16	2	12ft	48x42	4	03/25/04	4
LV	19	3	12ft	48x42	2	03/25/04	4
MS	2	3	15ft	48x42	2	01/04/06	3
MS	3	3	15ft	48x42	4	01/04/06	3

Utilizing the new rating guidelines provides a more restrictive evaluation of the bridge condition. As illustrated in Table 3-7 applying the new guidelines to just a small portion of one beam from each bridge would have reduced the bridge rating of three of the bridges to a critical level. The guideline provides a conservative guide and is appropriate for adjacent box beam systems.

3.8 Summary of As-Built Conditions

The following bullets highlight the findings of the in-situ investigation:

- On all sections the cast-in-place forms shifted during the concrete placement. This resulted in an increase or decrease in the web or flange thickness. The change in thickness was arbitrary and was not more prevalent in one location over another.
- On all sections the placement of the reinforcing steel strand was erratic resulting in considerable deviation between the specified locations and as-built locations. The variation was both in the horizontal plane and vertical plane.
- For beams with stirrups in the bottom flange, the reinforcement was placed on top of the bottom layer of strands. This additional weight likely deflected the strands prior to concrete placement resulting in a reduction in clear cover.
- Placement of transverse steel on top of the bottom layer of reinforcement was common in 1950-1960 prestressed beam construction. This practice is no longer allowed. Current construction requires that all strands be contained within stirrups and that the stirrups be self supported.
- Over 90% of all bottom layer strands had a clear cover less than the required 1.5 inches.
- A large delamination was found in LV7 during the visual inspection. The large delamination was not identifiable using sounding due to the depth of the delamination.
- Limited areas of localized “delamination” were identified in Beams MS2, MS3, and LV19 using the ASTM sounding method. A verification of these regions will be discussed in the following sections of the report.
- A visual inspection performed by SEI, in accordance with the current PennDOT inspection guidelines resulted in a critical rating for three of the beams: MS2, LV7, and LV19. The rating of the individual beams using the new guideline provided a lower rating than the previous in-service rating conducted using the old guideline. The new guideline is conservative and appropriate for adjacent non-composite box beam systems.

4 Examination of Concrete Core Samples

Core samples were extracted from each beam and laboratory tests were performed by both Lehigh University and The Erlin Company (TEC) of Latrobe, PA. Two types of cores were extracted. The 4 in. nominal diameter cores were used for petrographic examinations, chloride levels, and air-void analysis. The 2 in. nominal diameter cores were used for concrete compressive strength tests. Additional 4 in. cores – designated by a prime notation, i.e.,: 2B' – were taken with the intent of extracting the strands for visual inspection. This section of the report details the results obtained from the testing of these core samples.

4.1 Beam Core Locations

Cores with nominal diameters of 2 and 4 in. were extracted from each beam. Table 1-1 provides a summary of the core samples. The "S" cores have nominal diameters of 2 in. The core number, core type, and the location on the bottom flange relative to the corner of the section are specified below in Table 4-1:

Table 4-1: Core Locations on Beam Surface

Core ID	Category	Bridge	Beam	Span	Top/Bot	Location (X,Y) [in.]
1	S	MS	3	P3 --> P2	Bottom	(64.5,9.0)
1'	S	MS	3	P3 --> P2	Bottom	(68.7,9.1)
1"	S	MS	3	P3 --> P2	Bottom	(59.3,9.5)
2	B	MS	3	P3 --> P2	Bottom	(62.6,32.8)
2'	B	MS	3	P3 --> P2	Bottom	(68.6,32.3)
3	D	MS	3	P3 --> P2	Bottom	(61.6,42.0)
4	C	MS	3	P3 --> P2	Bottom	(144.5,12)
4'	C	MS	3	P3 --> P2	Bottom	(151.1,12.6)
5	A	MS	3	P3 --> P2	Bottom	(144.5,36.7)
5'	A	MS	3	P3 --> P2	Bottom	(151.3,36.6)
6	B	MS	3	P3 --> P2	Bottom	(168.8,32.4)
8	B	MS	2	P3 --> P2	Bottom	(86.8,16.9)
8'	B	MS	2	P3 --> P2	Bottom	(93.9,15.4)
9	S	MS	2	P3 --> P2	Bottom	(142,9.6)
9'	S	MS	2	P3 --> P2	Bottom	(145.9,9.4)
9"	S	MS	2	P3 --> P2	Bottom	(137,9.4)
10	D	MS	2	P3 --> P2	Bottom	(154.5,16.6)
11	B	CC	3	A1 --> P1	Bottom	(41.7,3.6)
12	C	CC	3	A1 --> P1	Bottom	(41.5,16.7)
12'	C	CC	3	A1 --> P1	Bottom	(48.7,16.6)
13	B	CC	3	A1 --> P1	Bottom	(41.7,21.9)
13'	B	CC	3	A1 --> P1	Bottom	(48.8,22.2)
14	D	CC	3	A1 --> P1	Bottom	(41.7,30.9)

Table 4-1: Core Locations on Beam Surface

Core ID	Category	Bridge	Beam	Span	Top/Bot	Location (X,Y) [in.]
15	S	CC	3	A1 --> P1	Bottom	(121.1,9.2)
15'	S	CC	3	A1 --> P1	Bottom	(124.6,9.0)
15"	S	CC	3	A1 --> P1	Bottom	(115.7,9.0)
16	A	CC	3	A1 --> P1	Bottom	(138.6,16)
16'	A	CC	3	A1 --> P1	Bottom	(145,15.8)
17	D	LV	19	P3 --> P2	Bottom	(21.0,5.2)
18	C	LV	19	P3 --> P2	Bottom	(132.0,12.5)
18'	C	LV	19	P3 --> P2	Bottom	(125.5,12.9)
19	S	LV	19	P3 --> P2	Bottom	(33.7,23.5)
19'	S	LV	19	P3 --> P2	Bottom	(39.3,23.5)
19"	S	LV	19	P3 --> P2	Bottom	(28.9,23.5)
20	B	LV	19	P3 --> P2	Bottom	(62.0,35.9)
21	B	LV	19	P3 --> P2	Bottom	(134.5,30)
21'	B	LV	19	P3 --> P2	Bottom	(128.2,30.0)
22	C	LV	16	P2 --> P1	Bottom	(77.4,8.6)
22'	C	LV	16	P2 --> P1	Bottom	(70.1,8.4)
23	D	LV	16	P2 --> P1	Bottom	(75.2,35.5)
24	S	LV	16	P2 --> P1	Bottom	(75.2,25.1)
24'	S	LV	16	P2 --> P1	Bottom	(79.6,25.2)
24"	S	LV	16	P2 --> P1	Bottom	(70.6,24.7)
24"'	S	LV	16	P2 --> P1	Bottom	(41.7,24.8)
24""	S	LV	16	P2 --> P1	Bottom	(38.5,24.8)
24""'	S	LV	16	P2 --> P1	Bottom	(45.0,24.8)
25	B	LV	16	P2 --> P1	Bottom	(133.5,28.1)
25'	B	LV	16	P2 --> P1	Bottom	(124.1,28.5)
26	C	LV	7	P1 --> A1	Top	(86,22)
27	D	LV	7	P1 --> A1	Top	(99,25.5)
28	A	LV	7	P1 --> A1	Top	(149,22)
29	S	LV	7	P1 --> A1	Top	(105.5,24)
29'	S	LV	7	P1 --> A1	Top	(116.5,25.5)
29"	S	LV	7	P1 --> A1	Top	(88.5,25)
29"'	S	LV	7	P1 --> A1	Top	(77,24)
30	B	LV	7	P1 --> A1	Top	(38.5,19)
30'	B	LV	7	P1 --> A1	Top	(30,19)
31	A	CC	4	P1 --> P2	Bottom	(23,16)
31'	A	CC	4	P1 --> P2	Bottom	(30,16)

Table 4-1: Core Locations on Beam Surface						
Core ID	Category	Bridge	Beam	Span	Top/Bot	Location (X,Y) [in.]
32	S	CC	4	P1 --> P2	Bottom	(132.5,27)
32'	S	CC	4	P1 --> P2	Bottom	(135.25,27)
32"	S	CC	4	P1 --> P2	Bottom	(128.5,27)
32"'	S	CC	4	P1 --> P2	Bottom	(28.5,27)
32""	S	CC	4	P1 --> P2	Bottom	(21,27)
33	B	CC	4	P1 --> P2	Bottom	(155,8)
33'	B	CC	4	P1 --> P2	Bottom	(150,8)
34	D	CC	4	P1 --> P2	Bottom	(156,30)
35	C	MS	2	P3 --> P2	Bottom	(25.3,7.1)
35'	C	MS	2	P3 --> P2	Bottom	(31.6,8.7)

In addition to the 2 and 4 in. cores, 1 in. nominal diameter cores – or “plugs” – were taken from beams CC4 and LV19. These were taken with the intent of acquiring a chloride profile along the bottom flange surface of the beams. The locations of the plugs are summarized in Table 4-2.

Table 4-2: Plug Locations on Beam Surface			
Plug ID	Ref. Pier End for X	Y Measured From Top/Bot?	X,Y (in)
CC4-1	P1	Bottom	22.75,31.5
CC4-2	P1	Bottom	22.75,29.25
CC4-3	P1	Bottom	22.75,26.75
CC4-4	P1	Bottom	22.75,24.25
CC4-5	P1	Bottom	22.75,21.5
CC4-6	P1	Bottom	22.75,12.25
CC4-7	P1	Bottom	22.75,10
CC4-8	P1	Bottom	22.75,7.75
CC4-9	P1	Bottom	22.75,5.25
LV19-1	P3	Bottom	61.5,42.75
LV19-2	P3	Bottom	61.5,30.5
LV19-3	P3	Bottom	61.5,26.75
LV19-4	P3	Bottom	61.5,23.25
LV19-5	P3	Bottom	61.5,19.5
LV19-6	P3	Bottom	61.5,15.75
LV19-7	P3	Bottom	61.5,12.25
LV19-8	P3	Bottom	61.5,8.75
LV19-9	P3	Bottom	61.5,5.25

Beam CC3 had 11 cores extracted overall, as shown in Figure 4-1. Three strength cores were taken, 5 cores were sent to TEC for analysis, and 3 cores were designated for strand extraction.

Core 14D was designated for petrographic testing. Cores 13 and 16 were located along a longitudinal crack.

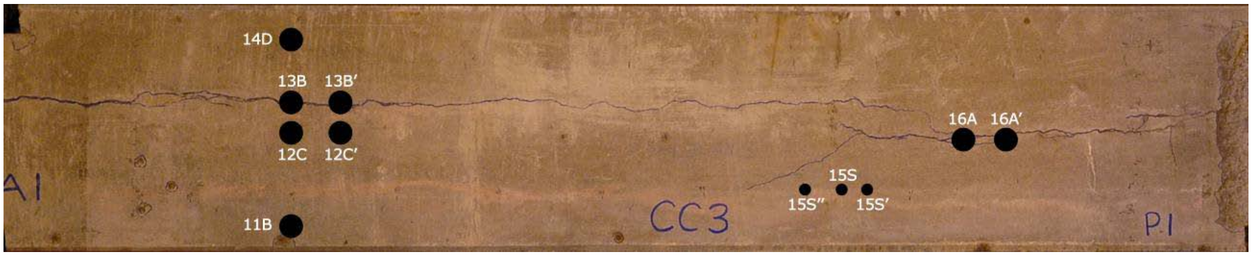


Figure 4-1: Core Locations for Beam CC3

Beam CC4 had 19 cores extracted overall, as shown in Figure 4-2. Five strength cores were extracted, 12 cores were sent to TEC for analysis, and 2 cores were designated for strand extraction. Core 34D was designated for petrographic testing. Cores 31 and 33 were located along a longitudinal crack.

Figure 4-2: Core Locations for Beam CC4

Beam LV7 had 8 cores extracted overall, as shown in Figure 4-3. Four strength cores were extracted, 3 cores were sent to TEC for analysis, and 1 core was designated for strand extraction. Core 27D was designated for petrographic testing. Core 30 was located along a longitudinal crack.

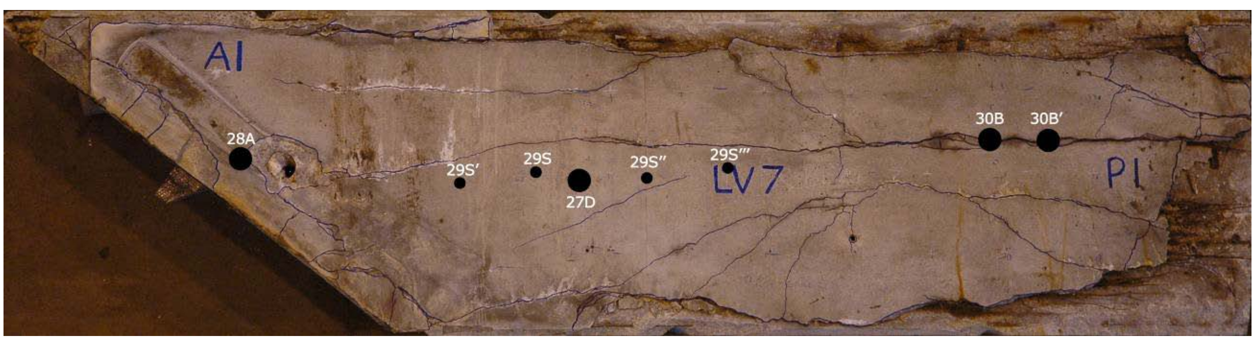


Figure 4-3: Core Locations for Beam LV7

Beam LV16 had 11 cores extracted overall, as shown in Figure 4-4. Six strength cores were extracted, 3 cores were sent to TEC for analysis, and 2 cores were designated for strand extraction. Core 23D was designated for petrographic testing. Core 25B was located along a longitudinal crack.

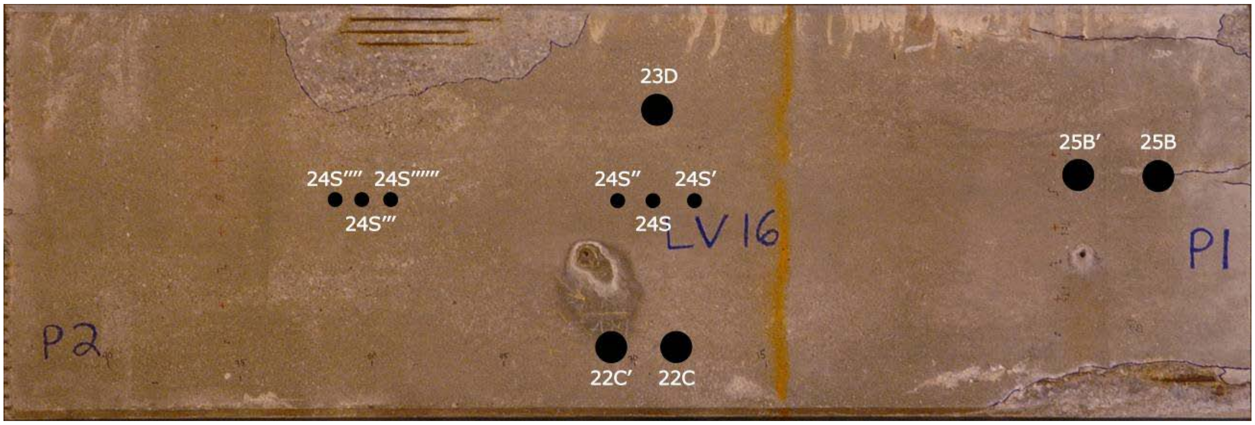


Figure 4-4: Core Locations for Beam LV16

Beam LV19 had 28 cores extracted overall, as shown in Figure 4-5. Three strength cores were extracted, 13 cores were sent to TEC for analysis, and 2 cores were designated for strand extraction. Core 17D was designated for petrographic testing. Cores 20 and 21 were located along a longitudinal crack.

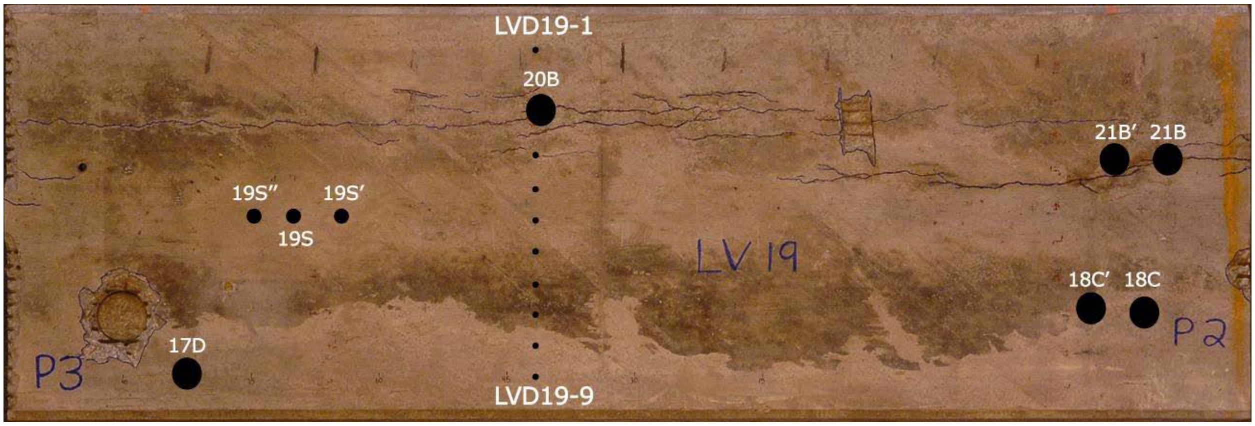


Figure 4-5: Core Locations for Beam LV19

Beam MS2 had 8 cores extracted overall, as shown in Figure 4-6. Three strength cores were extracted, 3 cores were sent to TEC for analysis, and 2 cores were designated for strand extraction. Core 10D was designated for petrographic testing. Core 8 was located along a longitudinal crack.

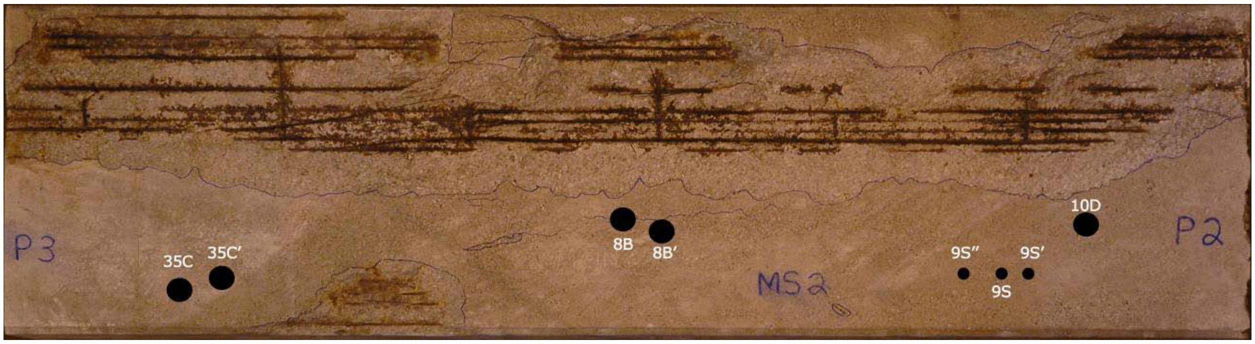


Figure 4-6: Core Locations for Beam MS2

Beam MS3 had 11 cores extracted overall, as shown in Figure 4-7. Three strength cores were extracted, 5 cores were sent to TEC for analysis, and 3 cores were designated for strand extraction. Core 3 was designated for petrographic testing. Cores 2 and 6 were located along a longitudinal crack.

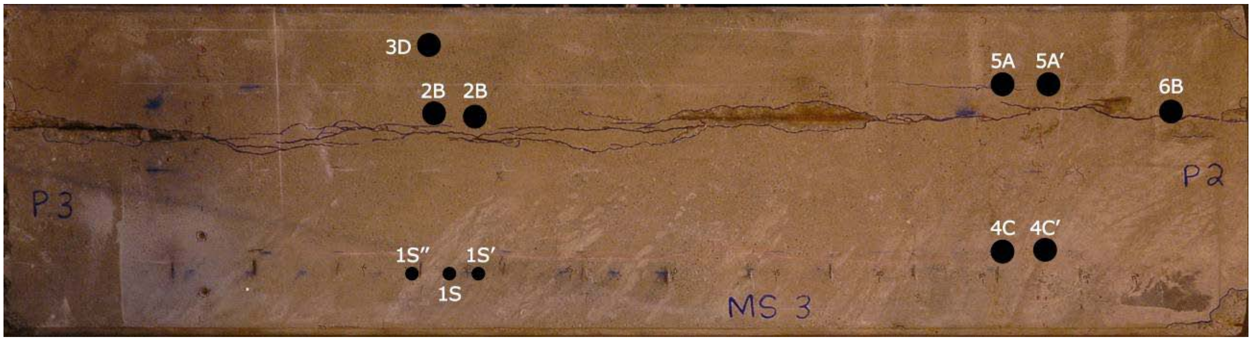


Figure 4-7: Core Locations for Beam MS3

4.2 TEC Core Evaluations

Petrographic examinations, air-void analyses, and chloride profile analyses were conducted on cores removed from all seven beams. The number of tests varied among the core category. Accordingly, appropriate samples were: (1) examined using methods of ASTM C856, “Petrographic Examination of Hardened Concrete”; (2) analyzed for air-void parameters using the modified point-count method of ASTM C457, “Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete”; (3) analyzed for chlorides using methods of ASTM 1152, “Acid-Soluble Chloride in Mortar and Concrete”; and (4) analyzed for depths of carbonation using a phenolphthalein indicator supplemented by petrographic microscopy. This section summarizes the results of these studies.

The content of this section of the report has been adapted from the final report by The Erlin Company (TEC), entitled “Petrographic Examinations and Air-Void and Chloride Analyses of Concrete Cores for ATLSS Lehigh University (Box Beams).” Report TEC 409124, Latrobe, PA, September 2009. In some cases, the adaptation is made verbatim with editorial changes to maintain consistency in terminology.

4.2.1 Core Summary

Twenty-four core samples between the seven beams were analyzed by TEC. They are identified relative to their location on the beam as shown below. Core diameters are 3 ¾ inches and generally vary in length from 4 inches to 7 3/8 inches. Cores from LV7 can be as short as 1 ½ inches because the large delamination created a problem for drilling. Outside ends of the cores are formed surfaces; other ends of Cores 3D, 10D, 14D, and 17D are formed surfaces – other ends of Cores 23D and 34D are fractured surfaces. A summary of core dimensions, type, size, and depths of reinforcing and prestressing steel, as well as tests completed for each core can be found in Table 4-3.

Table 4-3: Summary of Cores Submitted to TEC

Core No.	Core		Length (in.)	Strand Cover (in.)	Petrographic Examination	Air Void Analysis	Chloride Analysis[1]
1	3D	P2 M53	$6^{5}/_{8}$	$^{7}/_{8}$, $2^{1}/_{8}$	X	X	A
2	10D	M2	$5^{1}/_{4}$	$1^{1}/_{4}{}^{(2)}$, $2^{1}/_{2}$, $2^{1}/_{2}$, $2^{7}/_{8}$	X	X	A
3	14D	P1 CC3	$7^{1}/_{4}$	$1^{1}/_{2}$, $1^{1}/_{2}$, 3, 3	X	X	A
4	17D	P2 LV19	$7^{3}/_{8}$	$^{7}/_{8}$, 1, $2^{1}/_{2}$, $2^{1}/_{2}$, 4, $4^{1}/_{2}{}^{(3)}$	X	X	A
5	23D	P1 LV16	$4^{1}/_{2} – 4^{5}/_{8}$	1, 1, $2^{1}/_{2}$, $2^{1}/_{2}$	X	X	A
6	27D	P1 LV7	$3^{1}/_{2} – 4^{1}/_{8}$	No strands	X	X	A
7	34D	P2 CC4	$4^{1}/_{8} – 5^{1}/_{8}$	$1^{1}/_{8}$, $1^{1}/_{4}$, $2^{5}/_{8}$, $2^{5}/_{8}$	X	X	A
10	5A	P2 M53	$4 – 4^{3}/_{8}$	$1^{1}/_{8}$, $2^{1}/_{2}$			B
15	16A	P1 CC3	$4 – 4^{1}/_{4}$	$1^{1}/_{2}$, $2^{3}/_{4}$			B
21	28A	P1 LV7	$1^{1}/_{2} – 2^{1}/_{8}$	$1^{1}/_{8}$, $1^{1}/_{4}{}^{(3)}$			B
23	31A	P2 CC	$5^{1}/_{8} – 5^{3}/_{8}$	$1^{1}/_{4}$, $1^{1}/_{4}$, 3, 3			B
8	2B	P2 M53	$3^{3}/_{4}$ - 4	$^{7}/_{8}$, $^{7}/_{8}$, $1^{7}/_{8}$, 2			B
11	8B	P2 M52	$5^{7}/_{8}$	$1^{3}/_{8}$, $2^{5}/_{8}$			B
14	13B	P1 CC3	$4 – 4^{1}/_{4}$	$1^{1}/_{4}$, $1^{1}/_{2}{}^{(3)}$, $2^{3}/_{4}$			B
17	20B	P2 LV19	$4^{(4)}$	$^{7}/_{8}{}^{(2)}$, 2, 2, 2			B
18	21B	P2 LV19	$3^{3}/_{4}{}^{(4)}$	$^{3}/_{4}{}^{(3)}$, $1^{7}/_{8}$, 2, $2^{1}/_{2}$			B
20	25B	P1 LV16	$4^{1}/_{2} – 4^{3}/_{4}$	1, $2^{5}/_{8}$			B
22	30B	P1 LV7	4	$1^{1}/_{8}$, $1^{1}/_{4}{}^{(3)}$			B
24	33B	P2 CC4	8	$1^{1}/_{4}$, $2^{3}/_{4}$			B
9	4C	P2 M53	$3^{1}/_{2} – 4^{3}/_{8}$	$1^{1}/_{8}$, $2^{3}/_{8}$			B
13	12C	P1 CC3	$4^{1}/_{4}$	$1^{1}/_{2}$, $1^{1}/_{2}$, $2^{5}/_{8}$, $2^{7}/_{8}$			B
16	18C	P2 LV19	$4 – 4^{1}/_{4}$	$^{7}/_{8}$, $^{7}/_{8}$, $^{7}/_{8}$, 2, $2^{1}/_{8}$, $2^{1}/_{8}$			B
19	22C	P1 LV16	6	1, 1, $2^{1}/_{2}$, $2^{1}/_{2}$			B
12	35C	P2 M52	7	$1^{1}/_{4}$, $2^{3}/_{4}$			B

(1) "A" denotes 4 analyses per core: outer surface; 2 strand levels; and inside surface. "B" denotes 3 analyses per core: outer surface; and 2 strand levels.
(2) Strands separated.
(3) Cast impression.
(4) Estimate.

Seven cores scheduled for the petrographic and air-void analyses were saw-cut longitudinally and lapped so cross-sections were available for the studies, the remaining concrete was saw-cut into 1/2-inch thick sections at appropriate depths, processed, and used for chloride analyses; the remainder was broken-up and used for more detailed petrographic examinations and depths of carbonation analyses. The remaining 3.75 in. diameter cores were saw-cut into 0.5 in. thick sections at appropriate depths, processed, and used for chloride analyses. A sample of the in-situ condition of cores sent to TEC can be seen in Figure 4-8.

Side C

Top Bottom

Figure 4-8: Sample of 3 ¾ inch Core Submitted for Analysis

In addition to the standard 3 ¾ inch cores, there were eighteen 5/8 inch diameter cores, or plugs, submitted and processed for chloride analyses. These plugs are shown in Figure 4-9.

Figure 4-9: 5/8 inch Diameter Plugs Submitted for Chloride Analyses

4.2.2 Petrographic Examinations

One Category D core was taken from the bottom flange of each beam and analyzed petrographically. Each core was sliced, photographed, and tested for: air void analyses, carbonation, strand evaluation, and chloride analyses. The results of the study are presented through the figures and data as shown below.

4.2.2.1 Petrography Photos

A cross-section of Core 3D showing relatively good grading and distribution of aggregates can be seen in Figure 4-10. Non-corroded seven-wire prestressing strands are circled. Arrows point to a crack at the top strand level. Aside from the crack there is no evidence of distress to the concrete. The core was taped to facilitate processing. The acid-soluble chloride contents are overlaid on the core section; the values shown are percent by concrete mass. The scale is in inches.

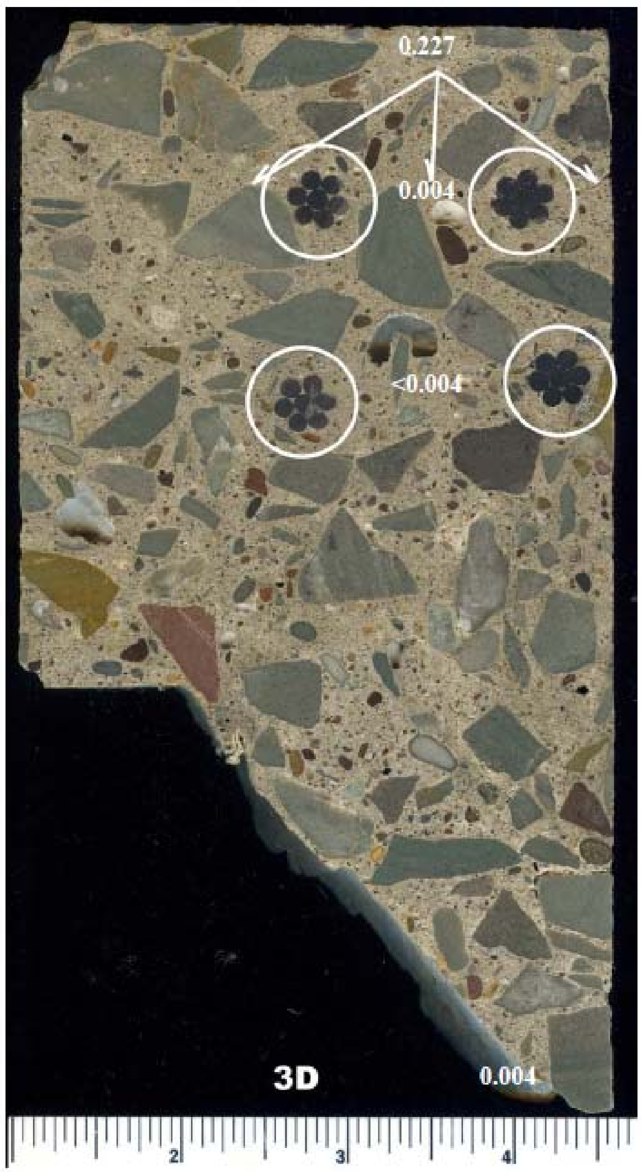


Figure 4-10: Petrographic Examination of Core 3D from Beam MS3

A cross-section of Core 10D showing relatively good grading and distribution of aggregates is shown in Figure 4-11. The crack separated the concrete such that the strand wires – lightly to severely corroded – were displaced. Within the yellow-boxed area is a vertically trending crack (arrows) and severely corroded wire strand (circled). The circled bottom strands are non-corroded. Aside from the cracks and corroded strand there is no evidence of distress to the concrete. The core was taped to facilitate processing. The acid-soluble chloride contents shown are percent by concrete mass. The scale is in inches.

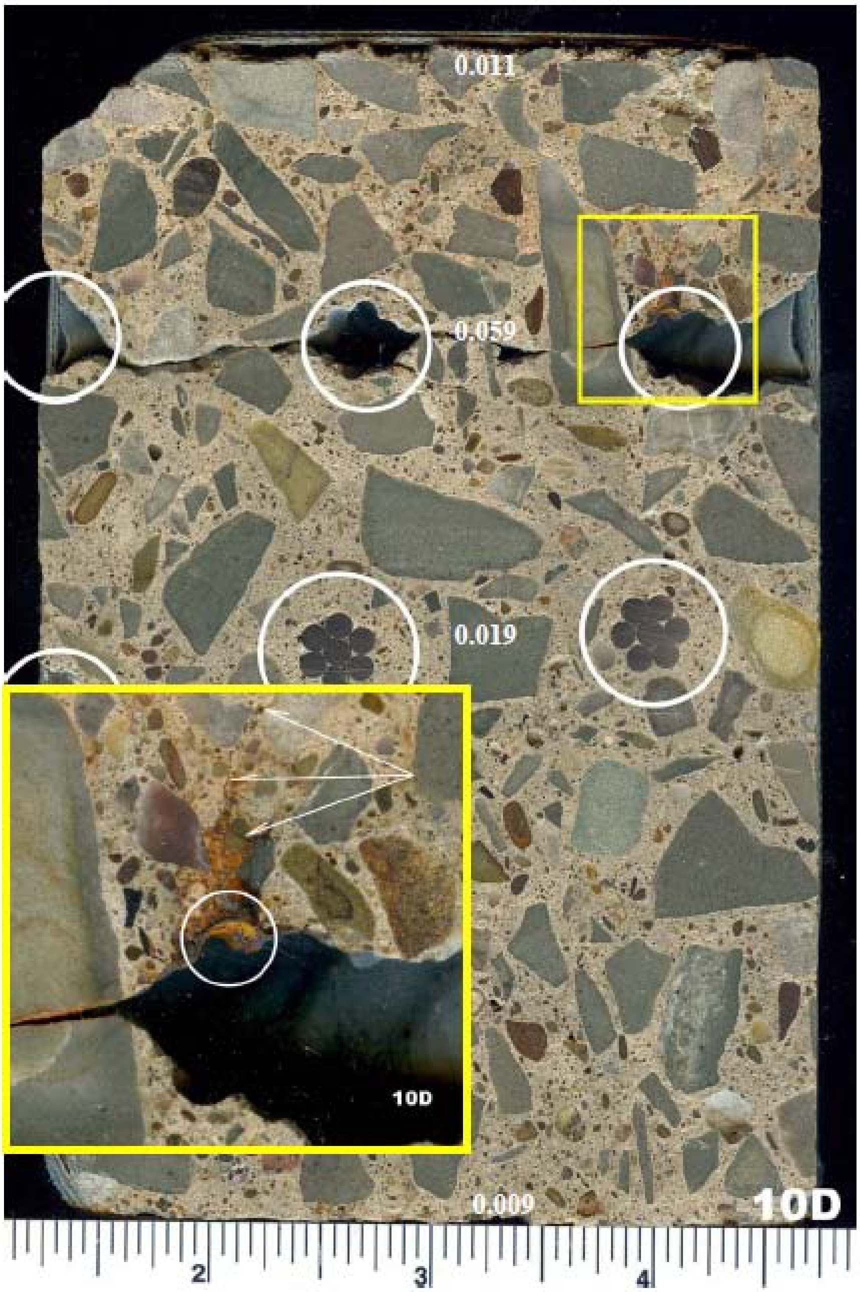


Figure 4-11: Petrographic Examination of Core 10D from Beam MS2

A cross-section of Core 14D showing relatively good grading and distribution of aggregates can be seen in Figure 4-12. Within the circles are non-corroded strands. The arrows point to cracks at the strand level. Aside from cracks there is no evidence of distress to the concrete. The core was taped to facilitate processing. The acid-soluble chloride contents shown are percent by concrete mass. The scale is in inches.

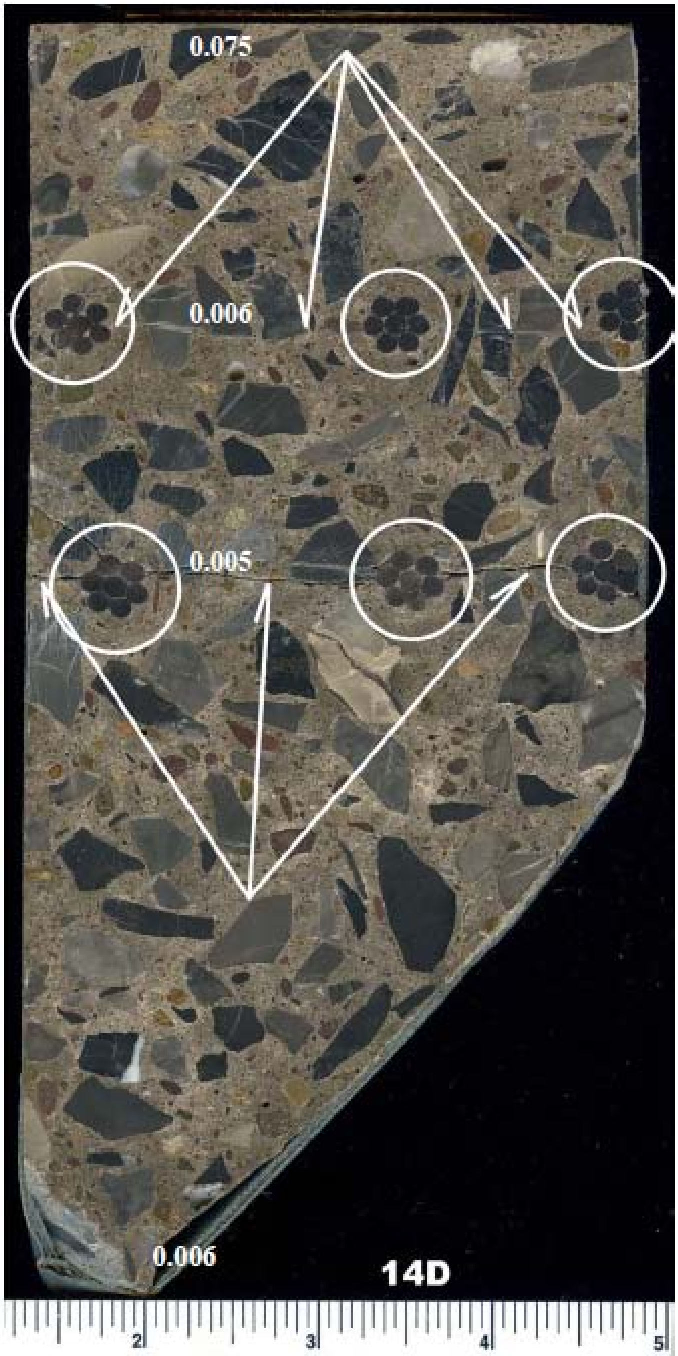


Figure 4-12: Petrographic Examination of Core 14D from Beam CC3

Cross-section of Core 17D showing relatively good grading and distribution of aggregates can be seen in Figure 4-13. Non-corroded seven-wire prestressing strands are circled. There is no evidence of distress to the concrete. The core was taped to facilitate processing. The acid-soluble chloride contents shown are percent by concrete mass. The scale is in inches.

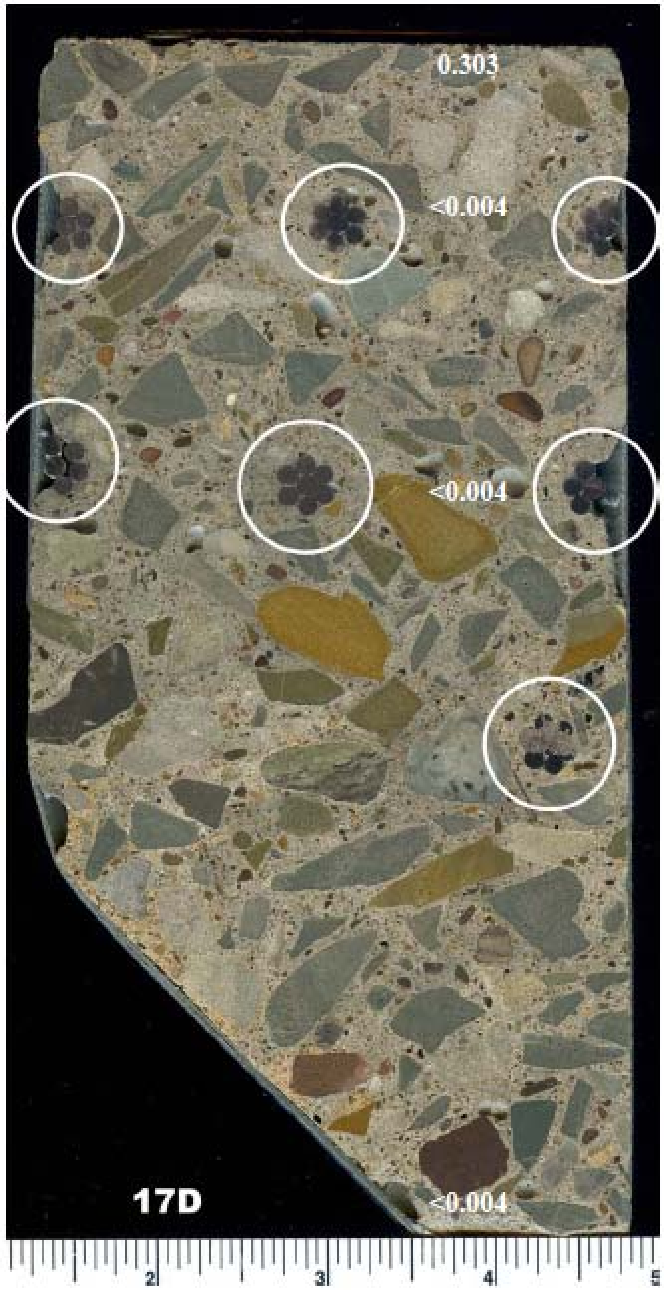


Figure 4-13: Petrographic Examination of Core 17D from Beam LV19

Cross-section of Core 23D showing relatively good grading and distribution of aggregates can be seen in Figure 4-14. Non-corroded seven-wire prestressing strands are circled. There is no evidence of distress to the concrete. The acid-soluble chloride contents shown are percent by concrete mass. The scale is in inches.

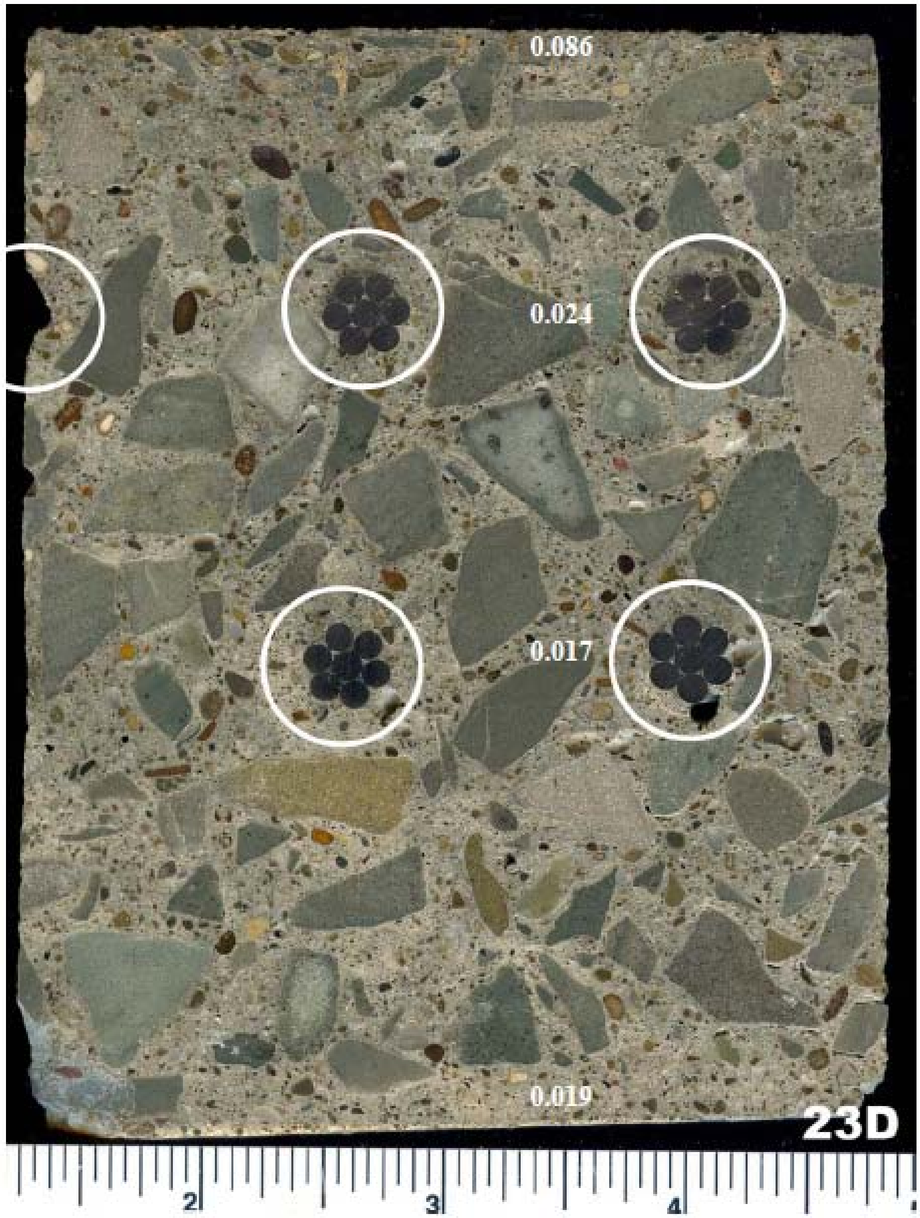


Figure 4-14: Petrographic Examination of Core 23D in Beam LV16

A cross-section of Core 27D showing relatively good grading and distribution of aggregates can be seen in Figure 4-15. There is no evidence of distress to the concrete. The acid-soluble chloride contents shown are percent by concrete mass. The scale is in inches.

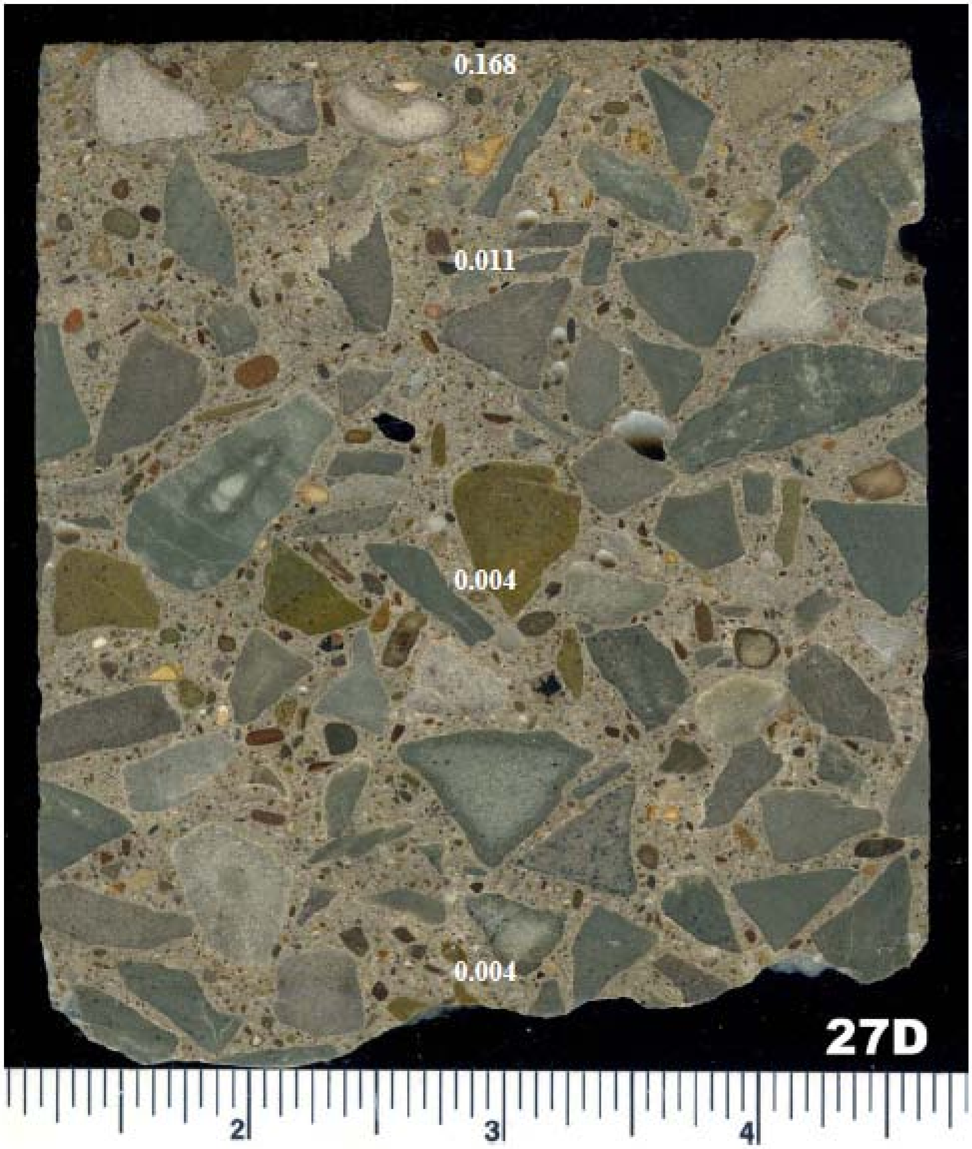


Figure 4-15: Petrographic Examination of Core 27D in Beam LV7

A cross-section of Core 34D showing relatively good grading and distribution of aggregates can be seen in Figure 4-16. Within the circles are non-corroded strands. The arrows point to cracks at strand levels. Aside from the cracks there is no evidence of distress to the concrete. The core was taped to facilitate processing. The acid-soluble chloride contents shown are percent by concrete mass. The scale is in inches.

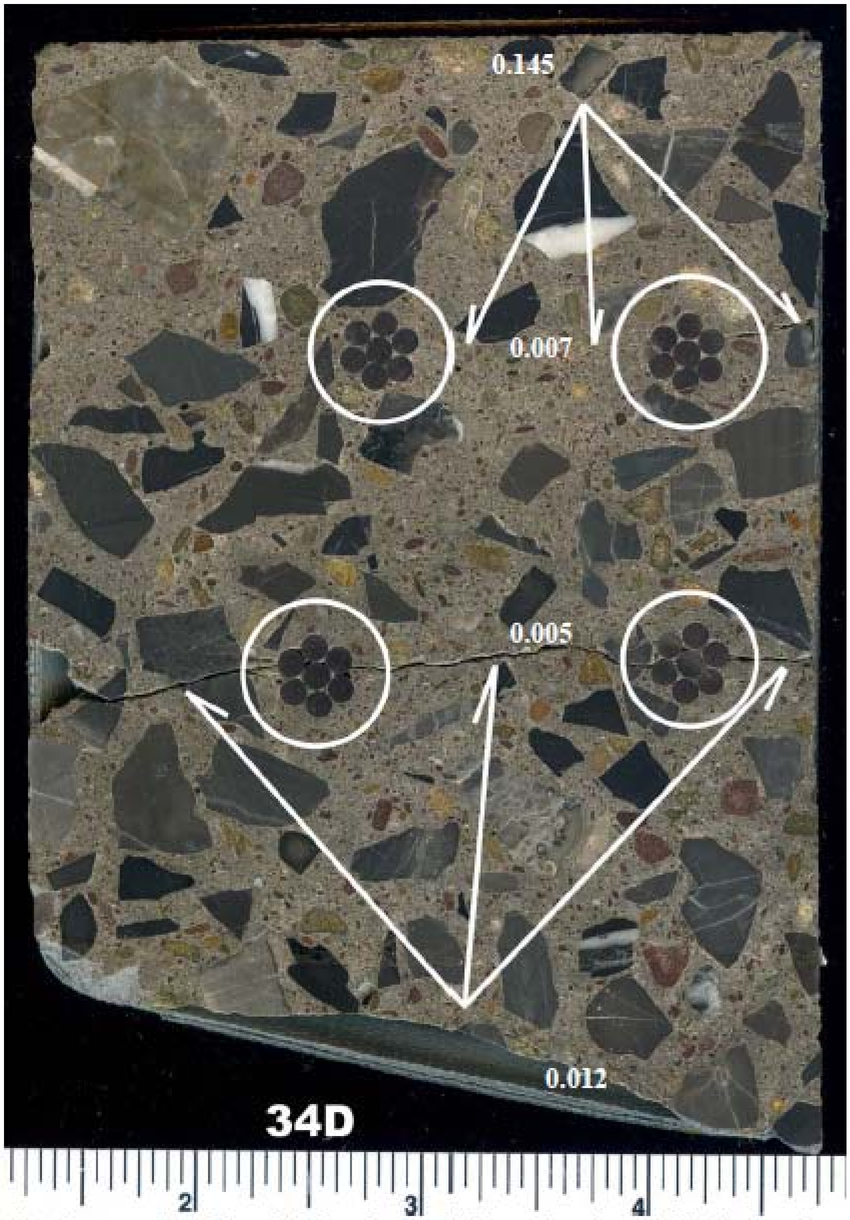


Figure 4-16: Petrographic Examination of Core 34D in Beam CC4

4.2.2.2 Aggregates

The cores contain two different types of coarse, crushed, argillaceous calcareous aggregate having 3/4-inch nominal maximum sizes. Both types are fine-grained, dense, and vug-free (no cavities). Coarse aggregate of Cores 14D and 34D is dark medium grey to mainly dark grey dolomitic limestone frequently finely fractured with the fractures healed by white, vein-type dolomite, and sometimes having a brecciated texture. Coarse aggregate of Cores 3D, 10D, 17D, 23D, and 27D is medium to dark grey, and occasionally medium yellow-brown calcitic limestone having a 3/4-inch nominal maximum size. The medium grey particles usually contain significant amounts of detrital quartz. The coarse aggregate types identified correspond to the bridges as illustrated in Table 4-4. The presence of Dolomitic or Calcitic Limestone should not have a significant effect on the strength or permeability of the concrete.

Table 4-4: Coarse Aggregate Types	
Bridge Beams:	Coarse Aggregate Type:
Clearfield Creek	Dolomitic Limestone
Main Street and Lakeview Drive	Calcitic Limestone

Fine aggregates are similar natural sand that contains major amounts of clear to translucent grey, translucent orange single and multi-crystal quartz, feldspars, and lesser amounts granite, weathered granite, sandstone, shale, coal, mafic minerals, volcanics, and magnetite. The fine aggregate of Cores 14D and 34D are somewhat coarser than fine aggregate in the other cores and contain greater amounts of quartz and feldspars. The aggregates have been chemically and physically sound during their service in the concrete. Aggregate content by core can be found in Table 4-5.

4.2.2.3 Pastes

The pastes of cores 3D, 10D, 17D, 23D, and 34D are similar except for color tones. Pastes are deep buff with a grey overtone, dense, hard, firm, and fracture surfaces have semi-conchoidal textures. Residual and relict Portland cement particles are abundant, hydration of the cement is advanced, and the calcium hydroxide (Ca(OH)2) component of cement hydration occurs as patchy and platy crystals. Residual and relict cement particles have sufficiently fine sizes to indicate that cement having a high fineness (e.g. Type III) was used, and mineralogical composition of residual cement particles is indicative of Types I/II Portland cement. Features of the pastes are indicative of 6 1/2 to 7 bags of Portland cement per cubic yard. Estimated water-cement ratios are variable between Cores 3D, 14D, 17D, 27D and 34D from 0.38 to 0.43, and variable within Core 10D from 0.47 to 0.54 and Core 23D from 0.43 to 0.51. The variability in Cores 10D and 23D is due to failures to thoroughly intermix the batch and/or tempering water. These cores correspond to MS2 and LV16, respectively. The variation in w/c ratio may result in a variation in mechanical properties within the beam.

Paste of Cores 14D and 34D are deep brown with a grey overtone, dense, hard, firm, and fracture surfaces have semi-conchoidal textures. Residual and relict Portland cement particles are abundant, hydration of the cement is not advanced, the calcium hydroxide (Ca(OH)2) component of cement hydration occurs as patchy and platy crystals, residual and relict cement particles have sufficiently fine sizes to indicate cement having a high fineness (e.g. Type III) was used, and mineralogical composition of residual cement particles is indicative of Types I/II Portland cement. Features of the pastes are indicative of 6 1/2 to 7 bags of Portland cement per cubic yard, and estimated water-cement ratios are 0.38. Paste content can be found in Table 4-5.

4.2.2.4 Air Void Analyses

Air in Cores 3D, 14D, and 34D occurs as a handful of small spherical voids characteristic of entrained air voids, and mainly as coarse spherical and non-spherical voids characteristic of entrapped air. The cores are very poorly air-entrained (if air-entrained at all) and have respective determined air contents of 5.1, 3.4, and 2.7 percent. Specific surfaces are low (350 to 550 in^2/in^3), and void spacing factors are very high (0.012 to 0.018 in.).

In the remaining cores, air contents are from 2.7 to 8.1 percent, specific surfaces are from 440 to 855 in^2/in^3, and void spacing factors are from 0.004 to 0.009 in. They all contain many small, discrete, spherical voids characteristic of entrained air voids; however, in Cores 10D and 27D entrained air voids are erratically distributed.

None of the cores meet industry requirements for the combination of air content, specific surface, and void spacing factor needed to protect critically saturated concrete from damage by cyclic freezing and deicing chemicals. Specific values for air content, specific surface, and void spacing factors which fail to meet industry requirements are highlighted in red in the table to follow. The summary of the aforementioned data is presented below in Table 4-5. Based on the measured hardened air analyses, all beams are prone to freeze-thaw damage in the presence of de-icing salts. The inspection of the beams, however, did not indicate any freeze thaw damage on the bottom flange surface. Consequently, while the hardened air quality is not satisfactory it is unlikely to influence the occurrence of corrosion.

Table 4-5: Air Void Analyses Data Summary

Beam:	**Core ID:**	**Air Content: (%):**	**Specific Surface: (in^2/in^3)**	**Void Spacing Factor: (in)**	**Paste Content: (%)**	**Aggregate Content: (%)**
MS2	10D	6.5	**530**	0.008	27.6	65.9
MS3	3D	5.1	**350**	**0.014**	30.3	64.5
CC3	14D	3.4	**355**	**0.018**	34.1	62.5
CC4	34D	**2.7**	**550**	**0.012**	32.3	65.0
LV19	17D	**8.1**	835	0.004	26.0	65.9
LV16	23D	7.0	**440**	**0.009**	26.4	66.7
LV7	27D	**2.7**	855	0.007	25.9	71.4
Industry Requirements		6 +/- 1.5	≥ 600	≤ 0.008	-	-

4.2.2.5 Carbonation Evaluation

Depths of carbonation at the exterior ends of the cores were determined using freshly fractured surfaces. The surfaces are coated with Phenolphthalein and regions that do not turn purple are indicative of carbonation. Depths of carbonation are from nil to 3/32 inch, except for Core 10D (beam MS2) where it is continuous to a depth of 5/8 inch and then sporadic to the level of the top strands, a depth of 1 1/4 inches. The depths of carbonation are summarized in Table 4-6.

Table 4-6: Summary of Carbonation Evaluation for Petrography Cores					
Beam ID:	Core ID:	Estimated *w/cm* ratio:	Estimated Cement Content: (bg/yd^3)	Carbonation Depth: [in.]	Cracks?
MS3	3D	0.43	6.5 to 7	3/32	Level of Top Strand
MS2	10D	0.47-0.54	6.5 to 7	5/8 and sporadic to 1.25	Level of Top Strand
CC3	14D	0.38	6.5 to 7	Nil	Level of Top & Bottom Strands
LV19	17D	0.41	6.5 to 7	1/64	None
LV16	23D	0.43-0.51	6.5 to 7	Nil to 1/8	None
LV7	27D	0.41	6.5 to 7	Nil to 1/64	None
CC4	34D	0.38	6.5 to 7	Nil	Level of Top & Bottom Strands

4.2.2.6 Strand Evaluation

Seven-wire prestressing strands having 3/8-inch diameters are present in Cores 3D, 10D, 14D, 17D, 23D, and 34D. Individual wire diameters are 1/8 inch. The degree of corrosion ranges from essentially no corrosion to heavy corrosion, as shown in Figure 4-17.

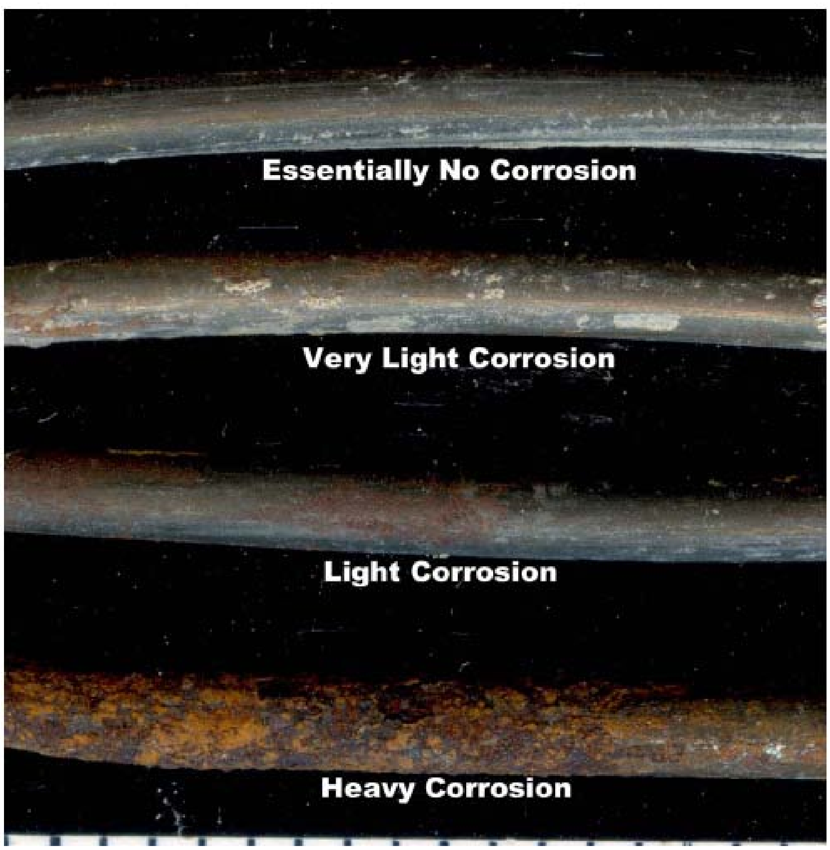


Figure 4-17: Degrees of Corrosion by TEC

The extent of strand corrosion for each core is presented along with chloride content at the strand level in Table 4-7. The chloride data is also shown above on the Petrographic cross-sections of the cores. Not shown in Figure 4-11 is the severely corroded strands from Core 10D (Beam MS2), wherein significant wire cross-sections were corroded; this was the only case for severely corroded top strands.

Table 4-7: Seven Wire Strand Corrosion Evaluation

Beam ID:	Core ID:	"Degree" of Corrosion: Top Strands / Bottom Strands	Corresponding Chloride Content (%) of Concrete Mass:	Cracks at Strand Level?
MS3	3D	Essentially None / Essentially None	0.004 / <0.004	Top Strands
MS2	10D	Very Severe / Very Light	0.059 / 0.019	Top Strands
CC3	14D	None to Very Light / None to Very Light	0.006 / 0.005	Top & Bottom Strands
LV19	17D	Essentially None / None to Very Light	<0.004 / < 0.004	None
LV16	23D	None to Very Light / None to Very Light	0.024 / 0.017	None
CC4	34D	None / None to Very Light	0.007 / 0.005	Top & Bottom Strands

4.2.2.7 Secondary Compounds

Secondary very fine, white, acicular ettringite ($3CaO{\cdot}Al_2O3{\cdot}\ 3CaSO_4{\cdot}2H_2O$) is occasionally present in a random void as a partial deposit on the void surface. Secondary very fine calcium hydroxide ($Ca(OH)_2$) is occasionally present in a random void as a partial deposit on the void surface. These compounds are innocuous and are a result of exposure of the concrete to moisture; they can be commonly found in outdoor and exposed concrete structures. In this particular case-study, they have no consequence on concrete performance and only signal the concrete has been exposed to moisture.

4.2.3 Chloride Analyses

For the four inch nominal diameter cores, chloride contents were determined for the top and bottom one-half inch and each strand level – or assumed strand levels for Core 27D where there are no strands – of the cores used for the petrographic examinations. For the 5/8 inch diameter concrete plugs taken from beams CC4 and LV19, the whole sample was analyzed because the lengths were very short. The data for the four inch diameter cores can be found in Table 4-8. The data for the 5/8 inch diameter cores can be found in Table 4-9.

The data from Table 4-8 shows the chloride by percent mass of concrete at different depths from each core. If the value is highlighted in red, it has exceeded the ACI threshold value of 0.026% by mass of concrete. The data shows a trend that the chlorides are highest at the bottom surface of the beam – top surface of the core – and that they decrease as you go deeper into the flange toward the void. The only exceptions to that are the cores taken from beams MS2 and LV7. Beam LV7 had a large delamination in the bottom flange; this may have allowed direct chloride penetration into the strands at that depth. Beam MS2 had half of the concrete bottom flange surface spalled; the petrography core also indicated large depths of carbonation. These indicators may have led to increased chlorides within the flange.

Comparing the chloride levels from the multiple cores taken from each beam indicates that the chloride level is sensitive to the beam type and location on the beam. Main Street beams 2 and 3 were located adjacent to each other but the chloride levels vary significantly. MS2 consistently had low surface chloride levels but high chloride levels at the strand depth for the three cores examined. MS3 however had high surface chloride levels but low chloride levels at the strand depth for 3 of the 4 cores.

Two cores from Lakeview Drive beam 7 had higher chloride levels on the interior, indicating intrusion from the inside of the beam. The third core indicated chloride intrusion from the outside of the beam. This variation between cores indicates that the beam was subjected to chlorides from both entrapped water within the beam void and from runoff over the exterior surface.

Table 4-8: Chloride Analyses for All 3 ¾ inch Cores

Core ID:	Chloride, % by Mass of Concrete for Depth Shown:			
	Top (in)	At 1st Steel (in)	At 2nd Steel (in)	Bottom (in)
MS2 8B	0 – 1/2	1 1/4 – 1 3/4	2 1/2 – 3	-
	<0.004	0.271	0.072	-
MS2 10D	0 – 1/2	1 – 1 1/2	2 1/2 – 3	4 3/4 – 5 1/4
	0.011	0.059	0.019	0.009
MS2 35C	0 – 1/2	1 – 1 1/2	2 3/4 – 3 1/4	-
	0.026	0.082	<0.004	-
MS3 2B	0 – 1/2	3/4 – 1 1/4	1 3/4 – 2 1/4	-
	0.277	0.005	<0.004	-
MS3 3D	0 – 1/2	3/4 – 1 1/4	2 – 2 1/2	5 – 6 (2)
	0.227	0.004	<0.004	0.004
MS3 4C	0 – 1/2	1 – 1 1/2	2 1/4 – 2 3/4	-
	0.439	0.007	<0.004	-
MS3 5A	0 – 1/2	1 – 1 1/2	2 1/2 – 3	-
	0.296	0.157	0.023	-
CC3 12C	0 – 1/2	1 1/2 – 2	2 3/4 – 3 1/4	-
	0.231	<0.004	<0.004	-
CC3 13B	0 – 1/2	1 1/4 – 1 3/4	2 3/4 – 3 1/4	-
	0.097	0.005	0.005	-
CC3 14D	0 – 1/2	1 1/2 – 2	3 – 3 1/2	6 1/2 – 7 1/2 (2)
	0.075	0.006	0.005	0.006
CC3 16A	0 – 1/2	1 1/2 – 2	2 1/2 – 3	-
	0.209	0.020	0.012	-

Table 4-8: Chloride Analyses for All 3 ¾ inch Cores				
Core ID:	Chloride, % by Mass of Concrete for Depth Shown:			
	Top (in)	At 1st Steel (in)	At 2nd Steel (in)	Bottom (in)
CC4 31A	$0 - \frac{1}{2}$	$1\frac{1}{4} - 1\frac{3}{4}$	$3 - 3\frac{1}{2}$	-
	0.086	0.016	0.006	-
CC4 33B	$0 - \frac{1}{2}$	$1\frac{1}{4} - 1\frac{3}{4}$	$2\frac{3}{4} - 3\frac{1}{4}$	-
	0.040	0.005	0.005	-
CC4 34D	$0 - \frac{1}{2}$	$1\frac{1}{4} - 1\frac{3}{4}$	$2\frac{1}{2} - 3$	$4 - 4\frac{1}{2}$
	0.145	0.007	0.005	0.012
LV7 27D	$0 - \frac{1}{2}$	$1 - 1\frac{1}{2}$	$2\frac{1}{2} - 3$	$3\frac{1}{2} - 4$
	0.168	0.011	0.004	0.004
LV7 28A	$0 - \frac{1}{2}$	$1 - 1\frac{1}{2}$	$1\frac{1}{2} - 2$ (3)	-
	0.059	0.061	0.089	-
LV7 30B	$0 - \frac{1}{2}$	$1 - 1\frac{1}{2}$	$2\frac{1}{2} - 3$	-
	0.088	0.177	0.233	-
LV16 22C	$0 - \frac{1}{2}$	$1 - 1\frac{1}{2}$	$2\frac{1}{2} - 3$	-
	0.029	0.023	0.012	-
LV16 23D	$0 - \frac{1}{2}$	$1 - 1\frac{1}{2}$	$2\frac{1}{2} - 3$	$4 - 4\frac{1}{2}$
	0.086	0.024	0.017	0.019
LV16 25B	$0 - \frac{1}{2}$	$1 - 1\frac{1}{2}$	$2\frac{1}{2} - 3$	-
	0.078	0.023	0.024	-
LV19 17D	$0 - \frac{1}{2}$	$\frac{3}{4} - 1\frac{1}{4}$	$2\frac{1}{2} - 3$	$6 - 7$ (2)
	0.303	<0.004	<0.004	<0.004
LV19 18C	$0 - \frac{1}{2}$	$\frac{3}{4} - 1\frac{1}{4}$	$2 - 2\frac{1}{2}$	-
	0.145	0.092	<0.004	-
LV19 20B	$0 - \frac{1}{2}$	$\frac{3}{4} - 1\frac{1}{4}$	$1\frac{3}{4} - 2\frac{1}{4}$	-
	0.466	0.101	0.102	-
LV19 21B	$0 - \frac{1}{2}$	$\frac{3}{4} - 1\frac{1}{4}$	$2\frac{1}{4} - 2\frac{3}{4}$	-
	0.404	0.171	0.093	-

The data from Table 4-9 shows the chloride percent by mass of concrete for the 5/8 inch diameter cores taken from the beam surface. The data indicates that chloride levels are higher than the ACI threshold of 0.026 percent in 16 out of the 18 samples. The average chloride value for beam LV19 (0.275) is higher than that for beam CC4 (0.071).

Table 4-9: Chloride Analyses for 5/8 inch Cores			
Core #:	Core ID:	Mass (g):	Chloride, % by Mass of Concrete:
1	CC4-1	4.5	0.183
2	CC4-2	4.7	0.040
3	CC4-3	4.4	**0.020**
4	CC4-4	4.0	0.094
5	CC4-5	5.8	0.109
6	CC4-6	6.1	**0.011**
7	CC4-7	3.5	0.084
8	CC4-8	3.5	0.080
9	CC4-9	6.5	0.049
10	LV19-1	5.9	0.188
11	LV19-2	6.3	0.394
12	LV19-3	6.6	0.206
13	LV19-4	6.8	0.209
14	LV19-5	2.6	0.316
15	LV19-6	5.6	0.425
16	LV19-7	5.6	0.217
17	LV19-8	7.5	0.244
18	LV19-9	5.4	0.280

4.3 Concrete Compressive Strength

A minimum of three strength cores – 2 in. nominal diameter – were extracted from each beam so that tests could be performed to establish the in-situ compression strength of the concrete, f'_c. These strengths were then compared to the minimum required compression strength specified in the design drawings, listed in Table 4-10.

Table 4-10: Concrete Minimum Required Compression Strength		
Bridge:	**Beam Types:**	**f'_c [psi]:**
Clearfield Creek	Beams 3 and 4	5000
Lakeview Drive	Beams 16 and 19	5900
	Beam 7	5000
Main Street	Beams 2 and 3	5600

The cores were tested in accordance with ASTM C39; “Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens.” The cores were tested in a computer-controlled SATEC universal testing machine. The results are as shown in Table 4-11:

Table 4-11: Strength Core Results									
Core #	Beam	L [in.]	L/D	Correction Factor[3]	f'_c [psi]	Avg. f'_c [psi]	COV	Type of Fracture	Req'd f'_c [psi]
1	MS3	3.47	2.02	1	8080	7728	16%	TYPE 3	5600
1'		3.24	1.88	1	8742			TYPE 1	
1"		3.24	1.88	1	6361			TYPE 2	
9	MS2	3.47	2.02	1	6111	5436	12%	TYPE 1	5600
9'		3.53	2.05	1	**4871**			TYPE 2	
9"		3.53	2.05	1	**5327**			TYPE 2	
15	CC3	3.47	2.02	1	6919	8192	15%	TYPE 2	5000
15'		2.89	1.67	0.974	8262			TYPE 1	
15"		2.89	1.67	0.974	9393			TYPE 1	
19	LV 19	3.47	2.02	1	**5514**	5263	10%	TYPE 2	5900
19'		2.88	1.67	0.974	**5598**			TYPE 2	
19"		2.88	1.67	0.974	**4676**			TYPE 1	
24	LV 16	3.47	2.02	1	8438	7914	20%	TYPE 2	5900
24'		3.54	2.06	1	10227			TYPE 1	
24"		3.54	2.06	1	6555			TYPE 1/4	
24""		3.24	1.88	1	6219			TYPE 2	
24"""		3.32	1.93	1	8133			TYPE 2	
29	LV7	2.88	1.67	0.974	6286	5917	7%	TYPE 3	5000
29'		2.4	1.4	0.947	6020			TYPE 1	
29"		2.62	1.53	0.962	5444			TYPE 2	
32	CC4	3.47	2.02	1	5968	7834	15%	TYPE 3	5000
32'		2.88	1.67	0.974	9117			TYPE 2	
32"		2.88	1.67	0.974	8413			TYPE 2	
32"'		3.54	2.06	1	7703			TYPE 3	
32""		3.54	2.06	1	7969			TYPE 2	

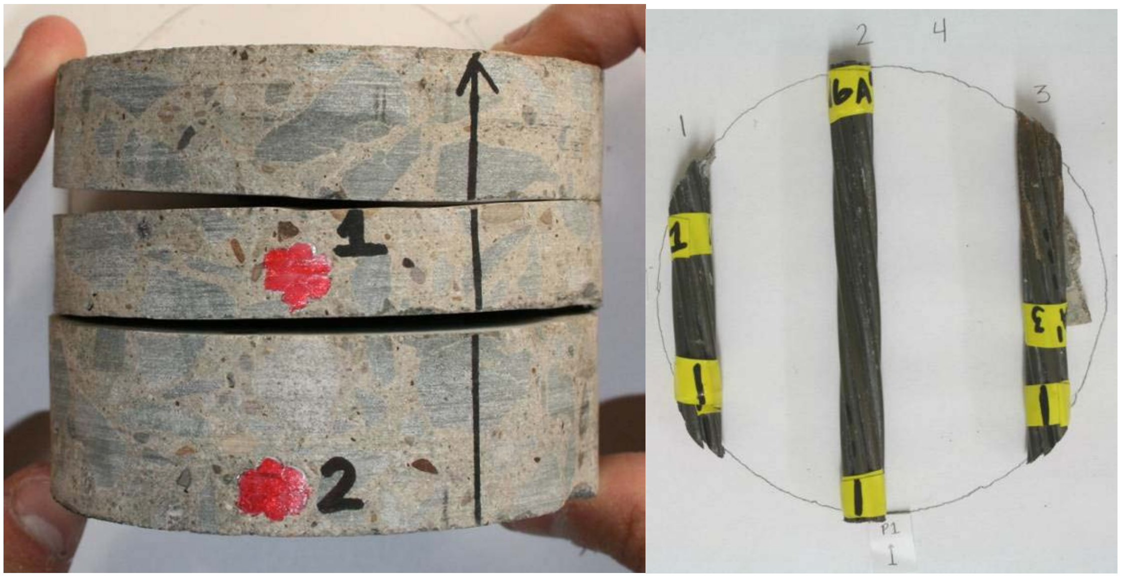
Notes:

1. *L = length of core sample*
2. *D = diameter of core sample*
3. *Correction factor per ASTM C39*
4. *Type of fracture per ASTM C39*

As shown in the table, the core samples of Beams MS2 and LV19 had compressive strengths less than the required strength indicated on the design drawings; as shown in bold font. In the case of beam LV19, all of the cores tested exhibited inadequate strength. All other beams exceeded their design compressive strength.

4.4 Strand Extraction

To examine if a correlation exists between strand corrosion on the first and second layer of reinforcement, a series of cores were taken where the strands within each core were extracted and inspected for corrosive damage. Figure 4-18 (a) shows a sliced core before extraction; the strands were numbered prior to extraction. The slices of the concrete containing the strands were then chiseled and hammered to remove the strand(s). Figure 4-18 (b) shows the final results from strand extraction.



(a) Before Strand Removal (b) After Strand Removal

Figure 4-18: Removal of Strands from 4 inch Cores

The recorded data is presented below in Table 4-12. Listed strand IDs are for strands at the 2nd level of reinforcement. The damage for the corresponding strand at the 1st level of reinforcement is also given; allowing for a comparison of the damage index at these two levels of steel reinforcement. The concept of a damage indices are discussed more thoroughly in Section 5.1. The correlation between damage at the 1st and 2nd levels is compared in detail in Chapter 7.

Table 4-12: Recorded Data for Extracted Strands

Beam ID	Core ID	Strand ID	Strand Damage Index	Damage Index for Strand at 1st Level of Reinforcement
CC3	12C'	3	0	0
	12C’	4	0	0
	13B'	1	1	N/A
	16A'	4	0	3
CC 4	31A'	3	0	1
	31A’	4	1	1
	33B'	2	0	0
LV 7	30B'	1	1	N/A
LV 16	22C'	4	1	1
	22C’	5	1	1
	22C’	6	1	1
	25B'	3	0	0
	25B'	4	0	0

Table 4-12: Recorded Data for Extracted Strands				
Beam ID	Core ID	Strand ID	Strand Damage Index	Damage Index for Strand at 1st Level of Reinforcement
LV 19	18C'	4	0	0
	18C'	5	1	1
	18C'	6	1	2
	21B'	1	3	N/A
	21B'	2	1	N/A
	21B'	3	1	N/A
MS2	8B'	1	0	N/A
	8B'	2	1	N/A
	35C'	1	0	N/A
MS3	2B'	3	0	1
	2B'	4	2	3
	4C'	2	0	1
	5A'	2	1	1

4.5 Core Summary

Cores with nominal diameters of 1 in., 2 in., and 4 in. were extracted from the bottom flange of each box beam. Seven cores – one from reach beam – were examined petrographically, analyzed for their air-void properties, and analyzed for chlorides at surface regions and strand levels by TEC. Seventeen cores were analyzed for chlorides in surface regions and strand levels by TEC. Eighteen small cores (1 inch diameter) were analyzed for surface chlorides by TEC. The following is a highlight of the conclusions made from the core evaluation.

- Six of the seven petrography cores are poorly air-entrained; only one core has air void properties that meet industry requirements for concrete that will be protected when critically saturated from damage by cyclic freezing and deicing chemicals. Visual inspection of the beams, however, did not indicate any freeze thaw damage.
- Aggregates are chemically and physically sound.
- Corrosion of strand wires is nil to light, except for Core 10D (Beam MS2) where top strands are severely corroded.
- Carbonation was very shallow among cores except for Core 10D (Beam MS2); the carbonation depth of Core 10D is 5/8 inch and sporadic to depths of 1 1/4 inch.
- The base-level chloride content – due to chlorides from the concrete-making components is from less than 0.004 percent to 0.006 percent by concrete mass. On that basis, chlorides from the environment have infiltrated all but two of the concretes analyzed.
- Chloride contents at top strand levels of 14 of the 24 cores analyzed exceed the American Concrete Institute (ACI) corrosion threshold of 0.013 percent. Chloride contents at second strand levels of 9 of the 24 cores analyzed exceed the ACI threshold.

- Chloride levels varied within each beam and between adjacent beams. A high reading at one region of the beam does not ensure that the elevated level will exist at all regions of the beam.
- Of the 18 surface cores analyzed from Lakeview Drive beam 19 and Clearfield Creek beam 4, 17 exceed the ACI threshold. The surface chloride content on LV19 was over 20 times the ACI corrosion threshold.
- Horizontal cracks are present in 4 of the 7 cores examined petrographically. In Cores 3D (MS3) and 10D (MS2) cracks are in-line with top strands; in Cores 14D (CC3) and 34D (CC4) cracks are in-line with the top and second strands. It could not be determined if these cracks formed during extraction or if they existed in-situ.
- Forensic evaluation of cores removed from MS2 indicates that corrosion is likely present in the beam. The estimated water-cement ratio varied from 0.47 to 0.54 indicating a failure to thoroughly intermix batch and/or tempering water. The compressive strength of the beam had a large standard deviation and failed to meet the required design strength. The air voids in the concrete were erratically distributed. The core had a depth of carbonation of up to 1.25 in. from the exterior surface. The chloride content at the first level of strand exceeded the ACI corrosion limit. Consequently, the exterior layer of strands had very severe corrosion and concrete spalling.
- Based on the chloride levels measured, corrosion is likely at the first level of strands on all beams. Corrosion is likely on the second level of strands for all Lakeview Drive beams, and both Main Street beams. The chloride content at the second layer of strands was low for Clearfield Creek beams.

5 Subsurface Investigation of Prestressed Strands

The concrete cover on the bottom flange of each beam was removed in order to expose and examine the condition of the lower level of strands. Incidences of corrosion damage were documented along the length of each strand. The observed corrosion was categorized into five levels of damage. These data points were then plotted to scale with a color-code indicating the level of damage, resulting in a comprehensive damage profile for each beam. This data was be used to validate which, if any, of the NDT methods adequately detect corrosion in these types of members.

5.1 Strand Visual Inspection

Every strand within each of the beam specimens was exposed along its entire length, with the exception of several strands from Lakeview Drive Beam #7. A tape measure was setup at one end of each beam and extended along the length of the member to gage the exact distance of a particular damage location relative to the pier end. In addition, at each location of corrosion damage, the number of strands damaged and the severity of damage was recorded. Five levels of corrosion damage were considered in the assessment, as well as a strand condition with no corrosion damage. These levels were (0) no corrosion, (1) light corrosion, (2) pitting, (3) heavy pitting, (4) wire loss, and (5) wire fracture; as shown in Figure 5-1. Photographs were taken of all cases of significant corrosion damage (heavy pitting or worse). The damage levels are also referred to as Damage Indices (DI) in this report. Descriptions of each damage level follow:

- DI = 0. No Corrosion. No visible indications of corrosion product on the surface of the strand.
- DI = 1. Light Corrosion. Light corrosion product visible on the surface of the strand removable by light sanding with no associated section loss.
- DI = 2. Pitting. Corrosion present with section loss between 0 and 20% of the wire section area.
- DI = 3. Heavy Pitting. Corrosion present with section loss greater than 20% of the wire section area.
- DI = 4. Wire Loss. Corrosion present and complete 100% degradation of individual strand wires.
- DI = 5. Fracture. Corrosion present and localized fracture of individual wires.

It is of importance to note that damage was not documented where concrete had spalled and strands were exposed or missing.

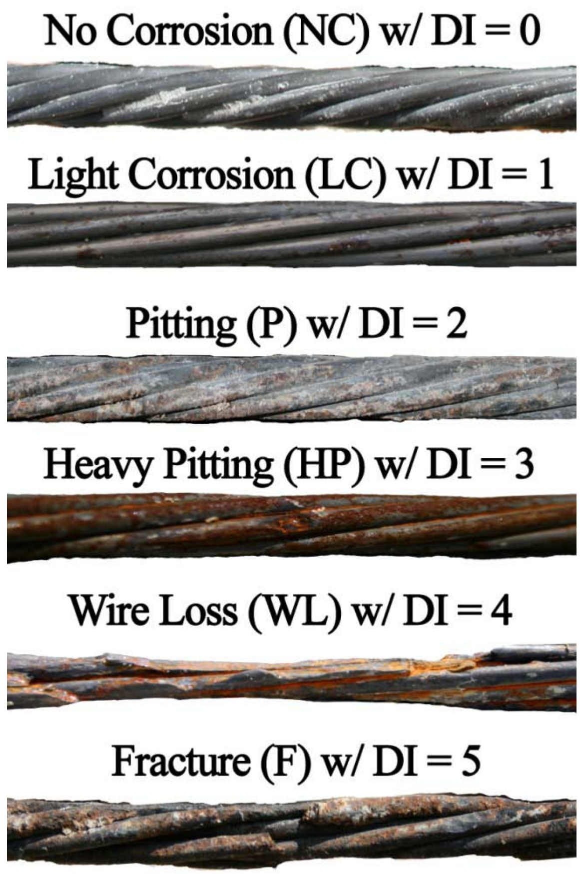


Figure 5-1: Various Strand Damage Conditions

5.2 Damage Profiles

The visual inspection data was then compiled and overlaid on the photos to produce a damage profile for each beam. The legend of corrosion damage levels is summarized in Figure 5-2. The damage conditions occurred either at specific points or over various lengths along each strand. Consequently these damage locations are illustrated as points and/or lines on the damage profile overlays that follow. In all cases the strands are numbered starting at 1 at the top of the image.

Figure 5-2: Legend to Identify the Different Types of Corrosive Damage

5.2.1 Clearfield Creek Bridge Beam #3

The damage profile for Clearfield Creek Beam 3 is presented in Figure 5-3. As shown, most of the damage is focused on strands seven, eight, and nine; starting from the A1 end. Many cases of heavy and light pitting can be found, as well as two cases of wire loss. Towards the other pier end (P1), damage in the form of heavy pitting can be found on strand ten. These incidences of corrosion damage lie under the large crack found during the visual inspection.

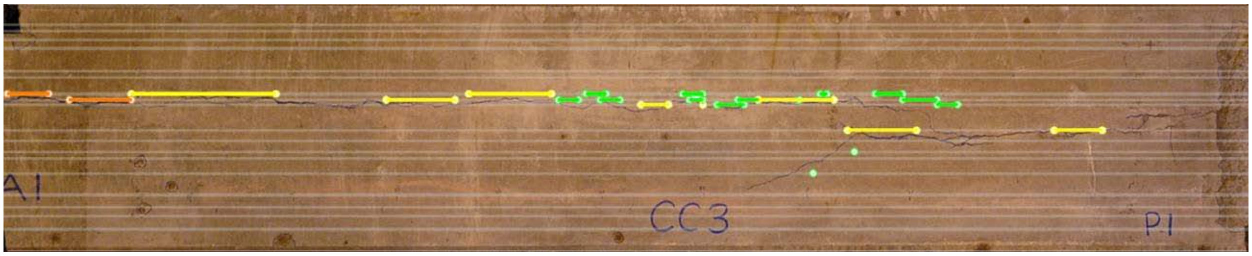


Figure 5-3: Damage Profile for Clearfield Creek Bridge Beam 3

5.2.2 Clearfield Creek Bridge Beam #4

As can be seen in Figure 5-4, this specimen is relatively damage free. Only light corrosion was found on the strands of this beam; with a concentration on strand fifteen. It is important to note the large longitudinal crack discovered during the visual inspection was directly over strand fifteen.

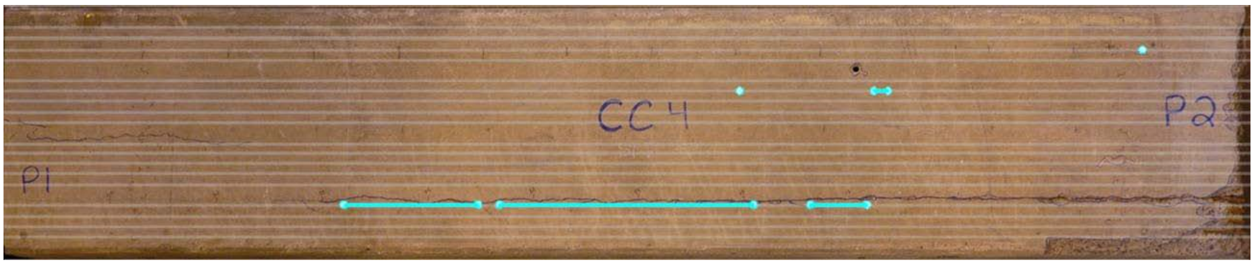


Figure 5-4: Damage Profile for Clearfield Creek Bridge Beam 4

5.2.3 Lakeview Drive Bridge Beam #7

In Figure 5-5, the white box shows the area that was exposed and inspected. Prior to the skinning process, the first three strands were completely missing, the next two were almost fully exposed and certainly corroded, and the final two strands were missing and exposed in spots. Both ends (P1 and A1) had a large amount of concrete spalled, limiting the length of strand that could be exposed and inspected.

The most severe damage – fracture and heavy pitting – can be found closer to the A1 side, on strands 21, 22, and 23; there was a large delamination detected along the bottom edge during the visual inspection. Due to this, no protection was being provided to those outer strands. During the visual inspection, cracks were also found above strands 11/12 and above strands 13/14; as shown, heavy pitting, and pitting occur along those locations.

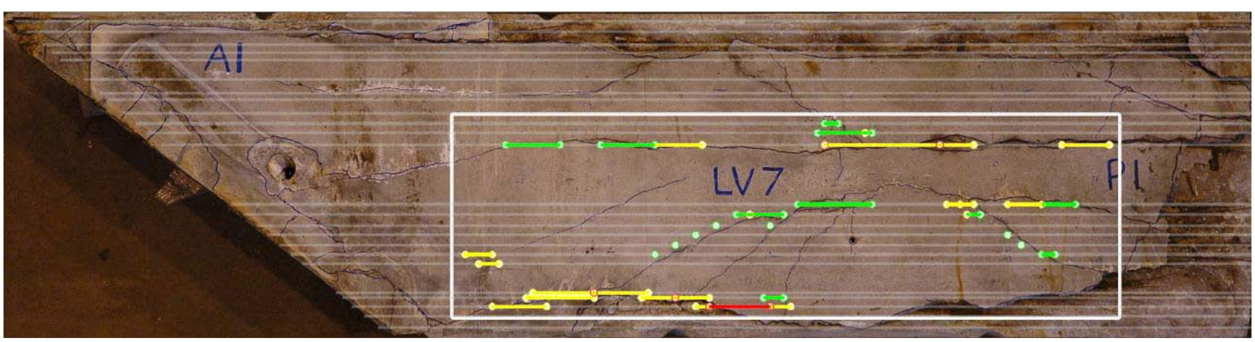


Figure 5-5: Damage Profile for Lakeview Drive Bridge Beam 7

5.2.4 Lakeview Drive Bridge Beam #16

As can be seen in Figure 5-6, most of the damage is focused in one distinct area. When using this data jointly with the visual inspection, it is noticed that the large cluster of damage found just off-center toward the P2 side is near the location of a large spall. Heavy pitting was found on strand three, toward the P1 end of the beam; there was a crack found at this location during the visual inspection. The most severe damage is the wire loss found on strand ten; where there were no surface indicators of corrosion.

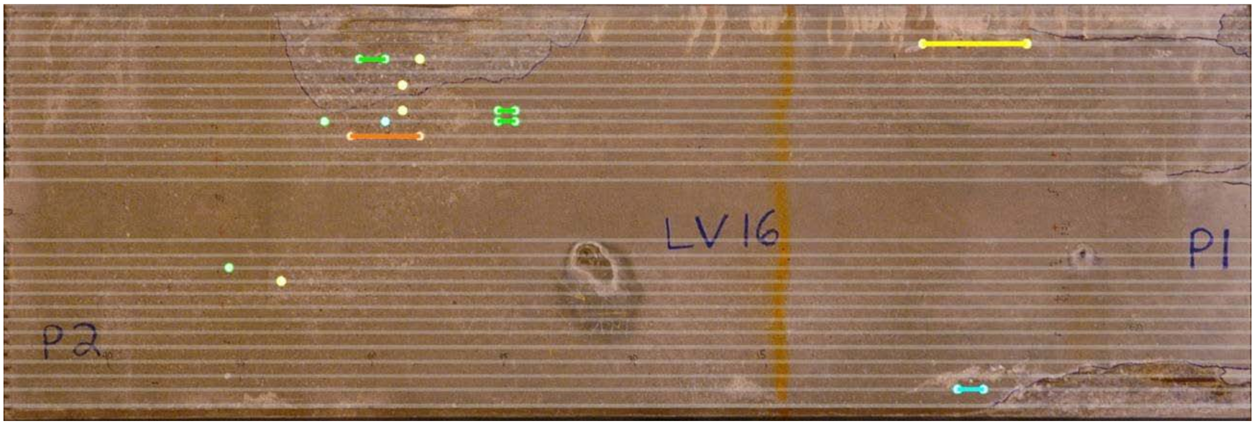


Figure 5-6: Damage Profile for Lakeview Drive Bridge Beam 16

5.2.5 Lakeview Drive Bridge Beam #19

As can be seen in Figure 5-7, most of the damage can be found on the upper half of the specimen. The visual inspection showed several cracks along this area. Even though there appears to be much damage on the beam, most of it is of the lighter nature; light corrosion and pitting. Wire loss was documented on strands six, seven, eight, and nine. The lower half of the beam is mostly undamaged, showing several spots with light corrosion damage and some isolated spots with heavy pitting and pitting.

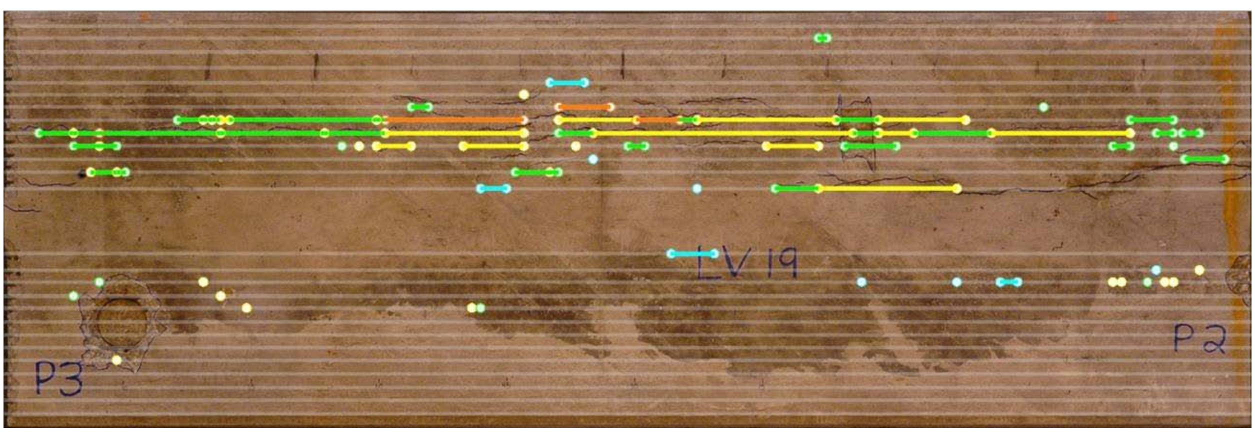


Figure 5-7: Damage Profile for Lakeview Drive Bridge Beam 19

5.2.6 Main Street Bridge Beam #2

The damage profile for beam MS2 is shown in Figure 5-8; it is important to note that the top half of the beam was almost completely spalled off. This limited the amount of inspecting and exposing done on the top half of the beam. The white box in the figure below indicates the area of the beam inspected; with limited locations inspected along the top half.

The bottom half of the beam had severe corrosion damage throughout its length. There was some rust staining detected during the visual inspection on the bottom half, as well as a small spalled area along the bottom of the beam. However, there was really no visual indication of the severe amount of damage found after exposure and inspection. Wire loss or fracture was detected on each of the strands in the bottom half of the beam except strand 15.

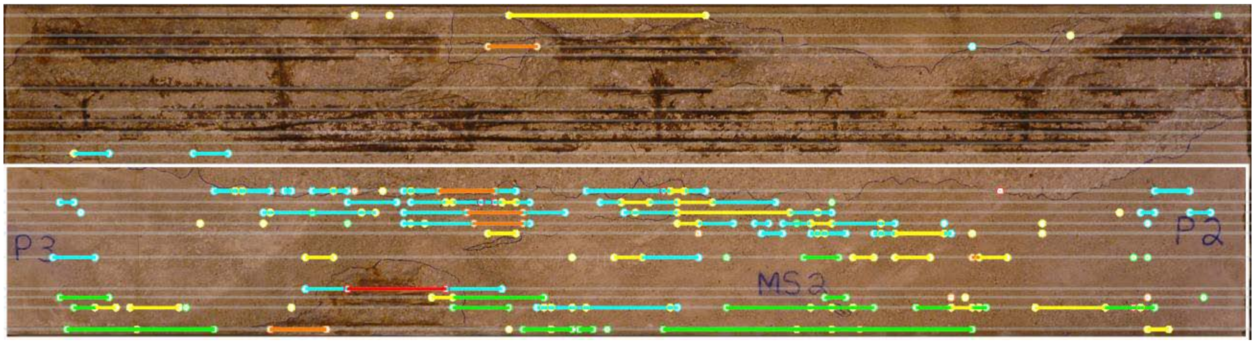


Figure 5-8: Damage Profile for Main Street Bridge Beam 2

5.2.7 Main Street Bridge Beam #3

As can be seen in Figure 5-9, the majority of the corrosive strand damage is present within the top half of the beam. During the visual inspection, a documented large crack was present along the beam length in this area. The major cases of corrosion documented for this specimen lie under that crack. Wire loss was detected on strands 6, 7, 8, and 9 and wire fracture was detected on strand 10. Only minor corrosion damage was detected in other areas away from the crack.

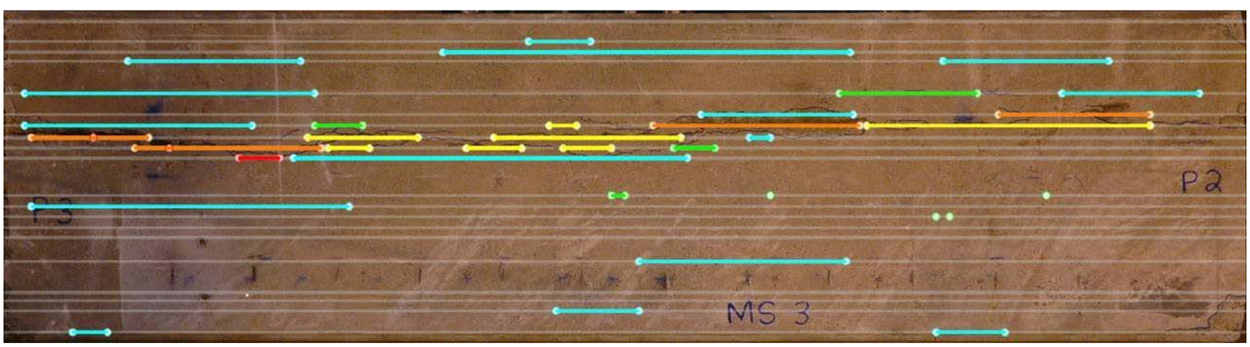


Figure 5-9: Damage Profile for Main Street Bridge Beam 3

5.3 Exposure of the 2nd Level of Strands

At several spots throughout each beam, after the exposure of the first layer of strands had taken place, locations were determined where information about the second layer of strands was desired. The acquired data will aide in determining the relationship between the two levels of strands. A jackhammer was taken and punched through the remaining concrete in the bottom flange of the girder to fully reveal the 2nd level. For a before and after the concrete was punched-out image, see Figure 5-10.

Figure 5-10: Before and After Deconstruction

These punch-outs were 12 in. long by 7 to 13.5 in. wide. A minimum of two punch outs were taken for each beam. Additional information can be found in Table 5-1. The location of the bottom-left corner of each punch-out is referenced relative to the lower left corner of each beam.

Table 5-1: 2nd Level Concrete Inspection Region Locations

Beam	**Punch out ID**	**Dimensions width*height (in)**	**X,Y (in)**	**Ref. Pier End**	**Top/Bot. Corner**
MS3	1	12x12	24,25	P3	Bottom
	2	12x10	96,25	P3	Bottom
	3	12x10	144,25	P3	Bottom
MS2	4	12x8	24,26	P3	Bottom
	5	12x8	60,14	P3	Bottom
	6	12x8	108,14	P3	Bottom
LV19	7	12x10	48,26.5	P3	Bottom
	8	12x11	96,26.5	P3	Bottom
LV16	9	12x11.5	36,26.5	P2	Bottom
	10	12x11.5	96,26.5	P2	Bottom
LV7	11	12x13.5	112,27	P1	Bottom
	12	12x13.5	64,27	P1	Bottom
CC4	13	12x7	36,2	P1	Bottom
	14	12x7	84,2	P1	Bottom
CC3	15	12x9	12,21.75	A1	Bottom
	16	12x9	72,21.75	A1	Bottom

The above data was used to create images of the punch outs superimposed on the beam surface; the punch outs are represented as the white rectangles in the figures to follow. The exposed 2nd level of strands had a visual inspection performed on them to assess the location of corrosion. The exposed 2nd level strands are shown as the shaded white lines if no damage was found or color coded to the same scale as shown in Figure 5-2 if damage was found.

As shown in Figure 5-11, two punch outs were taken for beam CC3. Punch-out 15 is located near the longitudinal crack; the strand closest to the crack has corrosion damage in the form of pitting. Punch out 16 is located near the longitudinal crack; the three strands inspected exhibited no signs of corrosion. It is important to note that the 2nd level of strands has an alternate layout than the 1st level of strands; therefore, the relationship to the 1st level of strands cannot be determined.

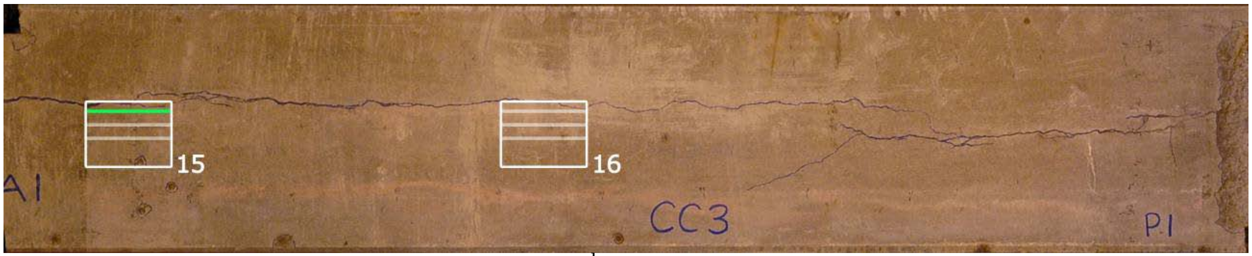


Figure 5-11: Location of 2nd Layer Punch outs on Beam CC3

As shown in Figure 5-12, two punch outs were taken for beam CC4. Punch out 13 is located at an area under the crack; no damage was observed in the exposed strands. Punch out 14 is located at an area under the crack; light corrosion was found on strand 15.

Figure 5-12: Location of 2nd Layer Punch outs on Beam CC4

As shown in Figure 5-13, two punch outs were taken for beam LV7. Punch out 11 is located at an area with two longitudinal cracks; all five inspected strands showed evidence of corrosion which ranged from pitting to fracture. Punch out 12 is located at a longitudinal crack where two strands were detected to have wire loss. It is important to note that the 2nd level of strands has an alternate layout than the 1st level of strands, limiting the comparison of the 1st and 2nd level of strands.

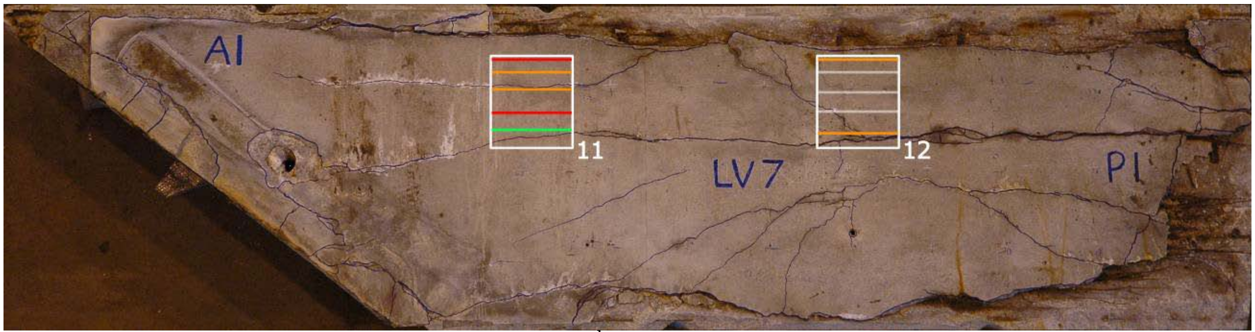


Figure 5-13: Location of 2nd Layer Punchouts on Beam LV7

As shown in Figure 5-14, two punch outs were taken for beam LV16. Punch out 9 is located near a spalled area where strand 10 had severe wire loss on the 1st strand level; the exposed 2nd strand level revealed heavy pitting on strand 10. Punch out 10 is taken over an area where no damage was found on the 1st level of strands; the exposed 2nd strand level revealed no corrosion damage.

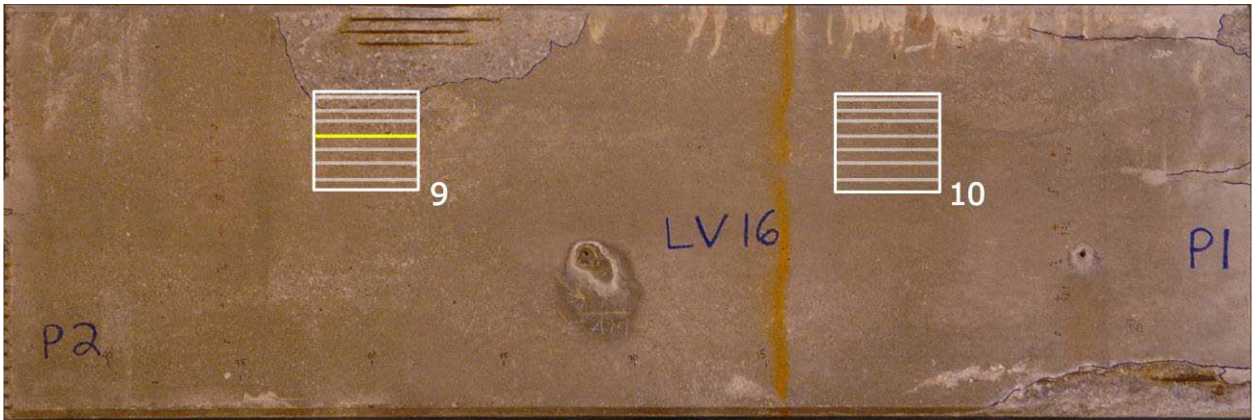


Figure 5-14: Location of 2nd Layer Punchouts on Beam LV16

As shown in Figure 5-15, two punch outs were taken for beam LV19. Punch out 7 is located along a longitudinal crack; exposed 2nd level strands revealed one case of pitting. Punch out 8 is located along a longitudinal crack; 2nd strand layer visual inspection revealed one case of wire loss.

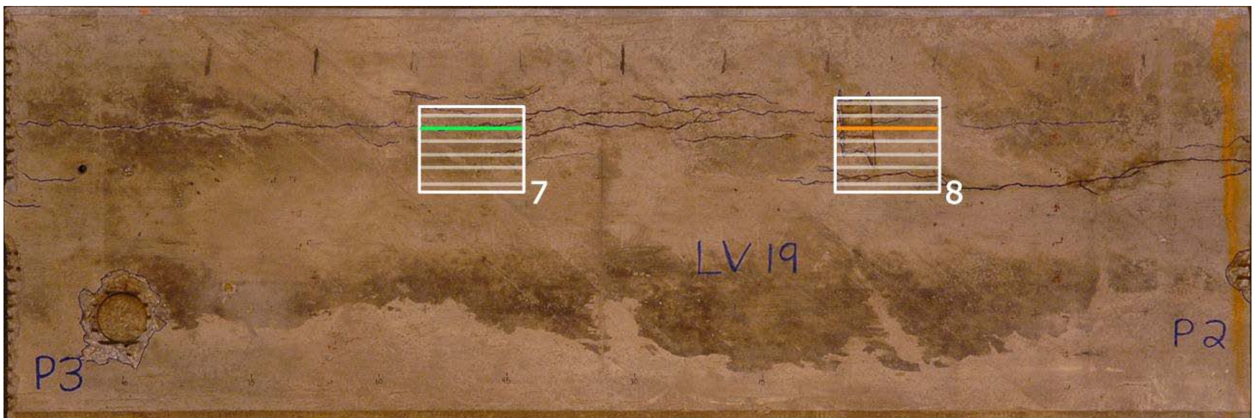


Figure 5-15: Location of 2nd Layer Punch outs on Beam LV19

As shown in Figure 5-16, three punch outs were taken for beam MS2. Punch out 4 is located above the spalled section on the beam; of the four exposed 2nd layer strands uncovered, two wires had fractured, one was heavily pitted, and the remaining one was clean. Punch outs 5 and 6 revealed completely clean strands at the exposed 2nd level.

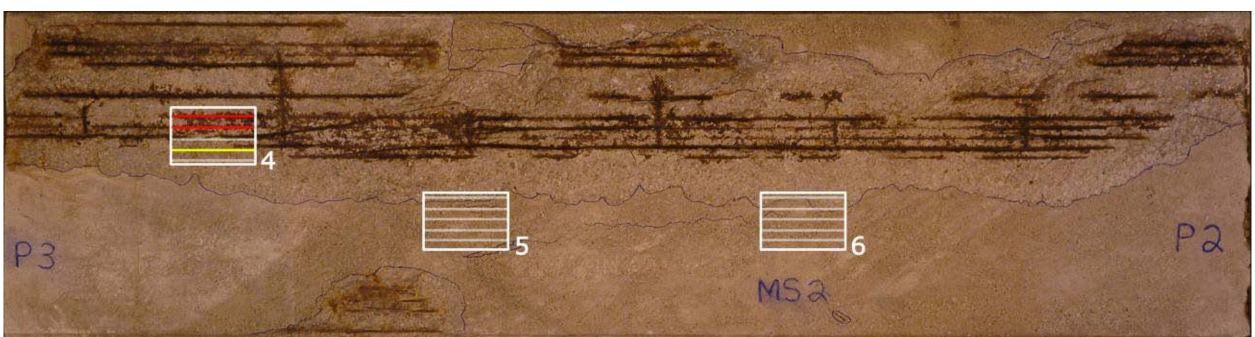


Figure 5-16: Location of 2nd Layer Punch outs on Beam MS2

As shown in Figure 5-17, three punch outs were taken for beam MS3. Punch outs 1, 2, and 3 were taken along the longitudinal crack in areas where severe wire loss was present at the 1st layer of strands. All three exposed sections revealed heavy corrosion damage – heavy pitting, wire loss, and fracture – at the second level of steel.

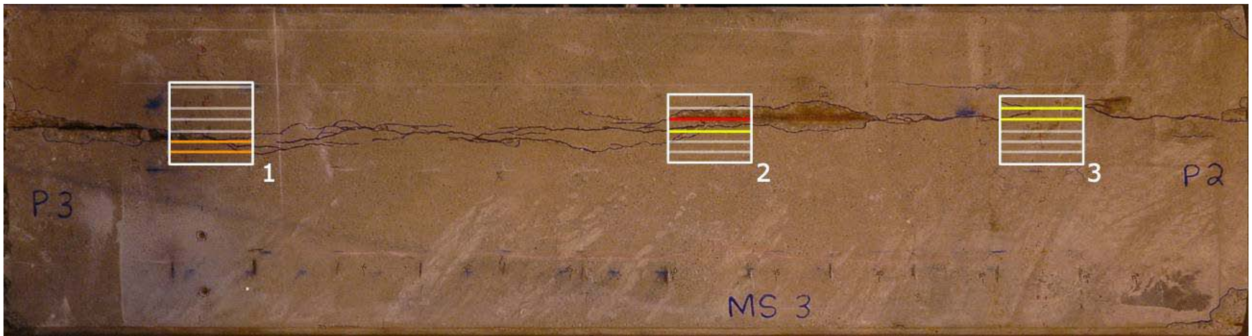


Figure 5-17: Location of 2nd Layer Punch outs on Beam MS3

5.4 Summary of Subsurface Investigation of Prestressing Strands

The general condition of the strands and the methodology used to determine the condition were presented in this chapter. The results of the visual inspection of the strands are presented qualitatively. Detailed analyses of the results are discussed in section 7. The results presented in this section are as follows:

- The subsurface strand condition varied over 6 levels. Undamaged, light corrosion, pitting, heavy pitting, wire loss, and fracture. These conditions were defined on a damage index from 0 to 5, with 0 being undamaged.
- Inspection of the first level of strands indicates that corrosion damage is typically associated with longitudinal crack locations.
- Inspection of the second level of strands indicates that corrosion damage is also typically associated with longitudinal crack locations.

6 Half-Cell Potential Case Study

An effective method for measuring the presence of corrosion relies on the electrochemical process to determine areas of active corrosion. The method has been standardized as ASTM C876: “Standard Test Method for Half-Cell Potentials of Uncoated Reinforcing Steel in Concrete.” This method has been developed and used extensively for concrete reinforced with conventional reinforcing bars. The procedure involves measuring the voltage differential between an external half cell electrode and the embedded steel. The half cell electrode is composed of copper/copper sulfate (CSE), silver/silver chloride (SCE), or Mercury/Mercury Chloride.

For conventional reinforcing steel in concrete, a voltage potential of less than -350mV (CSE) is a strong indicator of corrosion activity. This value is not definitive for prestressing steels, for large concrete covers, or for concretes with certain constituents. Consequently, it is recommended that a map of the potential of the beam be developed and that corrosion activity be identified by looking at large relative changes in potential over the surface.

The Half Cell Potential mapping was conducted using a copper electrode in a copper sulfate solution (termed CSE or $Cu/CuSO_4$). A prefabricated $Cu/CuSO_4$ probe produced by Elcometer was used along with a Fluke digital multimeter – which can be seen in Figure 6-1 – to measure the voltage potentials.

Figure 6-1: Half-Cell Electrode with Multimeter

It is important to note that for some prestressed concrete beam applications, the individual steel strands may be isolated from each other. For these conditions it is important to ground the half cell to each respective strand being investigated. Due to the damage caused by removing concrete cover in order to achieve an electrical connection to each strand, this approach may not be feasible for beams whose strands are electrically isolated. However, since the sample beams obtained for this project are fully cross-sectioned, all strands are exposed at the ends. Therefore, potential mapping was conducted relative to each strand in the bottom layer.

6.1 Potential Mapping Procedure

Each strand had to be partially exposed – approximately 0.5 in. – at one end to make a connection with the multi-meter (see Figure 6-2). The connection with the multi-meter is illustrated in Figure 6-3.

Figure 6-2: Exposed Strands at Sectioned Face of Beam

Figure 6-3: Electrical Connection between Multi-meter and Strand.

The half cell potential was conducted only on the layer of reinforcement that was closest to the bottom flange surface. Consequently, only the bottom row of strands was exposed at the end.

To ensure good electrical continuity, the end of each exposed strand was cleaned with a wire brush. This eliminated any rust contamination that may have been present. After the strands were cleaned, the secondary portion of the testing was to determine whether the concrete surface was adequately moist. To accomplish this, the following procedure was employed as recommended in ASTM C876. The multi-meter (set to measure DC voltage) was attached to an exposed strand and the half-cell electrode was placed at a point on the concrete surface somewhere along that strand. If the reading was a near-constant value, then the concrete was deemed to be adequately moist. However, if the reading consistently changed and was not repeatable then the concrete was too dry and moistening was required prior to taking the potential measurement.

6.2 Procedure for Wetting Concrete Surface

As noted in ASTM C876, the best way to wet the concrete surface to obtain the most accurate results is to use a solution consisting of 95 mL of commercial wetting agent or a liquid household detergent mixed with 5 gallons of potable water. This procedure was not followed due to concern that the presence of the water-detergent mixture on the concrete surface would have adverse effects on the non-destructive testing procedures to be performed subsequently.

In lieu of using the water-detergent mixture, two different methods were employed to wet the concrete surface. When the half cell measurements were taken for the Clearfield Creek and Main Street beams, the beams were sitting on blocks in their normal orientation. To wet the bottom surface a sprinkler was placed below the beam and set to oscillate along the length of the beam. This provided a near-constant stream of water on the beam for 2-3 hours before performing the half cell measurements. After thoroughly wetting the surface, the multi-meter was placed on the concrete for five minutes to ensure that the reading was now near constant. If a constant reading was observed, the half cell potential mapping continued.

The beam segments for Lake View Drive were soaked differently because they were brought into the lab inverted; now the bottom beam flanges were facing up. This facilitated the soaking of the beams without having to spray water on them. Instead, pieces of burlap were thoroughly soaked in water and placed on the bottom of the beam as shown in Figure 6-4. A long plastic tarp was then used to cover the burlap to prevent evaporation; see Figure 6-5. The burlap was left on the beam for roughly 2-3 hours to ensure the concrete was sufficiently saturated. When measurements were taken on these beams, the burlap was folded back progressively to only uncover a single strand at a time and therefore prevent the beam from drying out.

Figure 6-4: Burlap Soaking the Bottom of the Beam

Figure 6-5: Plastic Tarp Covering the Beam During the Moistening Procedure

After the beam surface was highly saturated, the half-cell testing commenced. A string was marked in one foot increments to use as a reference and simplify the data collection process. The string was then stretched along the beam directly over a single strand.

To ensure adequate wetting of the electrode, a wet sponge was wrapped around the negative end. The positive electrode of the half-cell apparatus was attached to the exposed strand while the negative electrode was placed at each marking on the string in succession. At each marking, the half-cell potential was measured and recorded. After completing the measurements for one strand, the positive electrode was moved to the next exposed strand and the measurements continued.

6.3 Interpretation of Results

These half-cell potential measurements were subsequently plotted as a color contour plot. An example of the plot for one beam segment is shown in Figure 6-6. It is important to note that the scale used in the following figure is used throughout the rest of this report. Dark blue through green shading represents a very small probability of corrosion, yellow through light orange shading represents an uncertain probability for corrosion, and dark orange through dark red shading is indicative of a very high probability of corrosion. The half-cell potential readings are discussed in more detail below.

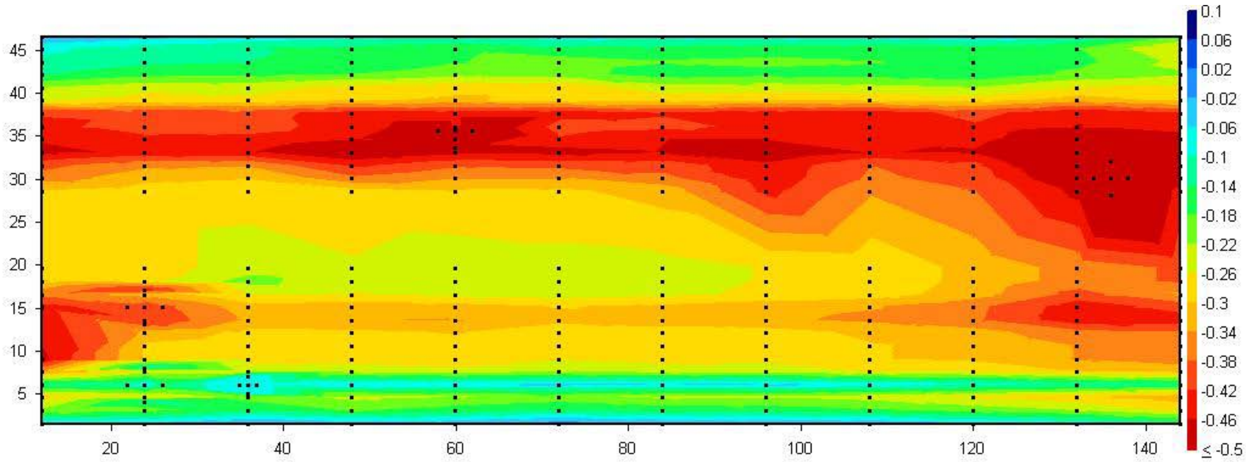


Figure 6-6: Half-Cell Potential Map of Lakeview Drive Span 3 Beam 19

As noted in ASTM C876, areas with half-cell potential values more positive than -0.2 V have a 90% probability that there is no corrosion in the reinforcing steel. Areas with values between -0.2 V and -0.35 V have an uncertain probability for corrosion. Finally, areas that have readings more negative than -0.35 V have a greater than 90% probability that corrosion exists. This description is summarized in Table 6-1:

Table 6-1: Probability of Corrosion for Regular Reinforcing Steel (per ASTM C876)

Half-Cell Potential	**Probability of Corrosion**
Greater than -0.20 V	10%
Between -0.20 V and -0.35 V	Uncertain
Less than -0.35 V	90%

These potential ranges were developed specifically for conventional reinforcing steel. Their accuracy with respect to prestressing strand is not available and will be examined in this study. Contour plots indicate relative changes in potential that give a good indication that corrosion is more likely to be initiating in certain regions over others.

6.4 Clearfield Creek Bridge Members

6.4.1 Clearfield Creek Span 1 Beam 3 Potential Mapping

The half-cell potential map, given in Figure 6-7, shows an increased propensity for corrosion along the crack, as well as in the area of the lower left corner of the beam (A1 side). These readings indicate that there may or may not be corrosion in those particular areas, illustrated through yellow shading. The readings indicate that in all other areas corrosion is not likely.

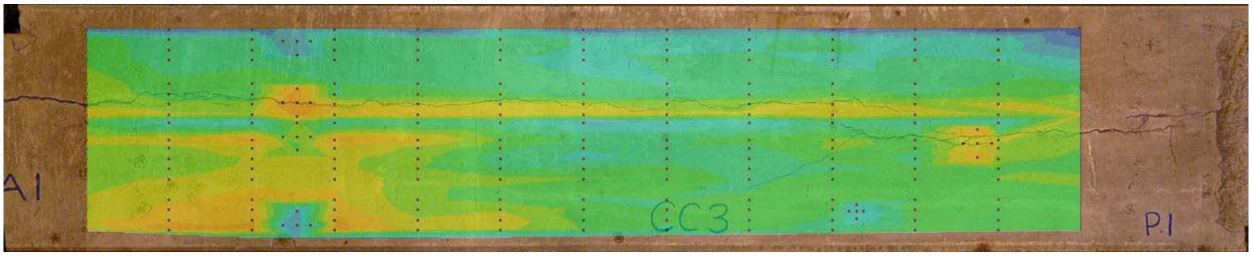


Figure 6-7: Bottom Flange Half-Cell Potential Map - Clearfield Creek Beam 3

6.4.2 Clearfield Creek Span 2 Beam 4 Potential Mapping

The half-cell potential map, as given in Figure 6-8, shows susceptibility for corrosion along the longitudinal crack. These readings insinuate that there may or may not be corrosion in that particular area, indicated by the yellow shaded region. The readings indicate that in all other areas corrosion is not likely.

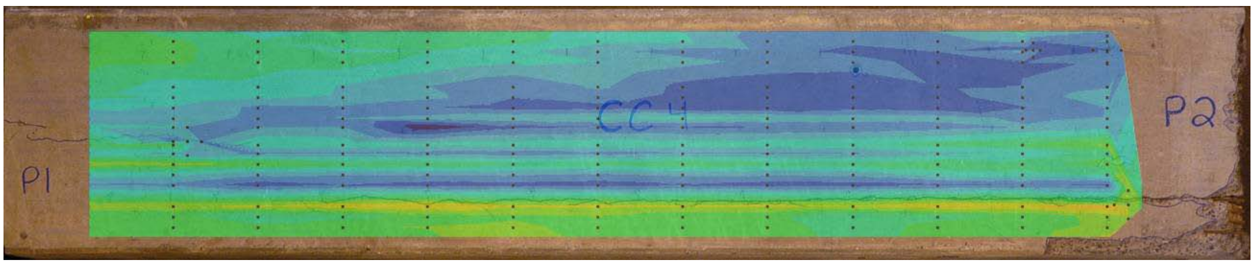


Figure 6-8: Bottom Flange Half-Cell Potential Map - Clearfield Creek Beam 4

6.5 Lakeview Drive Bridge Members

6.5.1 Lakeview Drive Span 1 Beam 7 Potential Mapping

The half-cell potential map, given in Figure 6-9, shows that the A1 side of the beam seems to have less corrosion than the P1 side of the member. The potential map indicates a strong likelihood of corrosion in the P1 area indicated by the red shaded regions.

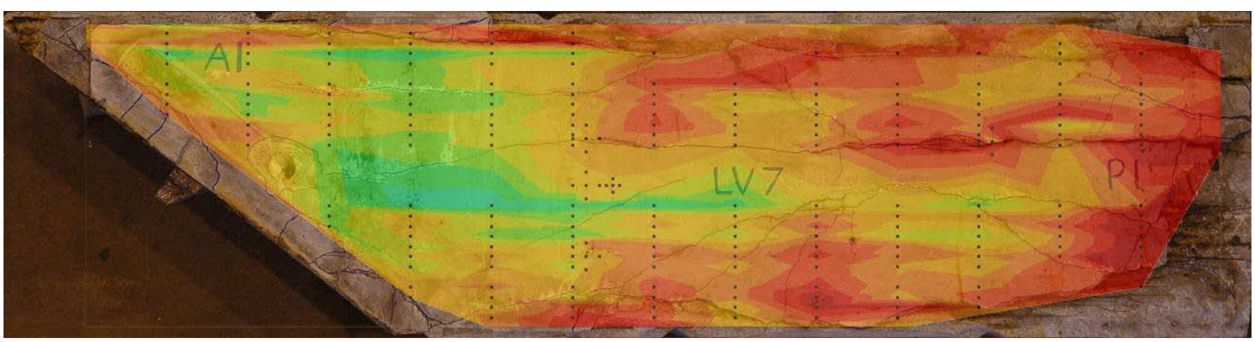


Figure 6-9: Bottom Flange Half-Cell Potential Map – Lakeview Drive Beam 7

6.5.2 Lakeview Drive Span 2 Beam 16 Potential Mapping

The half-cell potential map, as given in Figure 6-10, shows a band where corrosion is possible (indicated by yellow shading) above strands 25 and 26 near the bottom of the figure. The P1 side of the beam in this band seems to have an increased likelihood for corroded strands. The spalled sections have half cell measurements indicating a high likelihood for corrosion potential. This has been verified through the visual inspection.

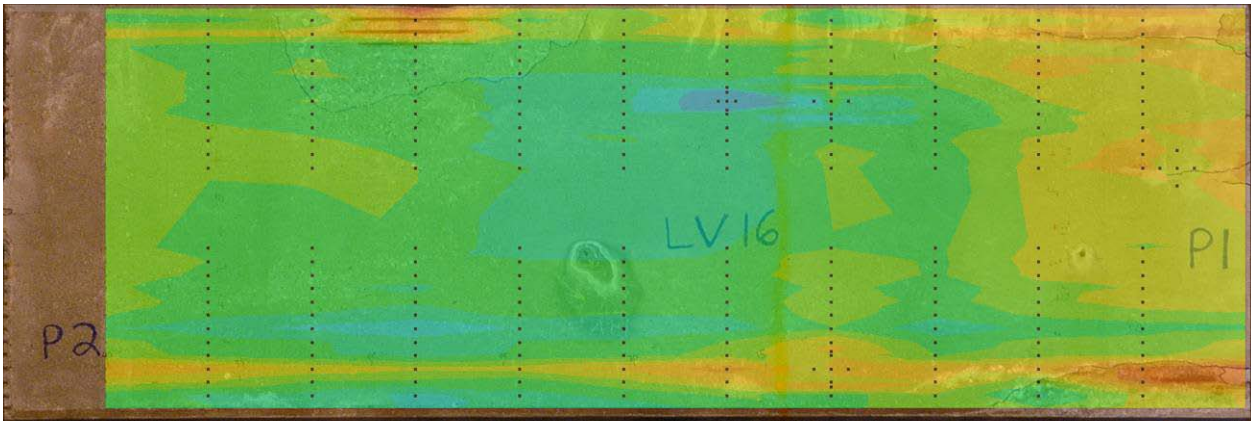


Figure 6-10: Bottom Flange Half-Cell Potential Map -- Lakeview Drive Beam 16

6.5.3 Lakeview Drive Span 3 Beam 19 Potential Mapping

The half-cell potential map, as given in Figure 6-11, indicates a potential for corrosion everywhere except along the top and bottom across the length of the beam. Under the longer crack, extending from the P3 side, corrosion is very likely.

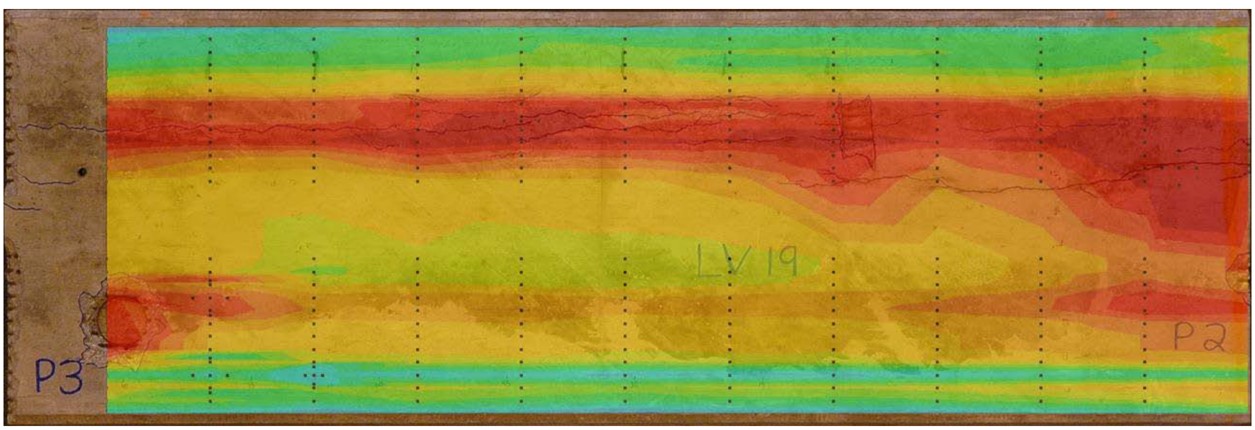


Figure 6-11: Bottom Flange Half-Cell Potential Map – Lakeview Drive Beam 19

6.6 Main Street Bridge Members

6.6.1 Main Street Span 3 Beam 2 Potential Mapping

The half-cell potential map, given in Figure 6-12, indicates a potential for corrosion almost everywhere on the beam. Of course, along the spalled areas the half-cell test gives readings that indicate severe corrosion which again is clearly validated through visual inspection. There also seems to be potential for corrosion under the entire unspalled area.

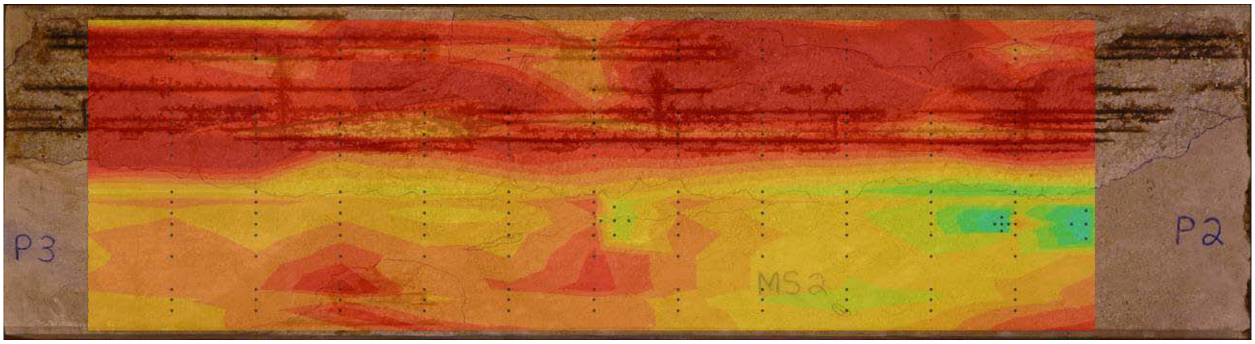


Figure 6-12: Bottom Flange Half-Cell Potential Map – Main Street Beam 2

6.6.2 Main Street Span 3 Beam 3 Potential Mapping

The half-cell potential map, as shown in Figure 6-13, indicates a potential for corrosion everywhere except along the entire top edge of the beam in the figure. It is very likely that corrosion exists under the crack and in that general vicinity, as indicated by the red shaded regions.

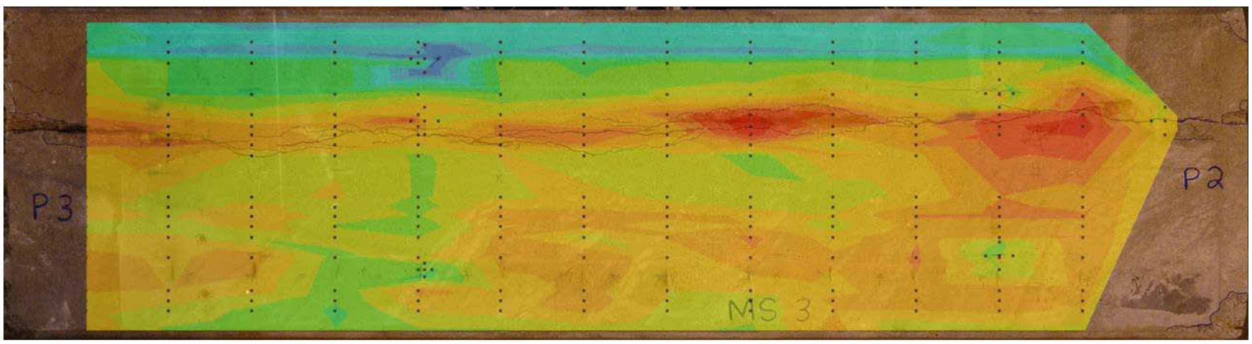


Figure 6-13: Bottom Flange Half-Cell Potential Map - Main Street Beam 3

7 Discussion of Results

This section of the report examines the results of the various tests that have been performed as described in the preceding sections. Conclusions are developed by comparing these results and assessing whether trends exist that can be used to improve the reliability of inspections of prestressed concrete structures. The concrete core evaluations were performed for petrography, concrete cylinder strength, chlorides, air void characteristics, and carbonation. Comparisons are made between the in-situ corrosion levels and the various measurements made to assess the ability to aide in the corrosion detection process of concrete structures. These comparisons are made with respect to half cell measurements and chloride levels. In addition, chlorides and half cell voltage readings are compared with each other to see if there is any correlation between the two. A statistical analysis will be presented in order to assess the viability of using longitudinal cracks to detect corrosion during a visual inspection. Finally, a presentation of the relationship between the 1st and 2nd layer of strands will be discussed.

7.1 Concrete Core Evaluations

Strength testing was performed on the 2 in. nominal diameter cores, in accordance with ASTM C39; refer to Table 4-11 to view those results. Chloride analyses of concrete cores were performed by TEC in accordance with ASTM 1152; refer to Table 4-8 and Table 4-9 to view those results. A more in-depth discussion on the aforementioned items is presented in this section.

7.1.1 Discussion of Concrete Core Strength Results

As the results indicate in Table 4-11, there were large variations in concrete compressive strength throughout some of the beams; the range being from 20% for beam LV16 to 7% for beam LV7. There are several factors that may have affected the results.

One such contributing factor was the diameter of the tested concrete cores. As detailed in ASTM C42, load-bearing structural members shall be at least 3.7 inches in diameter for the determination of compressive strength. Another requirement for concrete cores, as per ASTM C39, is that they must meet an approximate length-to-diameter ratio (L/D) of 2.0. The geometries of the beam (flange thickness) did not permit the minimum diameter of 3.7 inches while still satisfying the L/D ratio; thus, a nominal diameter of 2 inches was chosen. ASTM C39 states that the compressive strength for 2 inch nominal diameter cores is known to be both somewhat lower and more variable than that for 4 inch nominal diameter cores.

Another factor that contributed to the strength variation was the range in water-to-cement (w/cm) ratio detected by TEC. As shown in Table 4-6, two of the seven beams had a variable w/cm ratio; beam MS2 had a range of 0.47 to 0.54 and beam LV16 had a range of 0.43 to 0.51. This is most likely due to a failure to mix properly during batching. The variability in the water to cement ratio could have produced a large variation in strength; such is the case for beam LV16, which had the highest coefficient of variation (COV) for compressive strength. Beam MS2 had a 12% variation in strength. This issue could also have contributed to the reason why beam MS2 had several specimens which did not meet the required strength as indicated on the structural drawings.

All of the strength cores from beam LV19 failed to meet the required strength as indicated on the structural drawings; beam LV19 also had the lowest strength COV at 10%. This is suggestive that the mix for beam LV19 does not have adequate strength.

7.1.2 Discussion of Chloride Analyses

As discussed previously, chloride levels were measured in the bottom flange of the beams. Measurements were taken from cores at the bottom surface, bottom layer of strands, and the second layer of strands. The results were presented in Table 4-8 and Table 4-9. Chloride contents exceeded the American Concrete Institute (ACI) threshold of 0.026% at 9 of the 24 bottom strand levels and at 5 of the 24 second strand levels. Chloride levels were measured at the exterior surface of the beams; 37 of 42 samples exceeded the corrosion threshold.

To assess the accuracy of the ACI threshold limit the measured acid-soluble chloride levels are compared with the in-situ corrosion damage of the strands (i.e., damage index) found at those same locations. There were 43 cases in which to compare the chloride levels with strand damage. In 19% of the cases, strands had chloride values under the ACI threshold but still had some form of damage. In 16% of the cases, strands had chloride values over the ACI threshold but had no corrosion damage. As shown below in Table 7-1, the average chloride readings for all cases with a damage index of 0 (no corrosion) was 0.0113 – under the ACI threshold – compared to 0.0704 for the cases with a damage index greater than 0; which exceeds the ACI threshold.

The standard deviations of the different corrosion levels were large. This is summarized in Table 7-1 and illustrated in Figure 7-1; the large variation was expected. Corrosion is often correlated with chloride levels, but it is also sensitive to the amount of moisture present in service. Since the samples were taken after the beams were taken out of service, the moisture condition of the samples could not be evaluated.

Based on the chloride measurements and corrosion levels determined, the following conclusions can be made: (1) on average the ACI limit of 0.026% divides the region between corrosion and no corrosion, (2) the average chloride readings for all cases with a damage index of 0 (no corrosion) was 0.0113 – under the ACI threshold – compared to 0.0704 for the cases with a damage index greater than 0, (3) the variability in the chloride levels at each corrosion state is very large making determination of chloride thresholds inappropriate for the data set, (4) it is possible to have heavy pitting of the strands with chloride levels of 0.005%, (5) it is possible to have no corrosion with chloride levels of 0.082%.

Table 7-1: Average Chlorides (% by mass of concrete) Based on Strand Damage

Damage Index:	# of Samples	Chloride Average:	Standard Deviation:
0	21	0.0113	0.0183
> 0	22	0.0704	0.0822
1	15	0.0733	0.0920
2	2	0.0465	N/A
3	5	0.0712	0.0671

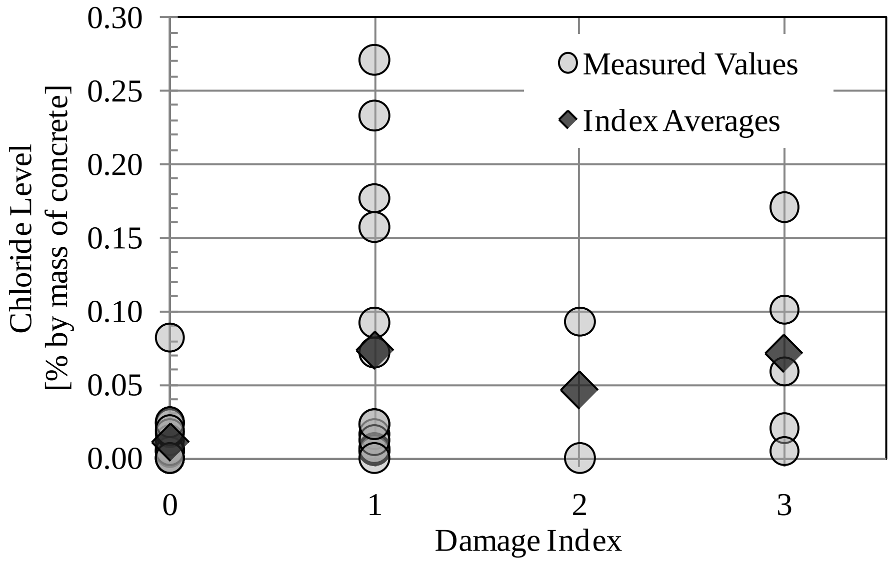


Figure 7-1: Chloride content relative to strand corrosion level

7.2 Half-Cell Profiles

To assess the effectiveness of the half-cell potential method, the strand damage profile has been overlaid on the half-cell contour for each beam. Whether the detected corrosion damage exists within regions of high half-cell potential can be assessed by examination of these composite images. A statistical analysis was performed on the data in order to assess the voltage range where corrosion is likely for prestressed steel.

7.2.1 Half-Cell and Damage Profiles

The damage indices and the half cell readings are compared in detail in this section. The half cell and damage index utilize the same legends as previously used. The legends are reproduced in Figure 7-2 for clarity.

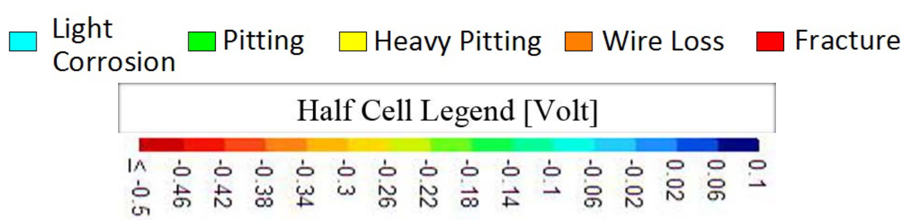


Figure 7-2: Legend of damage index and Half Cell profile

As shown in Figure 7-3, the detected damage found on beam CC3 was primarily under the crack. The half cell potential map clearly indicates the location of lower voltage readings under the crack, indicated by the yellow shaded region that spans from end to end.

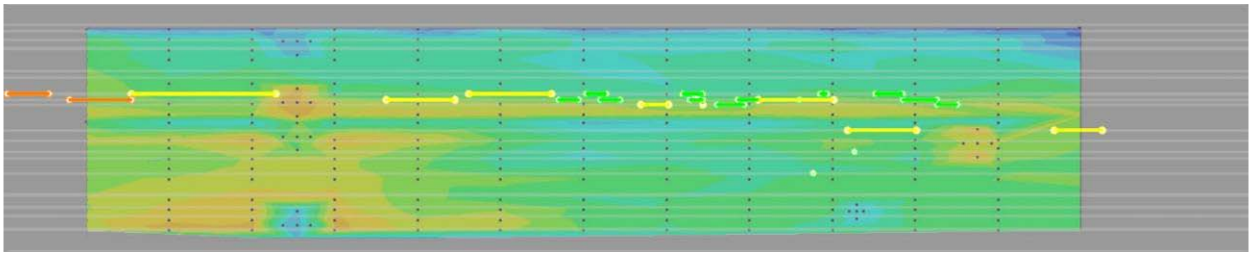


Figure 7-3: Overlay of Damage Profile on Half Cell Potential Map for Beam CC3

As shown in Figure 7-4, the detected damage found on beam CC4 was primarily under the crack. The half cell potential map clearly indicates the location of lower voltage readings under the crack, indicated by the yellow shaded region that spans from end to end.

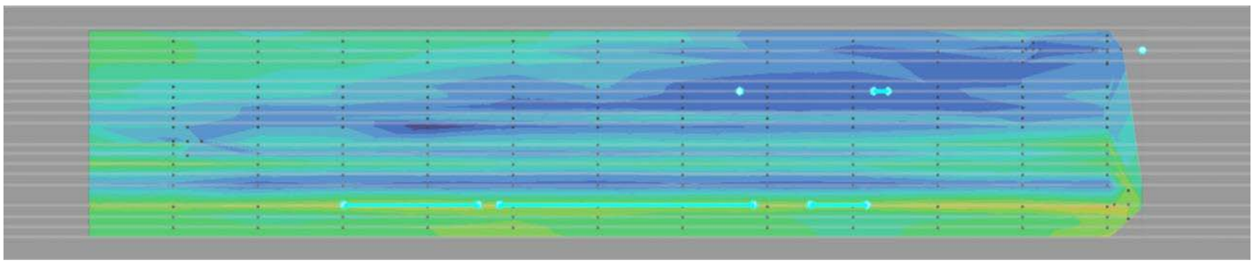


Figure 7-4: Overlay of Damage Profile on Half Cell Potential Map for Beam CC4

As shown in Figure 7-5, the detected damage found on beam LV7 – as shown in the white rectangle in the figure below – was most concentrated at the bottom where the delamination was found. The half-cell readings show high levels of voltage in this area.

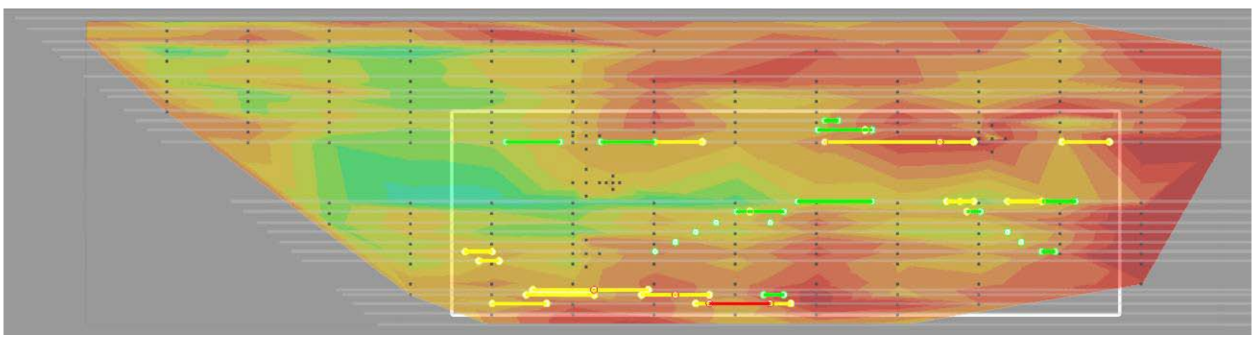


Figure 7-5: Overlay of Damage Profile on Half Cell Potential Map for Beam LV7

As shown in Figure 7-6, the detected damage found on beam LV16 was minimal. The half cell potential map shows an area of low voltage along the bottom of the beam spanning from end to end, however no damage was found in this area. Also, a large amount of wire loss was found on strand 10 (strands numbered starting with 1 at the top of the figure). The high half cell values indicate that it is unlikely that corrosion would exist at that location.

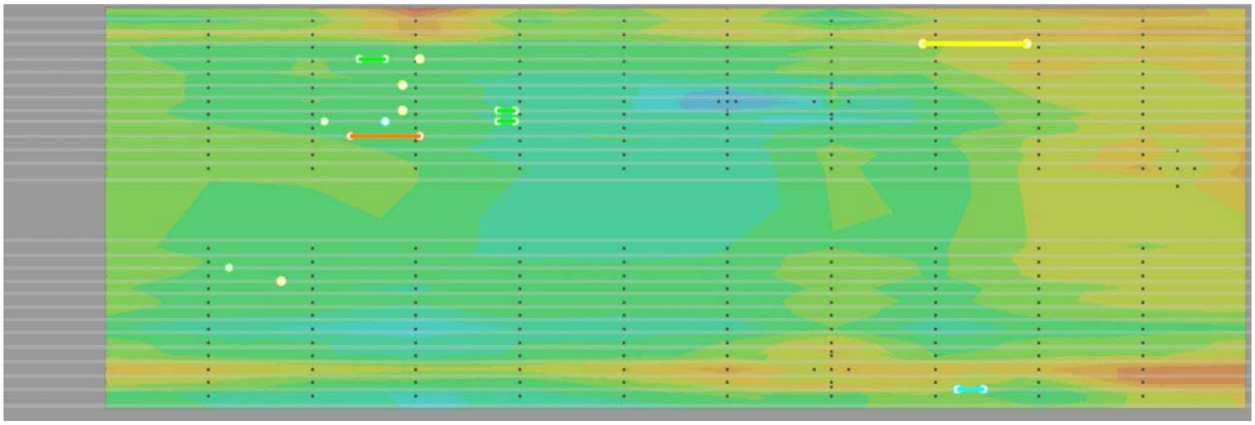


Figure 7-6: Overlay of Damage Profile on Half Cell Potential Map for Beam LV16

As shown in Figure 7-7, the detected damage found on beam LV19 was spread throughout a cracked region just above the centerline of the beam. This highly cracked region showed voltage readings which indicated corrosion.

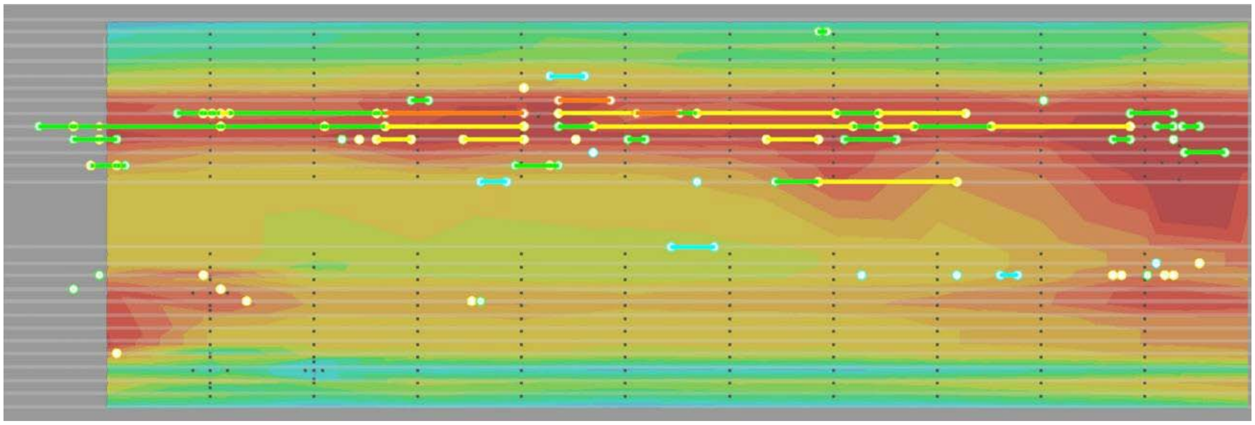


Figure 7-7: Overlay of Damage Profile on Half Cell Potential Map for Beam LV19

As shown in Figure 7-8, the detected damage found on beam MS2 was spread throughout the bottom half of the beam. The top half was spalled off and limited the inspection area to the white box shown in the figure below; with limited amounts of inspection in the top half. The highest levels of damage corresponded with low voltage readings.

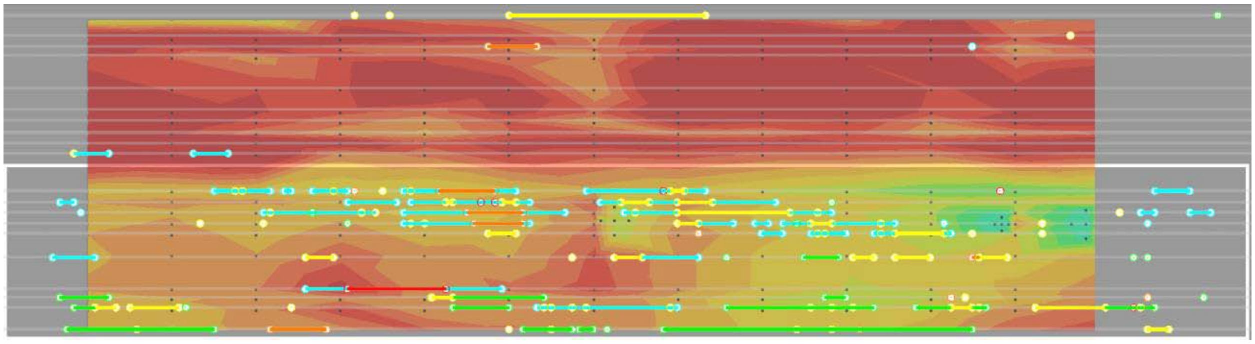


Figure 7-8: Overlay of Damage Profile on Half Cell Potential Map for Beam MS2

As shown in Figure 7-9, the detected damage found on beam MS3 was located along the longitudinal crack discovered during the visual inspection. The half cell potential readings for the strands under the crack indicated low voltage readings. Instances of light corrosion and pitting were found at locations with high half-cell potential readings.

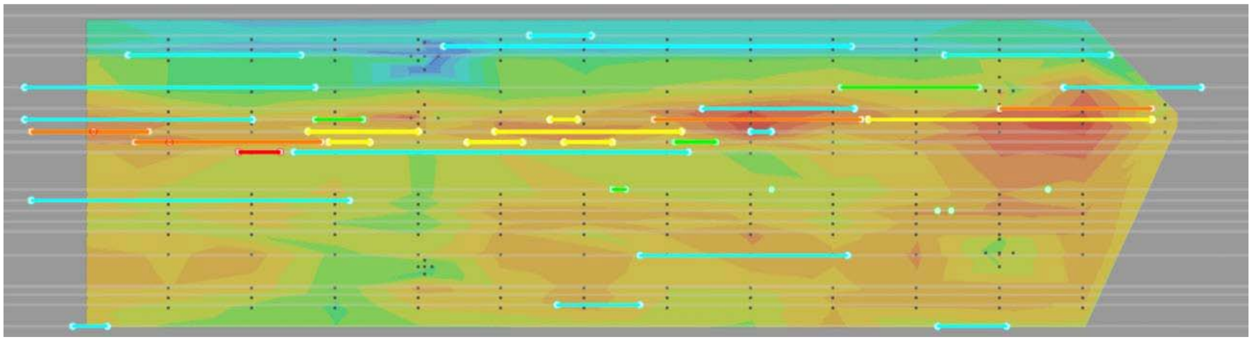


Figure 7-9: Overlay of Damage Profile on Half Cell Potential Map for Beam MS3

7.2.2 Statistical Analysis of Half-Cell Data

Half cell readings were taken at 1503 points on the bottom flange of the seven beams. Each black dot in the half-cell potential maps, shown in Figure 7-3 through Figure 7-9, corresponds to a half cell voltage potential measurement. Removal of the concrete cover and the inspection of the strands allowed for the development of a damage profile plot revealing if damage existed at any given half cell measurement location. A comparison of the half cell potential and the strand damage is summarized in Table 7-2 and Table 7-3.

Table 7-2: Statistical Analysis of Half Cell Voltage Readings in Relation to Extent of Damage

Damage Level	Damage Index	Sample Size	Min Half Cell (V)	Max Half Cell (V)	Average Half Cell (V)	COV
No Corrosion	0	1293	-0.600	0.091	-0.197	56.1%
Light Corrosion	1	63	-0.569	-0.027	-0.269	29.0%
Pitting	2	46	-0.528	-0.101	-0.316	32.9%
Heavy Pitting	3	79	-0.507	-0.084	-0.337	28.7%
Wire Loss	4	19	-0.454	-0.156	-0.349	25.4%
Fracture	5	3	-0.480	-0.253	-0.359	28.1%

The damage index corresponds to the type of damage: “0” represents no damage at the half cell reading location, “1” represents light corrosion, “2” represents pitting, “3” represents heavy pitting, “4” represents wire loss, and “5” represents wire fracture. It is important to note that the average half-cell potential reading increases with the severity of damage indicating that the more negative the voltage reading, the heavier the corrosion damage will be. There is a large coefficient of variation in the half cell readings in relation to the type of damage.

A statistical analysis was performed in order to predict the probability of corrosion for prestressing steels for different ranges of half cell potential. The results are summarized in Table 7-3.

Table 7-3: Probability of Corrosion for Prestressing Steels Based on Half-Cell Potential	
Half-Cell Potential	Probability of Corrosion (DI > 0)
Greater than -0.20 V	3.7%
Between -0.20 V and -0.25 V	9.0%
Between -0.25 V and -0.30 V	18.6%
Between -0.30 V and -0.35 V	26.5%
Less than -0.35 V	45.5%

It is concluded that as the half cell potential reading becomes more negative, the probability of corrosion increases. The results can be compared with the ASTM recommendations for conventional reinforcement (Table 6-1). Based on the results of the study conducted in this report it is found that conventional steel reinforcement is more likely to have corrosion than prestressing steel with the same voltage.

Based on the results of the study, Half Cell Potential methods are not a viable means of detecting corrosion of prestressing strands in box beams. Half Cell methods require connection to the reinforcement to evaluate the potential between different points along the member. For conventional construction (post 1980) strands are enclosed within stirrups which will likely maintain continuity of all the reinforcement. For this condition only one connection to the reinforcement is required. In older construction, however, continuity between the strands cannot be ensured. For these conditions half cell methods will require connection to each strand which results in considerable effort for inspection. To acquire a stable measurement the surface of the concrete must be properly saturated. This is not readily achieved in the field. When the half cell method is used under ideal laboratory conditions it correctly detected corrosion less than 50% of the time. For elevated half cell potential measurements, less than -0.35V there is only a 45% probability that corrosion will occur. Due to the difficulty in achieving a good measurement and the poor accuracy of the method under ideal conditions, half cell is not viable for detecting strand corrosion in pre-tensioned concrete box beams.

7.3 Chloride Half-Cell Correlation

From the four inch cores sent to The Erlin Company, the chloride percent by mass of concrete was obtained; half cell potential readings were also taken at these core locations. In addition to the four inch cores, 0.5 inch diameter plugs were taken along two of the beams widths; CC4 and LV19. The half cell potential readings at these locations are known as well. In comparing half-cell voltage readings and chlorides for the 0.5 inch diameter plugs, Figure 7-10 and Figure 7-11, it is clear that correlation does not exist.

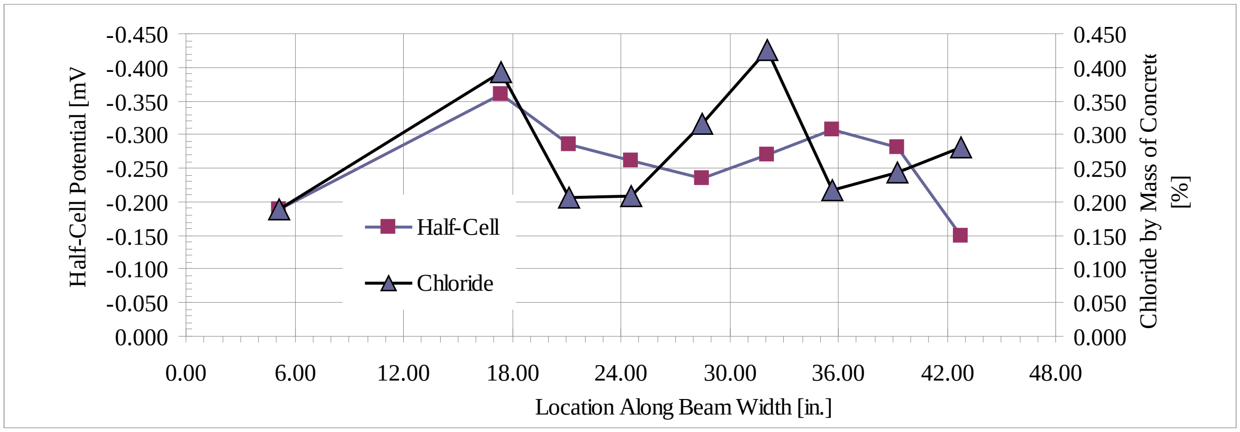


Figure 7-10: Voltage & Chloride Values along the Length of Beam LV19

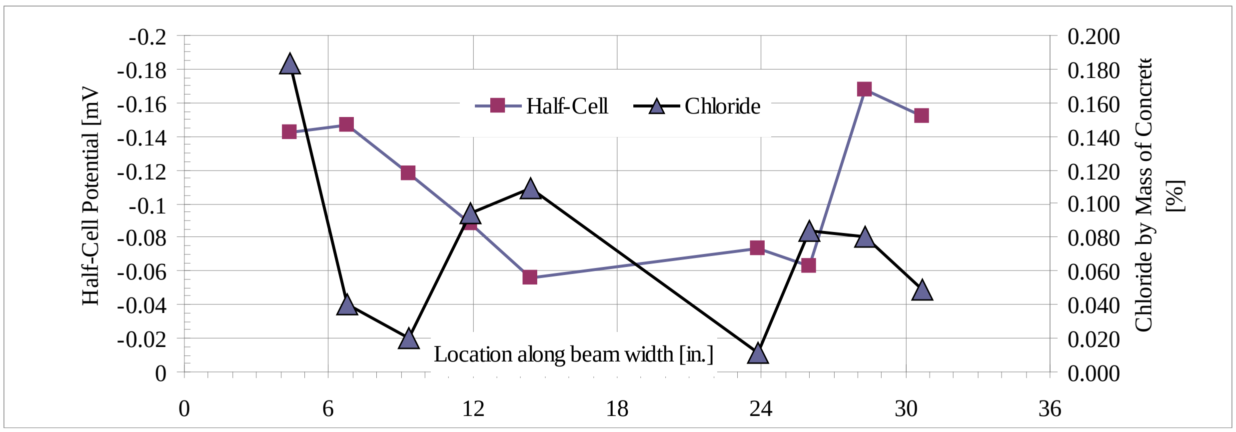


Figure 7-11: Voltage & Chloride Values along the Length of Beam CC4

The half cell voltage readings and chloride values from the 0.5 inch diameter plugs and the 4 inch diameter cores were compiled and analyzed together. The data is summarized in Figure 7-12. A low correlation exists between half cell potential and surface chloride level.

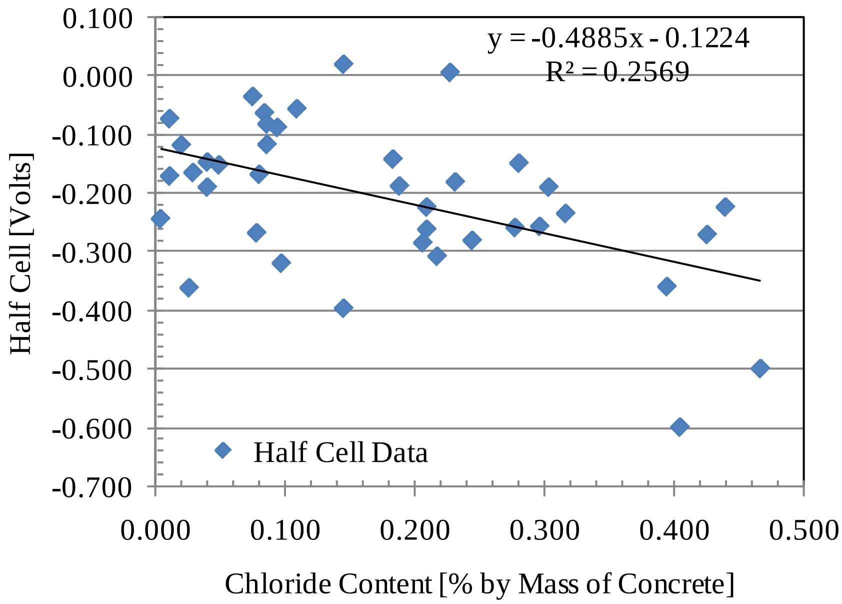


Figure 7-12: Correlation of Half-Cell Potential and Chloride Levels

7.4 Establishing Probabilities of Corrosion

In order to develop qualitative recommendations for PennDOT inspections, it is useful to compare the probabilities for corrosion under various conditions. These probabilities can be used to reduce the area – and correspondingly strength – of the prestressing steel; leading to a reduced moment capacity. Rating rules can be developed from these approaches which realistically estimate the strength of a particular bridge based on condition.

To compute these probabilities, the same data-points used to compile the half-cell statistical analysis are considered. The damage index of the strand at each of these data points is known; allowing for a comparison of the state of corrosion and the surface condition of the concrete bridge. The conditions considered for this analysis were: 1st level strands under a longitudinal crack, 1st level strands with no longitudinal crack, 2nd level strands under a longitudinal crack, 2nd level strands with no longitudinal crack, and 1st level strands adjacent to a longitudinal crack.

7.4.1 Probability of Corrosion of 1st Level Strands under a Longitudinal Crack

Every data point that corresponded to a longitudinal crack was collected and sorted to obtain the data for Table 7-4.

Table 7-4: Probabilities of Corrosion of 1st Level Strands under a Longitudinal Crack

Condition	# Cases	Probability
Overall	115	-
No Corrosion (DI = 0)	34	29.6%
Light Corrosion (DI = 1)	10	8.7%
Pitting (DI = 2)	20	17.4%
Heavy Pitting (DI = 3)	40	34.8%
Wire Loss (DI = 4)	9	7.8%
Fracture (DI = 5)	2	1.7%

Table 7-4: Probabilities of Corrosion of 1st Level Strands under a Longitudinal Crack		
Corrosion (DI > 0)	81	70.4%
Pitting or Greater (DI > 1)	71	61.7%

The data indicates that if a longitudinal crack is present above a strand, there is a 70.4% probability that the strand will have corrosion. Furthermore, if corrosion does exist under the crack, there is a 61.7% probability that the type of corrosion will be pitting or heavier; effectively reducing the cross section and strength of the prestressing strands.

7.4.2 Probability of Corrosion of 1nd Level Strands with no Longitudinal Crack

Every data point that did not occur over a longitudinal crack was collected and sorted to obtain the data for Table 7-5.

Table 7-5: Probabilities of Corrosion of 1st Level Strands with no Longitudinal Crack		
Condition	# Cases	Probability
Overall	1404	-
No Corrosion (DI = 0)	1259	89.67%
Light Corrosion (DI = 1)	53	3.77%
Pitting (DI = 2)	34	2.42%
Heavy Pitting (DI = 3)	47	3.35%
Wire Loss (DI = 4)	8	0.57%
Fracture (DI = 5)	3	0.21%
Corrosion (DI > 0)	145	10.33%
Pitting or Greater (DI > 1)	92	6.6%

The data indicates that when there are no surface indicators of corrosion (no crack) there is a 10.33% probability of finding corrosion in the prestressing strands underneath. Furthermore, there is a 6.6% probability of finding corrosion damage warranting a reduction in wire cross-section and strength (DI > 1) when no cracking is present.

7.4.3 Probability of Corrosion of 2nd Level Strands under a Longitudinal Crack

Every data point for a 2nd level strand under a longitudinal crack was collected and sorted to obtain the data for Table 7-6.

Table 7-6: Probabilities of Corrosion of 2nd Level Strands under a Longitudinal Crack		
Condition	# Cases	Probability
Overall	32	-
No Corrosion (DI = 0)	17	53.13%
Light Corrosion (DI = 1)	3	9.38%
Pitting (DI = 2)	2	6.25%
Heavy Pitting (DI = 3)	4	12.5%
Wire Loss (DI = 4)	3	9.38%
Fracture (DI = 5)	3	9.38%

Table 7-6: Probabilities of Corrosion of 2nd Level Strands under a Longitudinal Crack		
Corrosion (DI > 0)	15	46.9%
Pitting or Greater (DI > 1)	12	37.5%

The data indicates that when a crack is present, the 2nd layer strand above that crack has a 46.87% probability of having corrosion; of the various types of possible corrosion, heavy pitting appeared with the highest frequency. Furthermore, there is a 37.5% probability of finding corrosion damage of pitting or greater; leading to a reduction in strand cross-sectional area and strength.

7.4.4 Probability of Corrosion of 2nd Level Strands with no Longitudinal Crack

Every data point for a 2nd level strand with no longitudinal cracking was collected and sorted to obtain the data for Table 7-7.

Table 7-7: Probabilities of Corrosion of 2nd Level Strands with No Longitudinal Crack

Condition	# Cases	Probability
Overall	55	-
No Corrosion (DI = 0)	47	85.45%
Light Corrosion (DI = 1)	5	9.09%
Pitting (DI = 2)	1	1.82%
Heavy Pitting (DI = 3)	1	1.82%
Wire Loss (DI = 4)	1	1.82%
Fracture (DI = 5)	0	0%
Corrosion (DI > 0)	8	14.6%
Pitting or Greater (DI > 1)	3	5.5%

The data indicates that 2nd level strands with no surface indicators of corrosion have a 85.45% probability of having no corrosion whatsoever; conversely there is a 14.55% probability of having corrosion. However, light corrosion makes up a large portion (9.09%) of that probability. Therefore, there is only a 5.45% probability of finding heavy enough corrosion (DI > 1) to warrant a reduction in strand cross sectional area and strength.

7.4.5 Probability of Corrosion of 1st Level Strands Adjacent to a Longitudinal Crack

Every data point for a strand directly adjacent to a longitudinal crack was collected and sorted to obtain the data for Table 7-8.

Table 7-8: Probabilities of Corrosion of 1st Level Strands Adjacent to a Longitudinal Crack

Condition	# Cases	Probability
Overall	95	-
No Corrosion (DI = 0)	66	69.47%
Light Corrosion (DI = 1)	6	6.32%
Pitting (DI = 2)	8	8.42%
Heavy Pitting (DI = 3)	11	11.58%

Table 7-8: Probabilities of Corrosion of 1st Level Strands Adjacent to a Longitudinal Crack		
Wire Loss (DI = 4)	4	4.21%
Fracture (DI = 5)	0	0%
Corrosion (DI > 0)	29	30.5%
Pitting or Greater (DI > 1)	23	24.2%

The data indicates that when a longitudinal crack is present on the bottom beam surface, an adjacent strand has a 30.5% probability of having corrosion; further there is only a 24.2% probability of having corrosion heavy enough to reduce the cross sectional area and strength of the strand.

7.4.6 Probability of Corrosion Summary

A methodology for dealing with longitudinal cracking was presented in a previous paper from the ATLSS Center at Lehigh University [Naito, et. al. 2006]. It was recommended that when a longitudinal crack is found on a beam, the prestressing strands underneath the crack be discounted, as well as the strand adjacent to the crack.

From the probabilities as presented in Table 7-4 through Table 7-8, it is concluded that completely discounting the strength of a strand adjacent to a longitudinal crack is overly conservative. A summary of the probability of finding corrosion under various surface conditions were computed. They have been organized into Table 7-9 and Table 7-10. As illustrated there is 70.4% probability of corrosion on strands located above a crack. For strands located adjacent to a crack the probability decreases to 30.5%. Strands located in the second layer above a crack have a 46.9% probability of corrosion.

The reduction in strength associated with levels of corrosion was examined in 2006 [Naito, et. al. 2006]. The results of the study are reproduced in Table 7-11. Based on the findings light corrosion did not alter the strength of the material. Pitting and heavy pitting resulted in a decrease in the tensile strength due to the reduction in cross-section and stress concentrations generated at the pitted sections.

Table 7-9: Probability of Finding Corrosion of Steel Strands Under a Longitudinal Crack

Location	NC	LC	P	HP	WL	F	DI>0	DI>1
Under Crack (1st Level)	29.6%	8.7%	17.4%	34.8%	7.8%	1.7%	70.4%	61.7%
Adjacent to Crack (1st Level)	69.5%	6.3%	8.4%	11.6%	4.2%	0.0%	30.5%	24.2%
Under Crack (2nd Level)	53.1%	9.4%	6.3%	12.5%	9.4%	9.4%	46.9%	37.5%

The above table was compiled using data presented in sections 7.4.1, 7.4.3, and 7.4.5.

Table 7-10: Probability of Finding Corrosion of Steel Strands with No Longitudinal Crack

-	NC	LC	P	HP	WL	F	DI>0	DI>1
@ 1st Level Strands	89.7%	3.8%	2.4%	3.4%	0.6%	0.2%	10.3%	6.6%
@ 2nd Level Strands	85.5%	9.1%	1.8%	1.8%	1.8%	0.0%	14.6%	5.5%

The above table was compiled using data presented in sections 7.4.2 and 7.4.4.

Table 7-11: Average Wire Strength Due to Corrosion			
Wire Condition	Strength [ksi]	COV	Relative Strength
Light Corrosion	288.0	4.2%	100%
Pitting	230.0	10.6%	79.9%
Heavy Pitting	205.6	10.9%	71.4%

7.5 Sounding

Sounding was performed as described in Section 3.6 per ASTM D4580, "Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding". Small areas of delamination were expected in three beams (MS2, MS3, and LV19) and generally occurred over or alongside the main longitudinal cracks. Destructive evaluation did not find delamination at the areas identified. This method failed to detect the large delamination on Beam LV7 that was found in the destructive evaluation phase of testing.

The sounding technique identified regions of heavy pitting in the three beams examined. Comparison of the strand damage and the delamination regions can be seen for Main Street Beam 2 (Figure 3-15 and Figure 5-8), Main Street Beam 3 (Figure 3-16 and Figure 5-9), and Lakeview Drive Beam 19 (Figure 3-17 and Figure 5-7). The regions of expected delamination correspond directly with strand damage equivalent to Heavy Pitting or Wire Loss. From the limited study it appears that sounding may provide a means of identifying non-visible corrosion of prestressing strands. This may be attributed to the presence of corrosion product around the strand which would result in a hollow region. Further study of this phenomenon is recommended.

7.6 Relationship between 1st and 2nd Level of Strands

The second level of steel was inspected by two different methods: the punch-outs performed with a jackhammer and the extraction of the strands from the additional cores. The damage levels of the second level of strands were compared with the 1st level of strands to determine if a relationship existed. This data is compiled in Table 7-12.

Table 7-12: Relationship between 1st and 2nd Layer of Strands	
# of Cases for Comparison	87
# of Cases w/ Damage at 2nd Level	23
# of Cases w/ Damage at 1st Level	51
# of Cases w/ No Damage. at 2nd Level	64
# of Cases w/ Damage at 1st Level, but no Damage at 2nd Level	28
# of Cases w/ Damage at 1st Level, Damage at 2nd Level	23
# of Cases for DI2 > DI1	3
# of Cases for DI2 < DI1	20

The data indicates that when damage is found at the 1st level of strands, damage is present on the 2nd level of strands 45.1% of the time. Conversely, damage was not present 54.9% of the time when damage is found on the 1st level of strands. The average damage index for each of these conditions was computed. When damage existed on the 1st layer of strands but no damage on the 2nd layer of strands, the average damage index on the first level was 2.46. When there was damage found at both the 1st level and 2nd level of strands, the average damage index of the first level was 2.82. In analyzing cases where damage was present for both levels of steel, it is shown that the damage index is larger on the 1st level of strands 87% of the time.

Thus it can be concluded that if corrosion exists on the first level there is approximately a 50% chance that the second level will be corroded. This condition is more likely to occur with greater levels of corrosion on the 1st level. Also for 87% of the cases the corrosion will be lower on the second level of strands.

7.7 Effect of Strand Damage on Nominal Moment Capacity

The section geometry and material properties were taken from sections 2.2, 2.3, and 2.4 and used to calculate the nominal moment capacity (φMn) of each bridge beam. The moment capacities were calculated in two ways; through the strain compatibility method and the simplified fps formulation provided by ACI for flexural members with effective prestress in excess of 50% of the ultimate strand stress. The results are as follows in Table 7-13, where A_{ps} represents the area of prestressing steel, e is the eccentricity from the centroid of the strands to the centroid of the section, I_c is the moment of inertia of the section, Ac is the area of concrete, P_i is the initial prestressing force, and P_e is the effective prestressing force.

Table 7-13: Box Beam Flexural Capacity

Beam	A_{ps} (in^2)	e (in)	I_c (in^4)	A_c (in^2)	P_i (kip)	P_e (kip)	ACI f_{ps} φM_n	ε Comp. φM_n
CC3	3.15	16.28	146711	646.7	550.4	381.4	1890	1925
CC4	3.15	16.28	146711	646.7	550.4	381.3	1890	1925
LV7	3.23	7.76	167604	645.0	565.3	415.9	1080	1115
LV16	5.10	17.29	62943	779.8	892.5	568.5	3160	3235
LV19	5.10	17.36	191981	774.8	892.5	566.4	3160	3240
MS2	4.76	17.00	190187	772.2	833.0	543.3	2925	2995
MS3	4.76	17.00	190187	772.2	833.0	543.4	2925	2995

The nominal bending strength reductions were computed for the following three conditions:

- In-situ nominal moment capacity (φM1); based on the forensic investigation of the strands.
- The PennDOT Strike-off Letter 431-07-08 recommendations (φM2).
- Proposed nominal moment capacity (φM3); using the new recommended probability assessment based on a visual inspection;

For the in-situ nominal moment capacity, each damage profile (see sections 5.2 and 5.3) was analyzed for the worst case of cross-sectional strand damage; this critical cross-section is representative of the actual moment capacity of these bridge beams as they were in service. See Table 7-11 for the reduction in strength based on the damage index. Strands having a condition of WL or F were completely discounted from the strength calculations.

PennDOT Strike off letter 431-07-08 recommendations suggest that any strand intersecting a longitudinal crack, as well as any adjacent strands to that crack, be fully discounted from strength calculations.

The proposed recommendations for reduction in flexural capacity use previously presented probabilities (Table 7-9 and Table 7-10) in conjunction with random number generation to produce average strength reductions based on a visual inspection – i.e., crack or no crack. A random number was generated between 0 and 1 for ten thousand cases for each condition: adjacent strands to crack, strand at 1st level under crack, strand at 2nd level under crack, strand at 1st level with no crack, and strand at 2nd level with no crack. Based on where the random number fell relative to the cumulative probability of a specific condition, it was assigned a damage designation of NC/LC, P, HP, and WL/F. The data from Table 7-11 was then used to reduce the strength of a strand if a designation of P, HP, or WL/F was assigned (no strength reduction was used for the NC/LC cases). The strength was averaged over the ten thousand cases of random numbers to produce the average strength for a particular condition as shown below in Table 7-14 and Table 7-15. These values were included in the strength calculations to produce a new set of proposed reduced moment capacities.

Based on the probabilistic examination of strand corrosion in beams with longitudinal cracking it was found that for strands under a crack (1st or 2nd level of steel) as well as strands adjacent to a crack the cross sectional area should be reduced to 77.4% of the original area for capacity calculations. Additionally, for strands in areas where no visible damage is observed the cross sectional area should be reduced to 97.8% of the original area for capacity calculations.

Table 7-14: Strand Strength Reductions Based on Probabilities for Longitudinal Cracking

Condition:	Avg. Strength	Standard Deviation
Strand @ 1st Level Under Crack	77.5%	28.7%
Strand Adjacent to Crack	77.3%	20.3%
Strand @ 2nd Level Under Crack	77.4%	28.7%

Table 7-15: Strand Strength Reductions Based on Probabilities w/ No Longitudinal Cracking

Condition:	Avg. Strength	Standard Deviation
Strand @ 1st Level	97.8%	11.4%
Strand @ 2nd Level	97.8%	11.7%

In order to simplify the strength calculations, and additionally make the method slightly more conservative, the following is recommended:

- Strands intersecting a crack (1st or 2nd level of steel), as well as strands adjacent to a crack, shall have their cross sectional area reduced to 75% of the original area for capacity calculations
- All other strands in the section, not intersecting or adjacent to a longitudinal crack, shall have their cross sectional area reduced to 95% of the original area for capacity calculations.

Further clarification on this rating method will be given in the following section.

7.7.1 Rating Recommendations

The development of the proposed bridge inspection rating method was illustrated in the preceding section. In addition to longitudinal cracking, other factors such as deteriorated concrete, spalling, exposed reinforcement, etc. were taken into account to produce the following revised proposed bridge inspection summary. The summary is also included in the Appendix as a standalone document.

The following guidelines are recommended for the inspection of adjacent prestressed concrete non-composite box-girder bridges. The procedure requires that each beam member be evaluated for the presence of longitudinal cracking, spalled sections, exposed strands, and deteriorated concrete. The surface damage conditions for each member shall be recorded based on visual observations.

For the purpose of load rating all damage within a region of two development lengths shall be considered to occur at the same section. The computed development length can be used; however, if design information is unavailable the lengths presented in Table 7-16 can be used for typical seven wire strands:

Table 7-16: Inspection Window Size for Beams Based on Strand Diameter

Strand Nominal Diameter [in.]	3/8	7/16	1/2	½ Special
Inspection Window Length [in.]	128	150	170	180

The location of the reduced section strength shall be assumed to occur at the center of the inspection window. The strength reductions shall be based on the presence of longitudinal cracking and deteriorated concrete as noted in the following section.

FOR SPECIMENS WITH LONGITUDINAL CRACKING:

1. The following strand areas shall be reduced to 75% of the original cross-sectional area for capacity calculations:
   a. Strands on each level directly in line the crack.
   b. Strands closest to the exterior surface adjacent to the longitudinal crack. If the adjacent strand is greater than 3in. from the crack see the following item 2 for area reduction.
2. For beams with longitudinal cracking or corrosion induced spalling, all other strands in the section shall be reduced to 95% of the original cross-sectional area for capacity calculations.

FOR SPECIMENS WITH DETERIORATED CONCRETE:

(Adopted from "Guidelines for Estimating Strand Loss in Structural Analysis of PPC Deck Beam Bridges" by the Illinois Department of Transportation)

1. For exposed strands observed with sound concrete adjacent to and above the exposed strands, disregard the full strength of the exposed strands for capacity calculations.
2. For exposed strands observed with adjacent unsound concrete, disregard the full strength of the exposed strands and all strands in regions of unsound concrete for capacity calculations.
3. For exposed shear reinforcement bars, disregard the full strength of strands located in the lower row directly above the exposed section of stirrups for capacity calculations. If the concrete is found to be unsound adjacent to the exposed area, disregard the strength of all strands in all rows above the area of unsound concrete in capacity calculations.
4. For area of concrete where delaminations have been observed, remove all delaminated concrete to determine the depth of the concrete deterioration:

a. If shear reinforcement bars or strands are exposed, treat as in cases "1" through "3" as shown above.
b. If no shear reinforcement bars or strands are exposed but there are indications that the exposed concrete is unsound within the affected area, disregard the strength of all strands located in the rows of strands above the area for capacity calculations.
c. If no steel reinforcement is exposed in the affected area and the concrete is deemed as sound, do not disregard the strength of strands in the strength analysis.

5. For wet or stained areas of concrete observed on the bottom or side of beams, closely inspect those areas to determine the soundness of the concrete:
   a. If close inspection indicates that the concrete is unsound or delaminated, treat as in case "4" above.
   b. If close inspection confirms that the concrete is sound, do not disregard the strength of strands in the strength analysis.

7.7.2 Rating Recommendation Example

A prestressed concrete box beam section is illustrated in Figure 7-13. The damage within a region of one development length is included in the section image. Field inspection of the beam identified three longitudinal cracks, spalling and an area of unsound concrete. The construction documentation indicates that the beam is reinforced with 36 - 3/8 in. diameter seven-wire grade 270 prestressing strands. The spacing and arrangement of the strands is shown in Figure 1.

Using the recommended rating procedure the area reductions and reduced flexural strength is computed. This is conducted in the following stages: 1) the location of cracking, spalling and deteriorated concrete is used to determine a reduced area of prestressing steel (Figure 7-13), 2) a new center of gravity of steel and corresponding eccentricity is computed (Table 7-17), 3) a reduced nominal moment capacity is computed in accordance with ACI 318 recommendations.

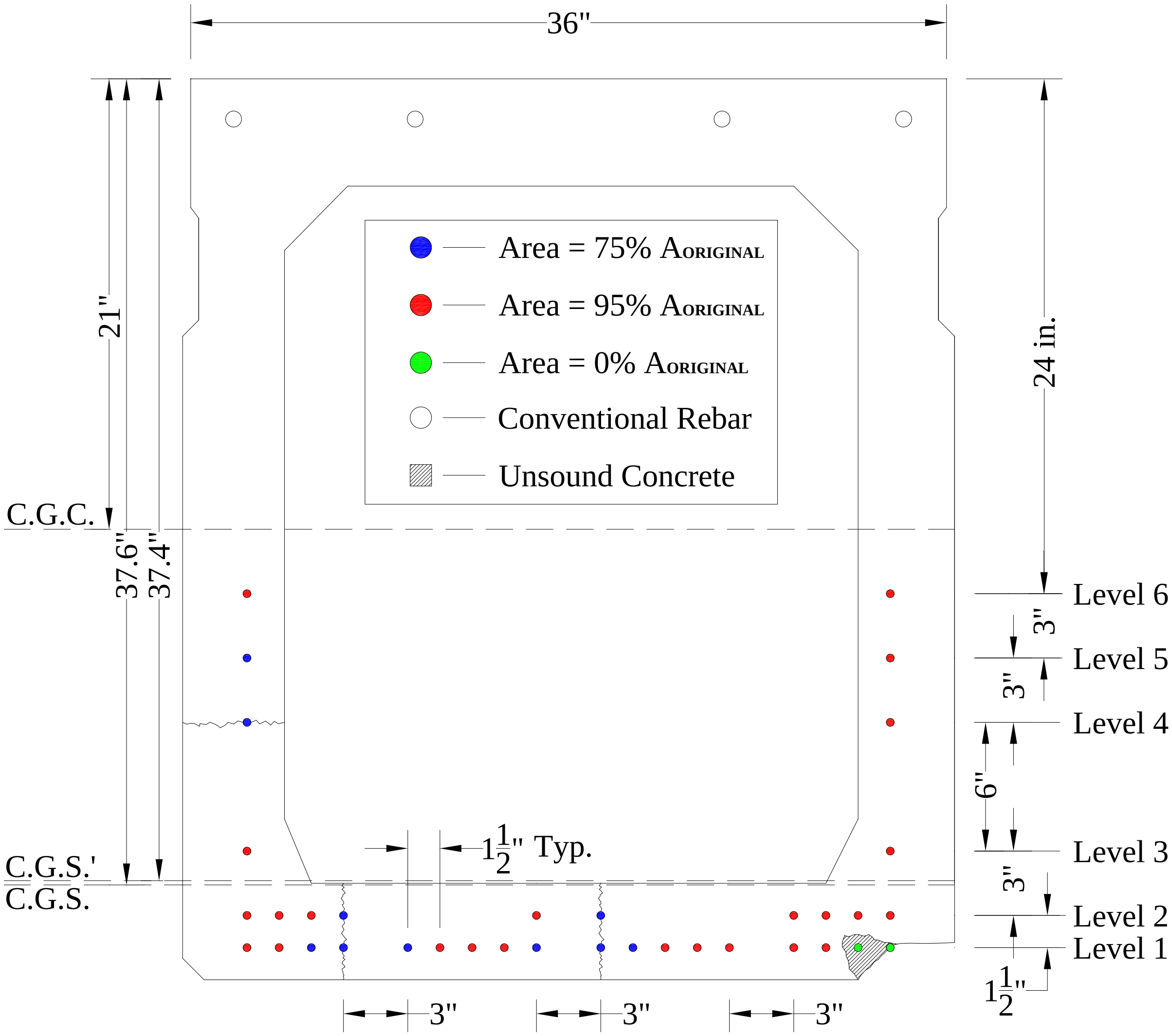


Figure 7-13: Sample Damaged Section Geometry

Table 7-17: Strand Area Reduction Calculations

Level:	Original Area:	Depth from Top:	Reduced Area for Capacity Calculations:
1	A_1 = 18 x 0.085 in^2 = 1.53 in^2	d_1 = 40.5 in	A'_1 = [(10 x 95%) + (6 x 75%) + (2 x 0%)] x 0.085 in^2 = 1.19 in^2
2	A_2 = 10 x 0.085 in^2 = 0.85 in^2	d_2 = 39 in	A'_2 = [(8 x 95%) + (2 x 75%)] x 0.085 in^2 = 0.77 in^2
3	A_3 = 2 x 0.085 in^2 = 0.17 in^2	d_3 = 36 in	A'_3 = (2 x 95%) x 0.085 in^2 = 0.16 in^2
4	A_4 = 2 x 0.085 in^2 = 0.17 in^2	d_4 = 30 in	A'_4 = [(1 x 95%) + (1 x 75%)] x 0.085 in^2 = 0.15 in^2
5	A_5 = 2 x 0.085 in^2 = 0.17 in^2	d_5 = 27 in	A'_5 = [(1 x 95%) + (1 x 75%)] x 0.085 in^2 = 0.15 in^2

Table 7-17: Strand Area Reduction Calculations			
Level:	Original Area:	Depth from Top:	Reduced Area for Capacity Calculations:
6	A_6 = 2 x 0.085 in^2 = 0.17 in^2	d_6 = 24 in	A'_6 = (2 x 95%) x 0.085 in^2 = 0.16 in^2

The distance from the extreme compression fiber to the center of gravity of steel is computed for the original section, d_p, and the damaged section d'_p as follows.

$$d_p = \frac{\sum A_i \cdot d_i}{\sum A_i} \qquad \text{and} \qquad d'_p = \frac{\sum A'_i \cdot d_i}{\sum A'_i}$$

Calculations:

$$\Sigma A_i = [1.53\ \text{in}^2 + 0.85\ \text{in}^2 + 0.17\ \text{in}^2 + 0.17\ \text{in}^2 + 0.17\ \text{in}^2 + 0.17\ \text{in}^2] = 3.06\ \text{in}^2$$

$$\Sigma A'_i = [1.19\ \text{in}^2 + 0.77\ \text{in}^2 + 0.16\ \text{in}^2 + 0.15\ \text{in}^2 + 0.15\ \text{in}^2 + 0.16\ \text{in}^2] = 2.58\ \text{in}^2$$

$$d_p = [1.53*40.5 + 0.85*39 + 0.17*36 + 0.17*30 + 0.17*27 + 0.17*24]/(3.06) = 37.6\ \text{in}$$

$$d'_p = [1.19*40.5 + 0.77*39 + 0.16*36 + 0.15*30 + 0.15*27 + 0.16*24]/(2.58) = 37.4\ \text{in}$$

$$e_p = d_p - \text{C.G.C.} = 37.6\ \text{in} - 21\ \text{in} = 16.6\ \text{in}$$

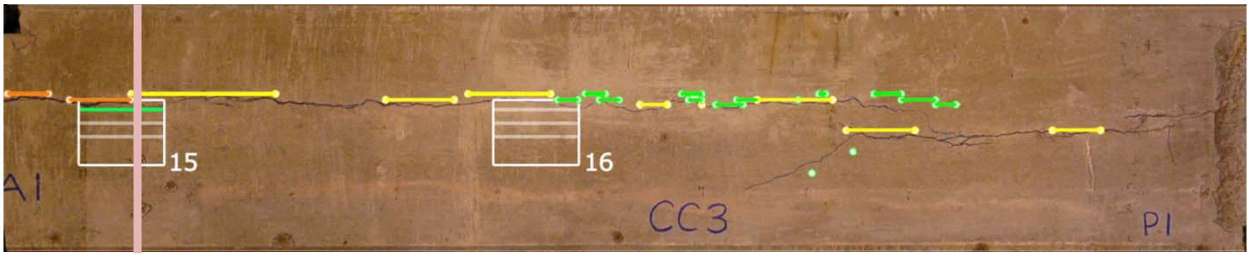
$$e'_p = d'_p - \text{C.G.C.} = 37.4\ \text{in} - 21\ \text{in} = 16.4\ \text{in}$$

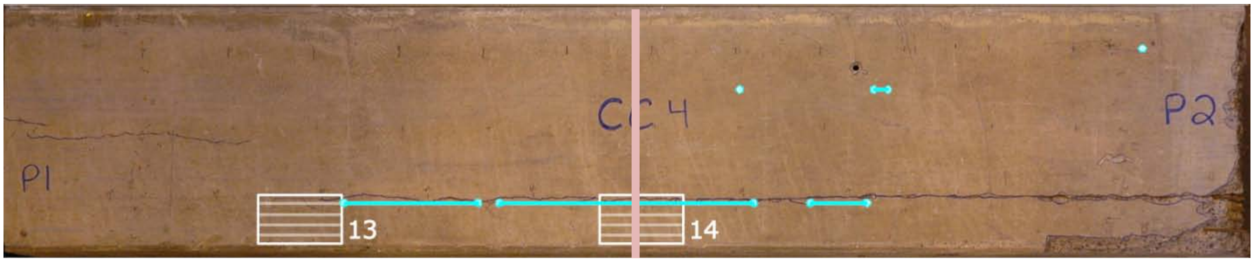
$$ØM_n = 1995\ \text{ft*k}$$

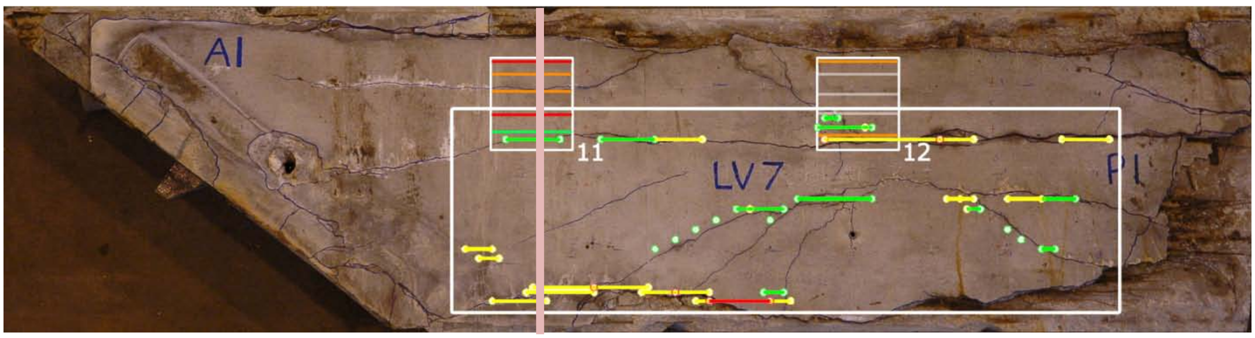
$$ØM'_n = 1715\ \text{ft*k}$$

7.7.3 Strength Reduction Based on Surface Damage

The seven beams examined in this report were studied to assess the reduction in flexural strength based on the three different methods: (1) detected in-situ damage, (2) PennDOT SOL 431-07-08 rating method, and (3) the proposed rating method. For clarification on what these different methods take into account, refer to section 7.7. For in-situ damage, the critical cross sections chosen for the analytical study are presented along with the strand conditions in Figure 7-14. These figures contain damage information for both levels of steel; refer to 5.2 and 5.3 for additional clarification. The section analyzed is marked with a vertical line at the location with the most strand damage. The PennDOT SOL 431-07-08 inspection method illustrated in the previous section was applied to obtain the second set of moment capacities. Finally, the average strand strength based on surface conditions from the proposed rating method was used to compute the last set of moment capacities. The reduced flexural capacities are summarized in Table 7-18 and Table 7-19. The inspection window recommendation previously mentioned was NOT used for these calculations due to the abbreviated length of the beam sections and because the locations of all corrosion damage were accurately determined through destructive evaluation. For conventional applications of this proposed method, follow the guidelines as shown in section 7.8.


A1
15
16
CC3
P1


P1
CC4
P2
13
14


A1
11
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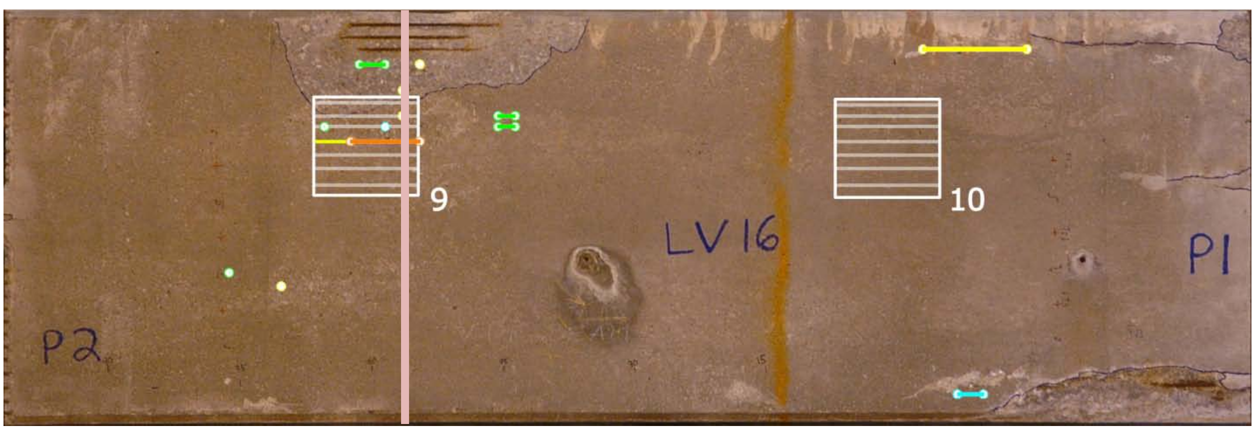
LV7
P1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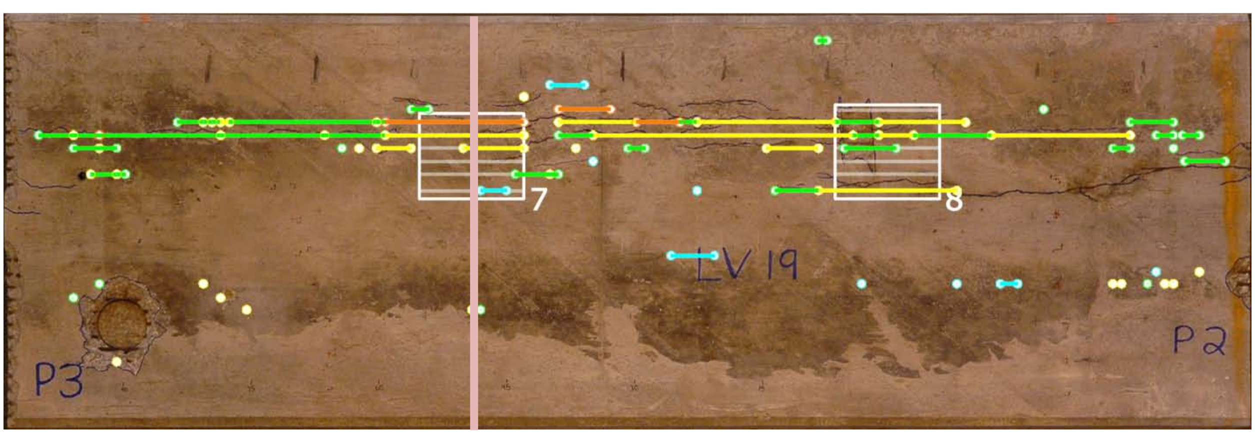
10
LV16
P2
P1


7
8
LV19
P3
P2

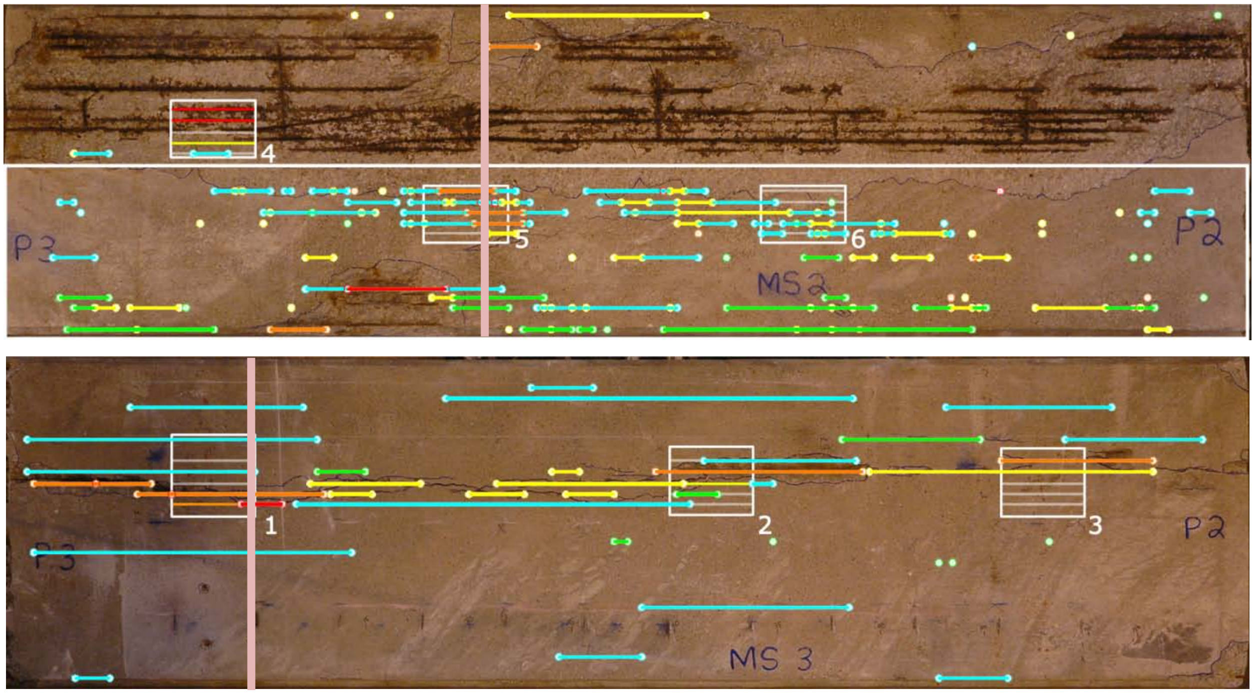


Figure 7-14: Sections evaluated

Table 7-18: Reduction in Flexural Capacity (fps Formulation)

Bridge	Beam	In-Situ Condition		PennDOT Pub.100 Recommendation		Proposed Recommendation	
		ACI f_{ps} φM_1	% Reduction $\varphi M_1/\varphi M_n$	ACI f_{ps} φM_2	% Reduction $\varphi M_2/\varphi M_n$	ACI f_{ps} φM_3	% Reduction $\varphi M_3/\varphi M_n$
Clearfield Creek	3	1810	95.8	1580	83.6	1740	92.1
	4	1855	98.2	1735	91.8	1770	93.7
Lakeview Drive	7	475	44.0	305	28.2	455	42.1
	16	2920	92.4	2780	88.0	2810	88.9
	19	3030	95.9	2755	87.2	2925	92.6
Main Street	2	2165	74.0	2035	69.6	2115	72.3
	3	2775	94.9	2530	86.5	2715	92.8

Table 7-19: Reduction in Flexural Capacity (ε Compatibility Formulation)

Bridge	Beam	Current Condition		PennDOT Pub.100 Recommendation		New Recommendation	
		ε Comp. φM_1	% Reduction $\varphi M_1/\varphi M_n$	ε Comp. φM_2	% Reduction $\varphi M_2/\varphi M_n$	ε Comp. φM_3	% Reduction $\varphi M_3/\varphi M_n$
Clearfield Creek	3	1885	97.9	1615	83.9	1780	92.5
	4	1925	100	1775	92.2	1810	94.0
Lakeview Drive	7	485	43.5	305	27.4	455	40.8
	16	2990	92.4	2840	87.8	2905	89.8

Table 7-19: Reduction in Flexural Capacity (ε Compatibility Formulation)

Bridge	Beam	Current Condition		PennDOT Pub.100 Recommendation		New Recommendation	
		ε Comp. φM_1	% Reduction $\varphi M_1/\varphi M_n$	ε Comp. φM_2	% Reduction $\varphi M_2/\varphi M_n$	ε Comp. φM_3	% Reduction $\varphi M_3/\varphi M_n$
	19	3105	95.8	2820	87.0	2995	92.4
Main Street	2	2205	73.6	2070	69.1	2155	72.0
	3	2845	95.0	2585	86.3	2775	92.7

On average the current condition of the beams reduces the flexural capacity to 85.2% of the undamaged capacity. The PennDOT recommendation reduces the capacity to 76.3% of the undamaged condition. The new recommendation reduces the capacity to 82.1% of the undamaged capacity. Based on this comparison the existing recommendation is overly conservative and the new recommendation provides a conservative estimate of the remaining strength. The flexural capacity computed with strain compatibility is marginally higher than that computed with the ACI simplification. The reduction in strength is approximately the same for both methods.

In comparing the values for the in-situ strength values to the proposed probability based strength reductions, Table 7-20, it is clear that the proposed approach yields slightly conservative yet accurate results.

Table 7-20: Comparison of Actual Box-Beam Strength and Proposed Strength Reduction

Bridge	Beam	In-Situ	Proposed	% Difference	In-Situ	Proposed	% Difference
		ACI f_{ps} φM_1	ACI f_{ps} φM_3	ACI f_{ps}	ε Comp. φM_1	ε Comp. φM_3	ε Comp.
Clearfield Creek	3	1810	1740	3.9	1885	1780	5.6
	4	1855	1770	4.6	1925	1810	6.0
Lakeview Drive	7	475	455	4.2	485	455	6.2
	16	2920	2810	3.8	2990	2905	2.8
	19	3030	2925	3.5	3105	2995	3.5
Main Street	2	2165	2115	2.3	2205	2155	2.3
	3	2775	2715	2.2	2845	2775	2.5

7.8 Field Application of Proposed Recommendation:

Specialty Engineering, Inc. (SEI) reviewed the proposed recommendation, as outlined in Section 7.7.1, and used the approach to establish bridge ratings for the Ash Street Bridge over Roaring Brook in Lackawanna County, PA; additionally, SEI also assessed that same bridge with the current PennDOT rating method. This report can be viewed in full in the Appendix.

The Ash Street Bridge is a non-composite adjacent prestressed concrete box beams bridge composed of 12 concrete box beams with a span length of 66.6 feet. The cross section of the bridge is shown in Figure 7-15.

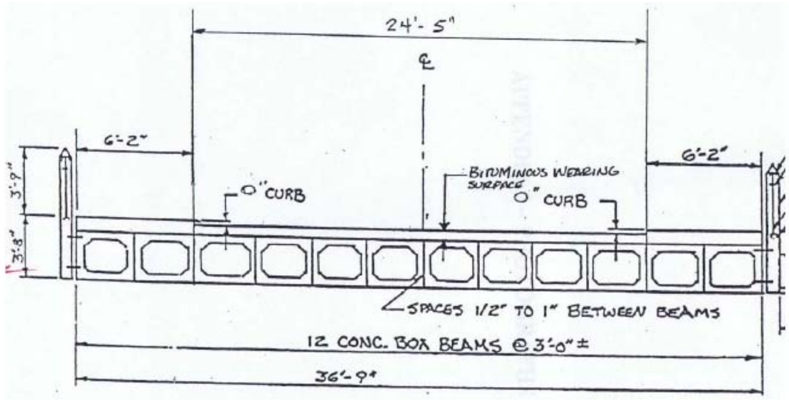


Figure 7-15: Ash Street Bridge Cross-Section

The visual inspection performed by SEI revealed many longitudinal cracks, areas of concrete spalling, exposed steel strand, and areas of delaminated concrete. A field sketch indicating the locations of these flaws along the bottom flanges of the members is shown below in Figure 7-16.

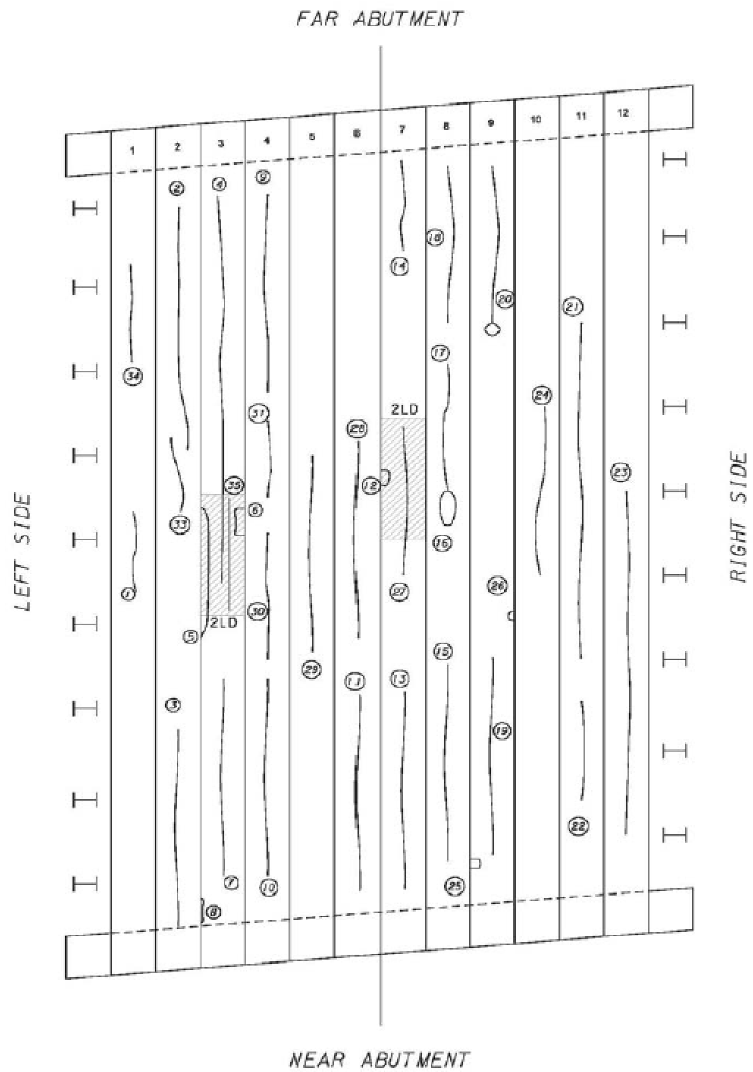


Figure 7-16: Field Sketch of Flaws from Visual Inspection

Due to the fact that no shop drawings were available for this particular bridge, the exact strand layout is unknown. Therefore, the PennDOT standard drawing ST-207 was used to recreate the strand layout most likely to have been used for this type of bridge built during this time period. It was determined to most likely be reinforced with twenty-two 7/16 in. diameter strands, having an initial tensile stress of 175 ksi and an ultimate tensile strength of 250 ksi. Due to the diameter of the reinforcement and as shown in Section 7.7.1, the inspection window to be used is 150 inches; this is shown as the shaded areas in Figure 7-16. The gross area of reinforcing steel is calculated to be 2.40 square inches.

It was determined by SEI that Beam 3 had the most damage for any given inspection window, therefore, this was the cross section in which the two rating method would be applied. In applying the proposed rating method to beam 3, Figure 7-17, the reduced area of steel due to flaws was determined to be 2.23 square inches. In applying the current PennDOT rating method to beam, Figure 7-18, the reduced area of steel due to flaws was determined to be 1.74 square inches.

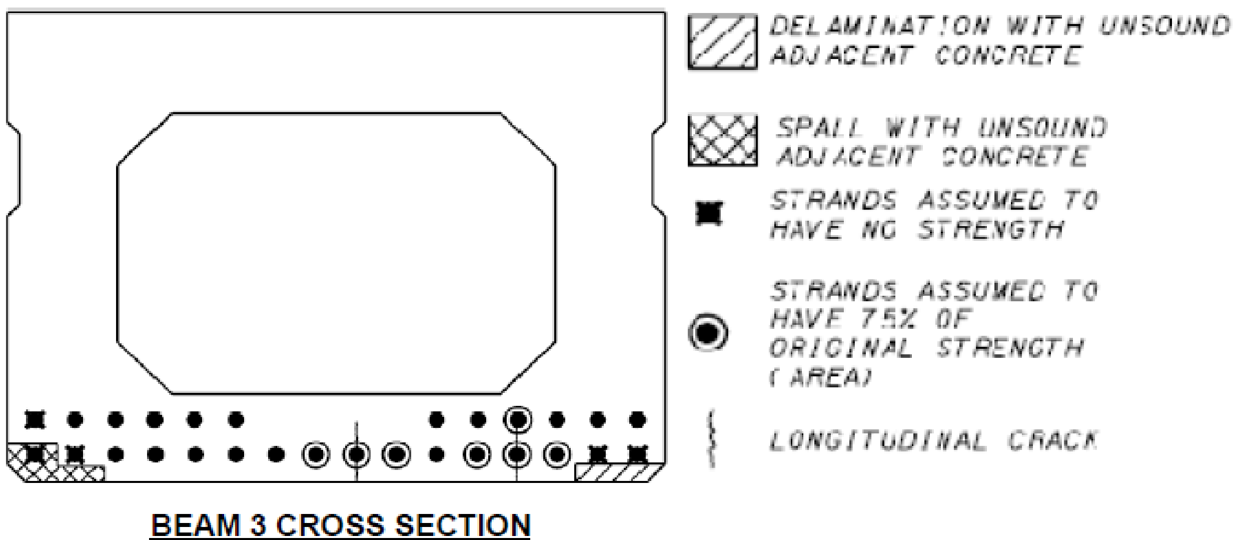


Figure 7-17: Strand Area Reductions by the Proposed Rating Method

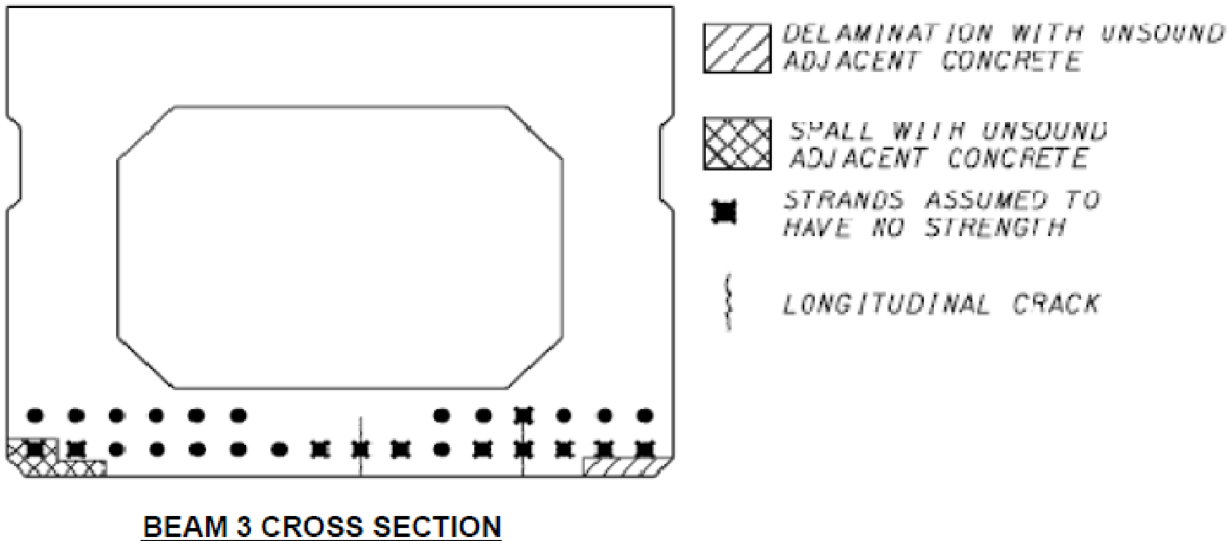


Figure 7-18: Strand Area Reductions by the Current PennDOT Rating Method

The strand areas and concrete properties were input into the PS3 program, which calculates the Inventory and Operating Ratings for bridge beams. As shown in Table 7-21, the average percent difference for an Inventory Rating between the two methods is 387.5%. As shown in Table 7-22, the average percent difference for an Operating Rating between the two methods is 75.9%. These results indicate that the current PennDOT rating method is much more conservative than the proposed method.

Table 7-21: Inventory Rating [Tons]

Vehicles:	Current Method: Beam 3	Proposed Method: Beam 3	Percent Difference:
H20	2.95	14.38	387.46
HS20	3.78	18.43	387.57
ML80	3.28	15.98	387.20
TK527	3.70	18.05	387.84

Table 7-22: Operating Rating [Tons]

Vehicles:	Current Method: Beam 3	Proposed Method: Beam 3	Percent Difference:
H20	24.54	43.17	75.92
HS20	31.47	55.36	75.91
ML80	27.28	48.00	75.95
TK527	30.82	54.22	75.92

7.9 Summary of Discussion of Results

The overall discussion of results was presented in this chapter. An in-depth analysis of concrete core results was presented; most specifically considering chlorides and concrete strength. The half-cell potential method was considered from both qualitative and quantitative comparisons. An attempt to correlate half-cell voltage readings and chloride percentages was made.

Quantitative analyses were performed with respect to longitudinal cracks, adjacent strands, and adjacent strand layers with the intent of aiding visual inspectors. The adjustments in nominal moment capacities were computed based on finding strand damage. The results presented in this section are as follows:

- The required concrete compressive strength was achieved in all but three beams: MS2, LV16, and LV19. MS2 and LV16 exhibited a large variation in the strength. This variation is attributed to the previously stated conclusion that these two beams had a variation in w/c ratio due to a failure to thoroughly mix during batching. LV19 had a low standard of deviation in strength and was likely fabricated from non-conforming concrete.
- Based on the chloride measurements and corrosion levels determined the following conclusions can be made: (1) on average the ACI limit of 0.013% divides the region between light corrosion and no corrosion, (2) the average chloride readings for all cases with a damage index of 0 (no corrosion) was 0.0113 – under the ACI threshold – compared to 0.0704 for the cases with a damage index greater than 0, (3) the variability in the chloride levels at each corrosion state is very large making determination of chloride thresholds inappropriate for the data set, (4) it is possible to have heavy pitting of the strands with chloride levels of 0.005%, (5) it is possible to have no corrosion with chloride levels of 0.082%.
- The average half cell potential reading tends to increase with the severity of damage (i.e., damage index). The average half potential for the different damage indexes are as follows: (0) undamaged -197mV, (1) light corrosion -269mV, (2) pitting -316mV, (3) heavy pitting -337mV, (4) wire loss -349mV, and (5) fracture -346mV.
- While the half-cell potential measurements and the in-situ stand corrosion damage were correlated the coefficient of variation (COV) was large. The COV for the different levels varied from 25% to 56%. Therefore while the half-cell readings provide an indication of corrosion damage they should not be explicitly applied to predict a particular type of damage.
- Statistical analysis of 1418 readings taken on the seven beams studied revealed that for a potential reading below -350 mV only 45.5% of the time corrosion will be present. Thus even for a high half cell potential measurements, the method was successful in identifying corrosion less than half the time.
- A comparison of surface acid soluble chloride measurements and half cell potential readings indicate that a poor correlation exists.
- Based on the results of the study, Half Cell Potential methods are not a viable means of detecting corrosion of prestressing strands in box beams. Half Cell methods require connection to the reinforcement to evaluate the potential between different points along the member. For conventional construction strands are enclosed within stirrups which will likely maintain continuity of all the reinforcement. For this condition only one connection to the reinforcement is required. In older construction however continuity between the strands cannot be ensured. For these conditions half cell methods will require connection to each strand which requires considerable effort for inspection. To acquire a stable measurement the surface of the concrete must be properly saturated. This is not

readily achieved in the field. When the half cell method is used under ideal laboratory conditions it correctly detected corrosion less than 50% of the time. For elevated half cell potential measurements, less than -0.35V there is only a 45% probability that corrosion will occur. Due to the difficulty in achieving a good measurement and the poor accuracy of the method under ideal conditions half cell is not viable for detecting strand corrosion in pretensioned concrete box beams.

- An examination of the relationship between longitudinal cracking and corrosion was conducted. It was found that if a longitudinal crack is present, there is a 70.4% probability of having corrosion underneath. If corrosion does exist under the crack, there is an 87.7% probability that the type of corrosion will be pitting or heavier; thus, reducing the cross section of the prestressing strand at that location.
- When there are no surface indicators of corrosion (no crack) there is a 10.3% probability of finding corrosion on the prestressing strands underneath; further there is only a 4.3% probability of finding heavy damage (heavy pitting through fracture) when no cracking is present.
- When corrosion damage was found at the 1st level of strands, damage was present on the 2nd level of strands 45.1% of the time. Conversely, damage was not present 54.9% of the time when damage is found on the 1st level of strands.
- Where corrosion damage was present for both levels of steel, it is shown that the damage index is larger on the 1st level of strands 87% of the time.
- When a longitudinal crack is present on the bottom beam surface, an adjacent strand has a 30.5% probability of having corrosion; further there is only a 15.8% probability of having heavy corrosion (heavy pitting through fracture) on adjacent strands to longitudinal cracks. Therefore, discounting a strand adjacent to a longitudinal crack is conservative.
- On average the in-situ condition of the beams reduces the flexural capacity to 85.2% of the undamaged capacity. The current PennDOT recommendation reduces the capacity to 76.3% of the undamaged condition. The proposed recommendation reduces the capacity to 82.1% of the undamaged capacity. Based on this comparison the existing recommendation is conservative and the new recommendation provides a less conservative estimate of the remaining strength.
- The proposed rating recommendation was used to estimate the flexural strength of the beams acquired. Comparing the in-situ strength with the probability based strength reductions it is clear that the proposed approach yields slightly conservative yet accurate results.
- The proposed rating recommendation and current PennDOT rating method were used by SEI to calculate the Inventory and Operation ratings for the Ash Street Bridge in Lackawanna County, PA. The average percent difference for an Inventory Rating between the two methods is 387.5%. The average percent difference for an Operating Rating between the two methods is 75.9%. These results indicate that the proposed rating method provides a significantly less conservative approach than what is currently being used for adjacent box beams.

8 Comprehensive Summary and Conclusions

Beams from the Lake View Drive, Main Street, and Clearfield Creek Bridges in PA were forensically examined. The study included: (1) detailed photography of external corrosion and spalling conditions corresponding to varying internal damage levels, (2) exposure of prestressing strands (1st and 2nd layer) to correlate external surface conditions with internal strand damage, (3) removal and evaluation of concrete cores for compressive strength evaluation, (4) removal and evaluation of concrete cores for chloride profile, depth of carbonation, and petrography, (5) measurement of beam cross-sections to assess as-built dimensions, (6) evaluation of the half cell potential method, (7) an assessment of the correlation between half cell potential, chloride content, and strand corrosion, (8) an evaluation of the effectiveness of sounding methods as means to determine areas of concrete delamination, (9) establishment of a relationship between corrosion on the 1st and 2nd level of strands, (10) improvement of current visual inspection methods, and (11) the resulting impact of the inspection method on bridge rating procedures. It is noted that this investigation and its results are limited to seven beams from three bridges. Other conditions may exist on other non-composite prestressed concrete box beam bridges. From the present study, the key findings and conclusions are as follows:

- The forensic evaluation revealed that large tolerances should be expected in 1950-1960 era prestressed box beams construction. Prestressing strands deviate horizontally and vertically in the cross-section from the locations specified on the design drawings. This may result in reduced cover on the lower layer of strands and difficulty in correlating surface damage with strands. The cardboard forms shift during the concrete placement altering the flange and web thickness of the box beams.

- The combination of vent holes, cardboard forms, and an asphalt wearing surface allowed for the possibility of water entry from the bridge deck surface during the service of the bridge. While the presence of water within the box was not guaranteed, when it was present it produced elevated chloride levels within the concrete on the interior of the beam.

- The concrete used in each beam was found to be sound. The aggregate was well graded and distributed. The water cement ratios varied from 0.38 to 0.43 for 5 of the 7 beams. Main Street Beam 2 and Lake View Drive Beam 16 had a large variation in w/c ratio due to a failure to thoroughly intermix batch or tempering water.

- The required concrete compressive strength was achieved in all but three beams: MS2, LV16, and LV19. MS2 and LV16 exhibited a large variation in the strength. This variation is attributed to the previously stated conclusion that these two beams had a variation in w/c ratio due to a failure to thoroughly mix during batching. LV19 had a low standard of deviation in strength and was likely fabricated with a lower strength concrete.

- The concrete air quality did not meet the industry requirements needed to protect critically saturated concrete from damage by cyclic freezing and deicing chemicals. MS3, CC3, and CC4 were very poorly air-entrained. MS2, LV19, LV16, and LV7 contained air void characteristics of entrained air however, for MS2 and LV7 the voids were erratically distributed. The air content, void spacing factor, and specific surface was only achieved in LV19. Nevertheless, none of the beams exhibited freeze thaw damage.

- Carbonation was not present in six of the beams studied. Main Street Beam 2 was found to have carbonation present up to 1.25 in. below the soffit surface. The adjacent beam on the main street bridge (beam 3) was found to have significantly lower level of carbonation (3/32 in.). Beam MS2 had half of the concrete surface spalled off. It also showed sign of carbonation. This could have led to the increased chlorides at the middle surface.

- Significant corrosion damage was observed on the bottom layer of strands in the beams. Clear cover of the strands was measured at each cut section. The clear cover was less than the prevailing AASHTO requirement of 1.5 in. in 92% of the cases inspected. The clear cover varied from a maximum of 1.75 in. to a minimum of 0.69 in.
- Chloride content was found to be highest at the lower surface (soffit) of the beam, decreasing towards the top of the bottom flange. This indicates that the chlorides leach into the beam from the bottom surface. The two exceptions in this case study were beams LV7 and MS2 which had water present within the beam section during service.
- The in-situ strand condition varied from clean strands with no corrosion to heavy corrosion damage and fracture. Six indices were defined to represent the level of damage present. The damage indices were defined as (0) no corrosion, (1) light corrosion, (2) pitting, (3) heavy pitting, (4) wire loss, and (5) full fracture.
- The average chloride percent by mass of concrete for strands with corrosion damage was 0.0704; this exceeds the ACI chloride threshold of 0.026. The average chloride percent by mass of concrete for strands with no corrosion damage was 0.0113; this is under the ACI threshold.
- The average half cell potential reading tends to increase with the severity of damage (i.e., damage index). The average half potential for the different damage indexes are as follows: (0) undamaged -197 mV, (1) light corrosion -269 mV, (2) pitting -316 mV, (3) heavy pitting -337 mV, (4) wire loss -349 mV, and (5) fracture -346 mV CSE.
- While the half-cell potential measurements and the in-situ stand corrosion damage were correlated the coefficient of variation (COV) was large. The COV for the different levels varied from 25% to 56%. Therefore while the half-cell readings provide an indication of corrosion damage they are not reliable in identifying a particular level of corrosion damage.
- Statistical analysis of 1418 readings taken on the seven beams studied revealed that for a potential reading below -350 mV CSE only 45.5% of the time corrosion will be present. Thus even for a high half cell potential measurements, the method was successful in identifying corrosion less than half the time.
- Based on the results of the study, Half Cell Potential methods are not a viable means of detecting corrosion of prestressing strands in box beams. Half Cell methods require connection to the reinforcement to evaluate the potential between different points along the member. For conventional construction strands are enclosed within stirrups which will likely maintain continuity of all the reinforcement. For this condition only one connection to the reinforcement is required. In older construction however continuity between the strands cannot be ensured. For these conditions half cell methods will require connection to each strand which requires considerable effort for inspection. To acquire a stable measurement the surface of the concrete must be properly saturated. This is not readily achieved in the field. When the half cell method is used under ideal laboratory conditions it correctly detected corrosion less than 50% of the time. For elevated half cell potential measurements,

less than -0.35 V CSE there is only a 45% probability that corrosion will occur. Due to the difficulty in achieving a good measurement and the poor accuracy of the method under ideal conditions half cell is not viable for detecting strand corrosion in pretensioned concrete box beams.

- A comparison of surface acid soluble chloride measurements and half cell potential readings indicate that a poor correlation exists.
- An examination of the relationship between longitudinal cracking and corrosion was conducted. It was found that if a longitudinal crack is present, there is a 70.4% probability of having corrosion underneath. If corrosion does exist under the crack, there is an 87.7% probability that the type of corrosion will be pitting or heavier; thus, reducing the cross section of the prestressing strand at that location.
- When there are no surface indicators of corrosion (no crack) there is a 10.3% probability of finding corrosion on the prestressing strands underneath; further there is a 4.3% probability of finding heavy damage (heavy pitting through fracture) when no cracking is present.
- When corrosion damage was found at the 1st level of strands, damage was present on the 2nd level of strands 45.1% of the time. Conversely, damage was not present 54.9% of the time when damage is found on the 1st level of strands.
- Where corrosion damage was present for both levels of steel, it is shown that the damage index is larger on the 1st level of strands 87% of the time.
- When a longitudinal crack is present on the bottom beam surface, an adjacent strand has a 30.5% probability of having corrosion; further there is a 15.8% probability of having heavy corrosion (heavy pitting through fracture) on adjacent strands to longitudinal cracks. Therefore, discounting a strand adjacent to a longitudinal crack is conservative.
- On average the current condition of the beams reduces the flexural capacity to 85.2% of the undamaged capacity. The PennDOT recommendation reduces the capacity to 76.3% of the undamaged condition. The new recommendation reduces the capacity to 82.1% of the undamaged capacity. Based on this comparison the existing recommendation is overly conservative and the new recommendation provides a conservative estimate of the remaining strength.
- The proposed rating recommendation was used to estimate the flexural strength of the beams acquired. Comparing the in-situ strength with the probability based strength reductions it is clear that the proposed approach yields slightly conservative yet accurate results.
- The proposed rating recommendation and current PennDOT rating method were used by SEI to calculate the Inventory and Operation ratings for the Ash Street Bridge in Lackawanna County, PA. The average percent difference for an Inventory Rating between the two methods is 387.5%. The average percent difference for an Operating Rating between the two methods is 75.9%. These results indicate that the proposed rating method provides a significantly less conservative approach than what is currently being used for adjacent box beams.

9 References

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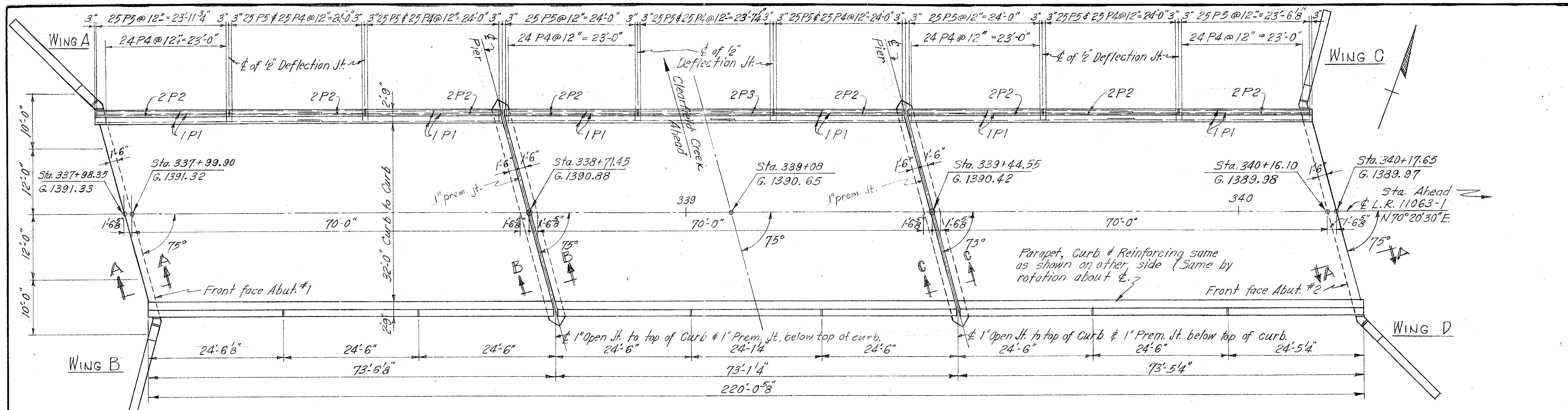
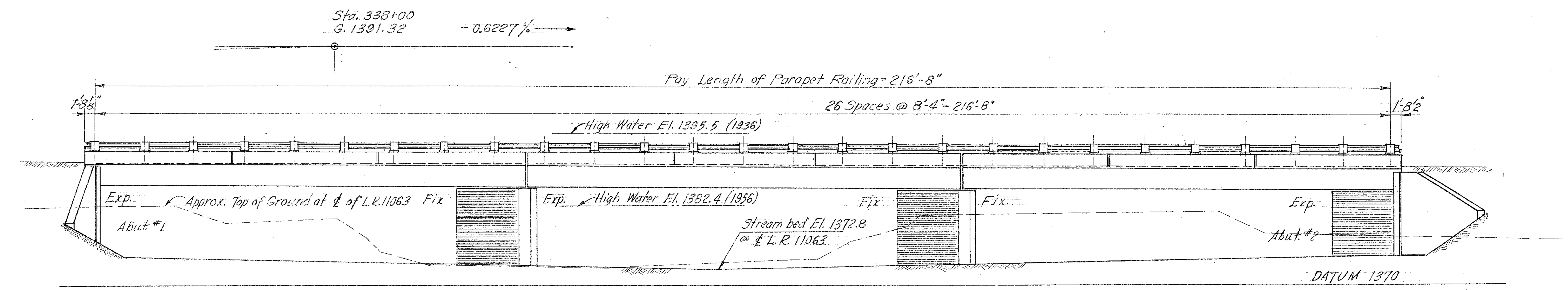
Appendix A

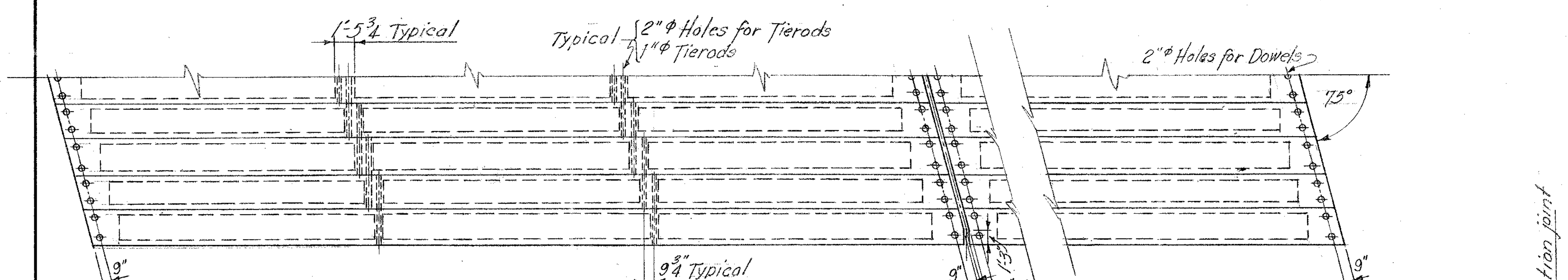
Bridge Shop Drawings and DOT Details



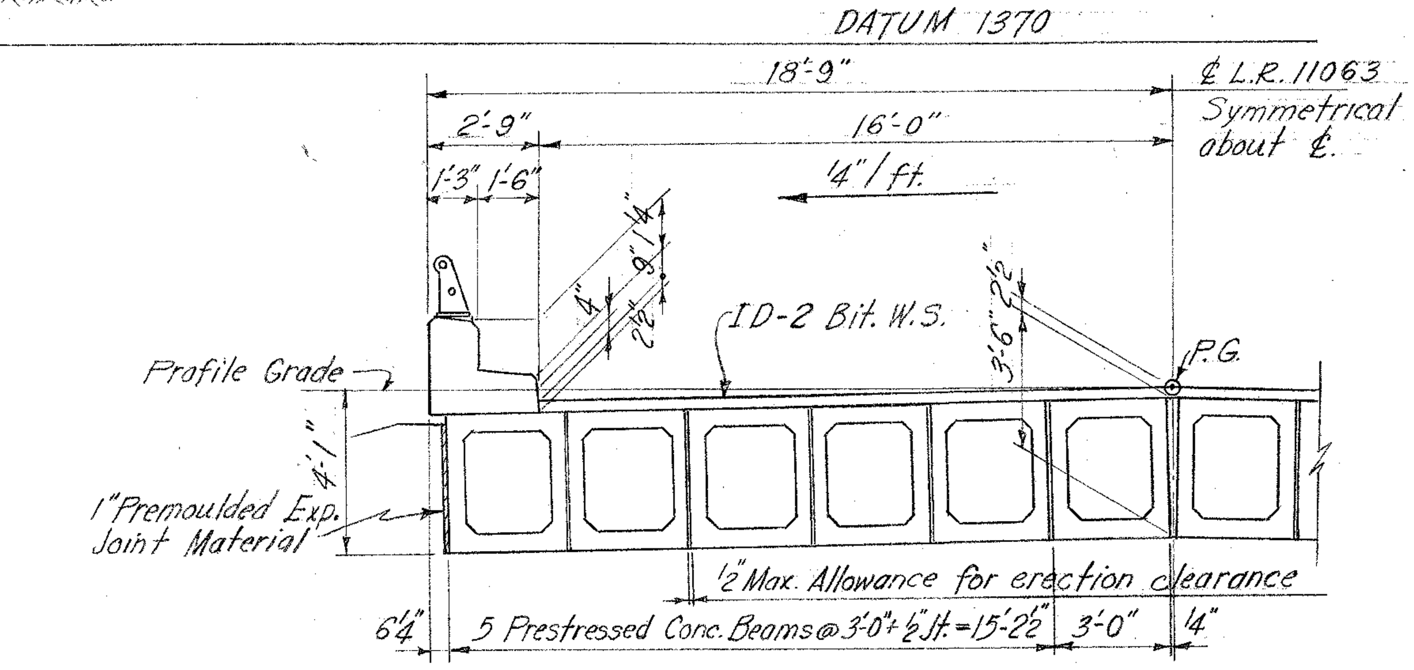
PLAN
Scale: 3/32" = 1'-0"



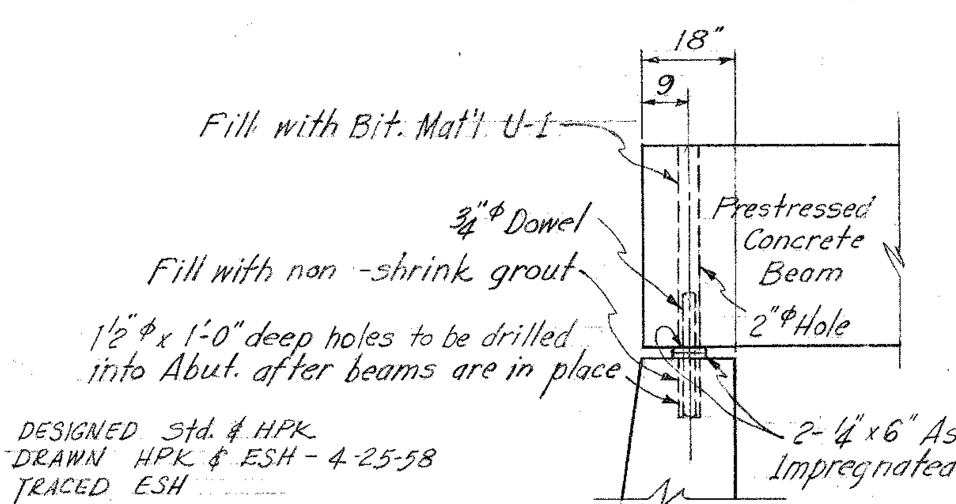
ELEVATION
Scale: 3/32" = 1'-0"



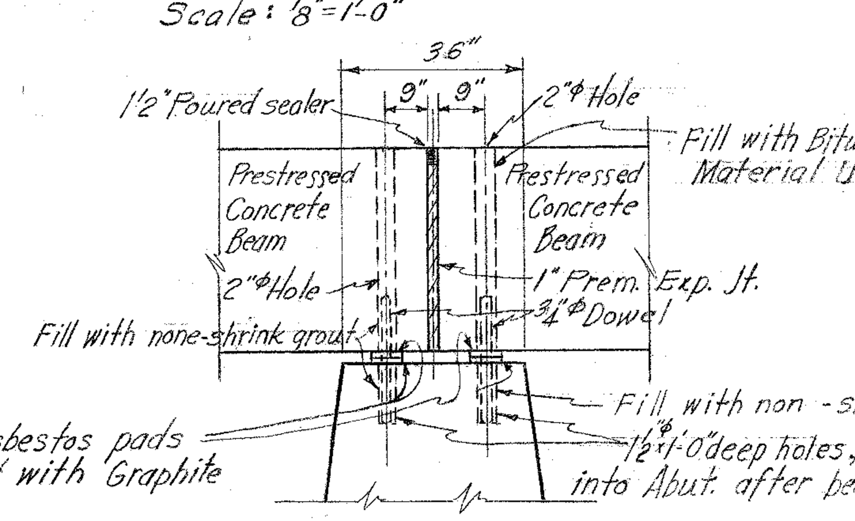
PART PLAN SHOWING DIAPHRAGM TIERODS
Scale: 1/8" = 1'-0"



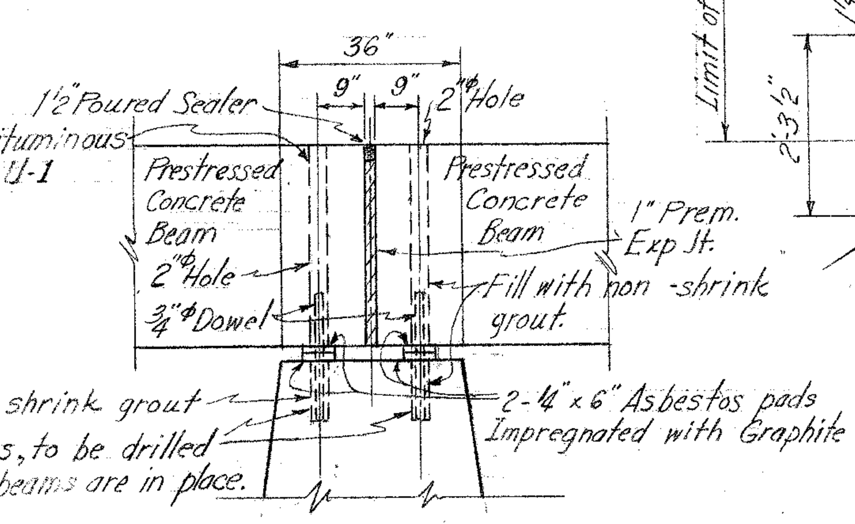
TYPICAL SECTION
Scale: 1/4" = 1'-0"



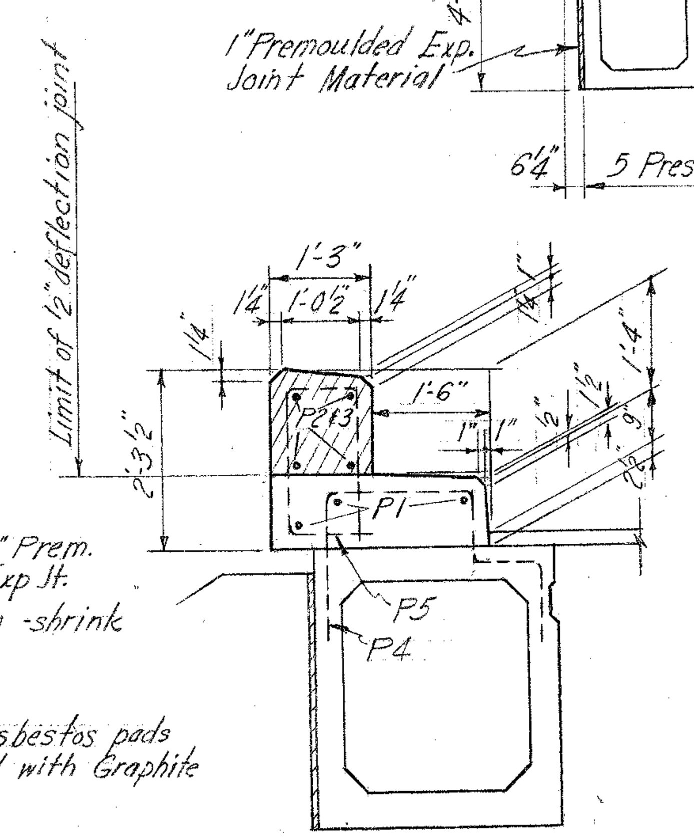
SECTION A-A
Scale: 3/8" = 1'-0"



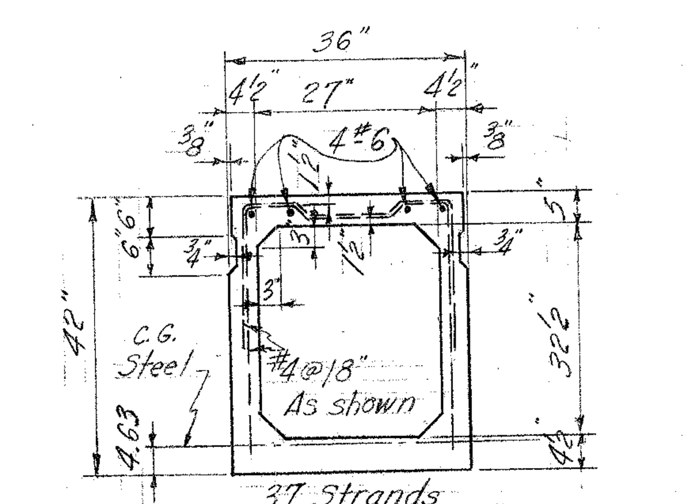
SECTION B-B
Scale: 3/8" = 1'-0"



SECTION C-C
Scale: 3/8" = 1'-0"



DETAIL OF CURB BEAMS
Scale: 1/2" = 1'-0"



PRESTRESSED BEAM SECTION
Scale: 1/2" = 1'-0

SUPERSTRUCTURE BAR SCHEDULE				
MARK	No.	Size	LENGTH	BENDING
P1	36	4	37'-0"	Straight
P2	64	4	24'-0"	Straight
P3	8	4	23'-7"	Straight
P4	442	4	5'-6"	1'-10", 9"
P5	450	4	5'-8"	1'-11"
D1	144	6	2'-0"	Dowels Straight

SUMMARY OF QUANTITIES							
Item	Unit	Abut. #1	Abut. #2	Pier #1	Pier #2	Superstructure	Total
Class III Excavation	c.y.	410	437	131	131	—	1109
Class A Concrete	c.y.	—	—	—	—	60	60
Class B Concrete	c.y.	221	218	125	125	—	689
Reinforcement Bars	lbs.	11,023	10,398	3,522	3,522	5,810	34,275
Prestress. Conc. Br. Deck	sq. ft.	—	—	—	—	7996	7996
Parapet Railing	L.F.	—	—	—	—	434	434
Guard Fence Connections	each	—	—	—	—	4	4
Bit. Surface Course - ID-2 2½" Depth	sq. yd.	—	—	—	—	780	780
Stone Backfill for Structures	c.y.	9	9	—	—	—	18
Fabricated Structural Steel	lbs.	—	—	295	295	—	590

GENERAL NOTES:

- All materials and workmanship shall be in accordance with P.D.H. forms 408-54, 409-49 & Sect. 6.24 Supplement for prestressed concrete bridge superstructure,
- Design Specs: Divison III of 1953 "Standard Specs. for Highway Bridges" of the A.A.S.H.O.
- Live Load: H-20-S-16-44.
- Steel Reinforcement Bars designed for fs = 18,000 p.s.i and detailed as per A.C.I. Code. Bars to be lapped min. 40 Dia. except as noted.
- Class "A" concrete shall be used in roadway curbs and parapets. All other concrete shall be Class "B" unless otherwise noted.
- Base of footings may be ordered by the Engineer to be at any elevation, or of any dimensions to provide proper foundation.
- Two-coat painted waterproofing shall be applied to rear faces of walls as directed by the Engineer.
- Provide 2" Cover on reinforcement bars unless otherwise noted.
- For Parapet railing & guard fence connections see P.H.D. Dwgs. S-1614B & S-3361
- Exposed conc. edges to be chamfered 1"x1" unless specified by the Engineer.

PRESTRESSED NOTES

- Min. Tensile Strength of steel shall be 250,000 p.s.i.
- Min. 28-day cylinder strength shall be 5,000 p.s.i.
- Initial force at each strand shall be 14,000 lbs.
- Min. strength of conc. at release of prestress shall be 4000 p.s.i.

APPROVED APR 28 1958
BRIDGE ENGINEER

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

CAMBRIA COUNTY

L.R. 11063-1 STATION 339+08
3 SPAN (70'-0" + 70'-0" + 70'-0") PRESTRESSED CONC. BRIDGE
SKEW 75° R.H. Ahd.

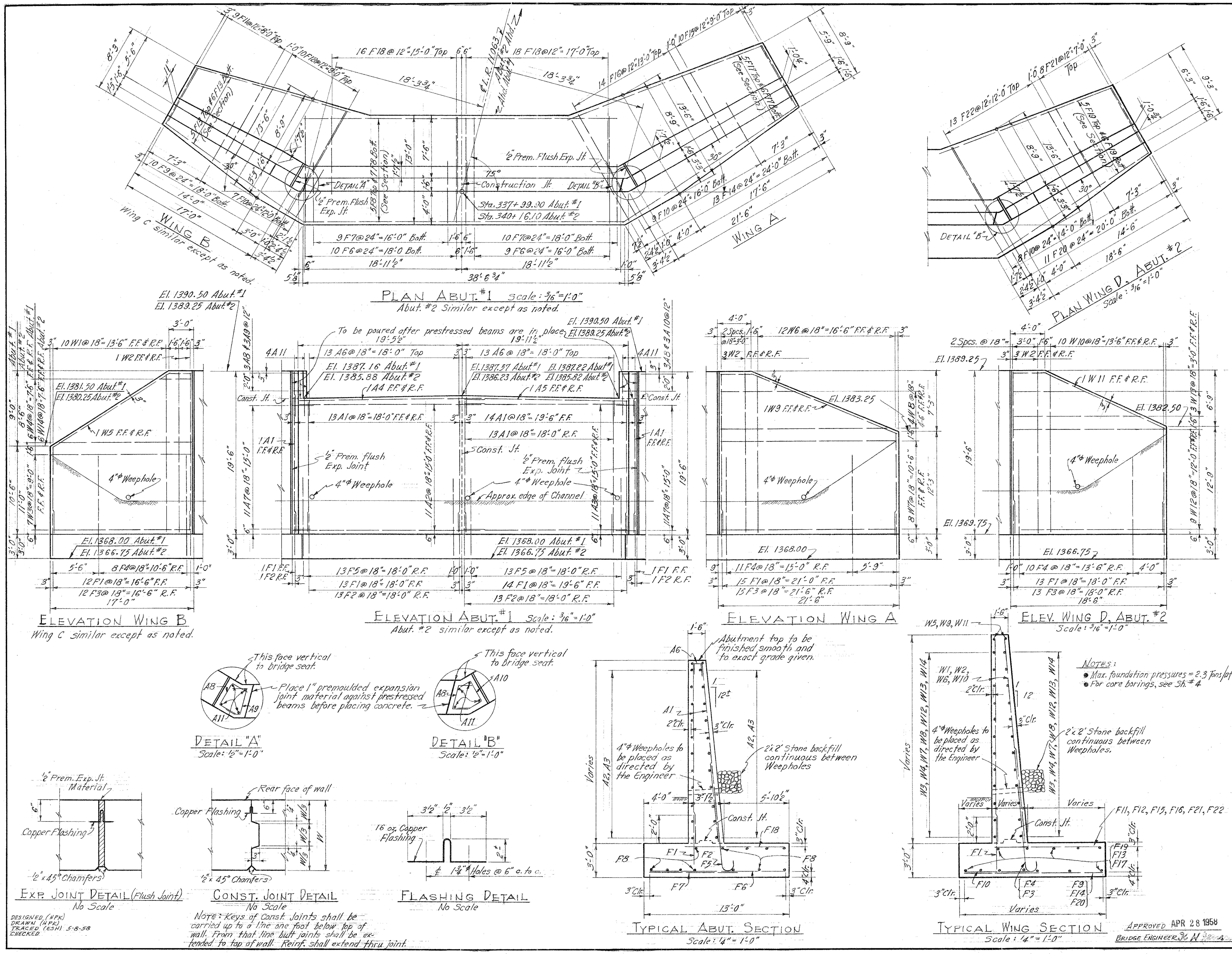
GENERAL PLAN

DESIGNED Std. & HPK
DRAWN HPK & ESH - 4-25-58
TRACED ESH
CHECKED

SCALE: AS NOTED
SHEET No. 1 OF 4 + S-1614-B, S-3361

S-3346

APR 28 1958



ABUTMENT BAR SCHEDULE

MARK	NUMBER ABUT. #1	NUMBER ABUT. #2	SIZE	LENGTH	BENDING
F1	56	54	5	4'-0"	Straight
F2	28	28	9	8'-3"	7'-0"
F3	27	25	9	7'-9"	6'-6"
F4	19	18	7	13'-0"	12'-2"
F5	26	26	6	10'-3"	9'-7"
F6	19	19	8	12'-6"	Straight
F7	19	19	8	7'-3"	Straight
F8	12	12	5	39'-0"	Straight
F9	10	10	5	7'-6" to 14'-3"	Vary 1 each by 9"-Straight
F10	16	15	7	6'-0"	Straight
F11	9	9	7	7'-3" to 8'-11"	Vary 1 each by 2½"-Straight
F12	10	10	9	9'-1½" to 11'-0"	Vary 1 each by 2½"-Straight
F13	11	11	5	20'-3"	Straight
F14	13	—	5	8'-3" to 14'-3"	Vary 1 each by 6"-Straight
F15	10	—	7	7'-6" to 8'-7½"	Vary 1 each by 1½"-Straight
F16	14	—	9	9'-10½" to 11'-6"	Vary 1 each by 1½"-Straight
F17	11	—	5	24'-6"	Straight
F18	34	34	7	9'-0"	Straight
F19	—	11	5	21'-6"	Straight
F20	—	11	5	8'-7" to 14'-0"	Vary 1 each by 6½"-Straight
F21	—	8	7	7'-9" to 8'-11"	Vary 1 each by 2"-Straight
F22	—	13	9	9'-9" to 11'-9"	Vary 1 each by 2"-Straight
W1	20	20	5	10'-3" to 19'-3"	Vary 2 each by 12"-Straight
W2	10	4	5	19'-3"	Straight
W3	14	14	5	16'-6"	Straight
W4	12	—	5	3'-6" to 15'-7"	Vary 2 each by 2'-5"-Straight
W5	2	2	7	19'-0"	16'-6", 2'-1½", 2'-6", 1'-4"
W6	24	—	5	12'-3" to 18'-8"	Vary 2 each by 7'-0"-Straight
W7	16	—	5	21'-0"	Straight
W8	8	—	5	8'-6" to 19'-6"	Vary 2 each by 3'-8"-Straight
W9	2	—	7	22'-3"	18'-6", 3'-5½", 3'-9", 1'-5"
W10	—	10	5	12'-6" to 19'-3"	Vary 2 each by 9"-Straight
W11	—	2	7	19'-6"	15'-9", 3'-4", 3'-9", 1'-6½"
W12	—	18	5	18'-0"	Straight
W13	—	6	5	8'-6" to 15'-0"	Vary 2 each by 3'-3"-Straight
W14	—	12	5	3'-6" to 16'-0"	Vary 2 each by 2'-6"-Straight
A1	57	57	5	16'-0"	Straight
A2	22	22	5	21'-3"	Straight
A3	22	22	5	19'-0"	Straight
A4	2	2	7	22'-0"	Straight
A5	2	2	7	19'-0"	Straight
A6	26	26	4	4'-0"	1'-5", 1'-1", 9", 1'-5", 1'2"
A7	22	22	4	4'-0"	1'-2", 1'-4", 9", 9", 7¾", 4½"
A8	6	6	4	3'-2"	1'-2", 1'-0"
A9	3	3	4	3'-2"	4½", 9", 1'-0", 7¾", 3", 3½", 1½", 1'-1½"
A10	3	3	4	3'-7"	4½", 9", 1'-0", 7¾", 8", 3½", 1½", 1'-1½"
A11	8	8	5	5'-3"	Straight

Commonwealth of Pennsylvania

Department of Highways

BRIDGE UNIT

CAMBRIA COUNTY

L.R. 11063-1 STATION 339+08

3-SPAN (70+70+70) PRESTRESSED CONCRETE BRIDGE

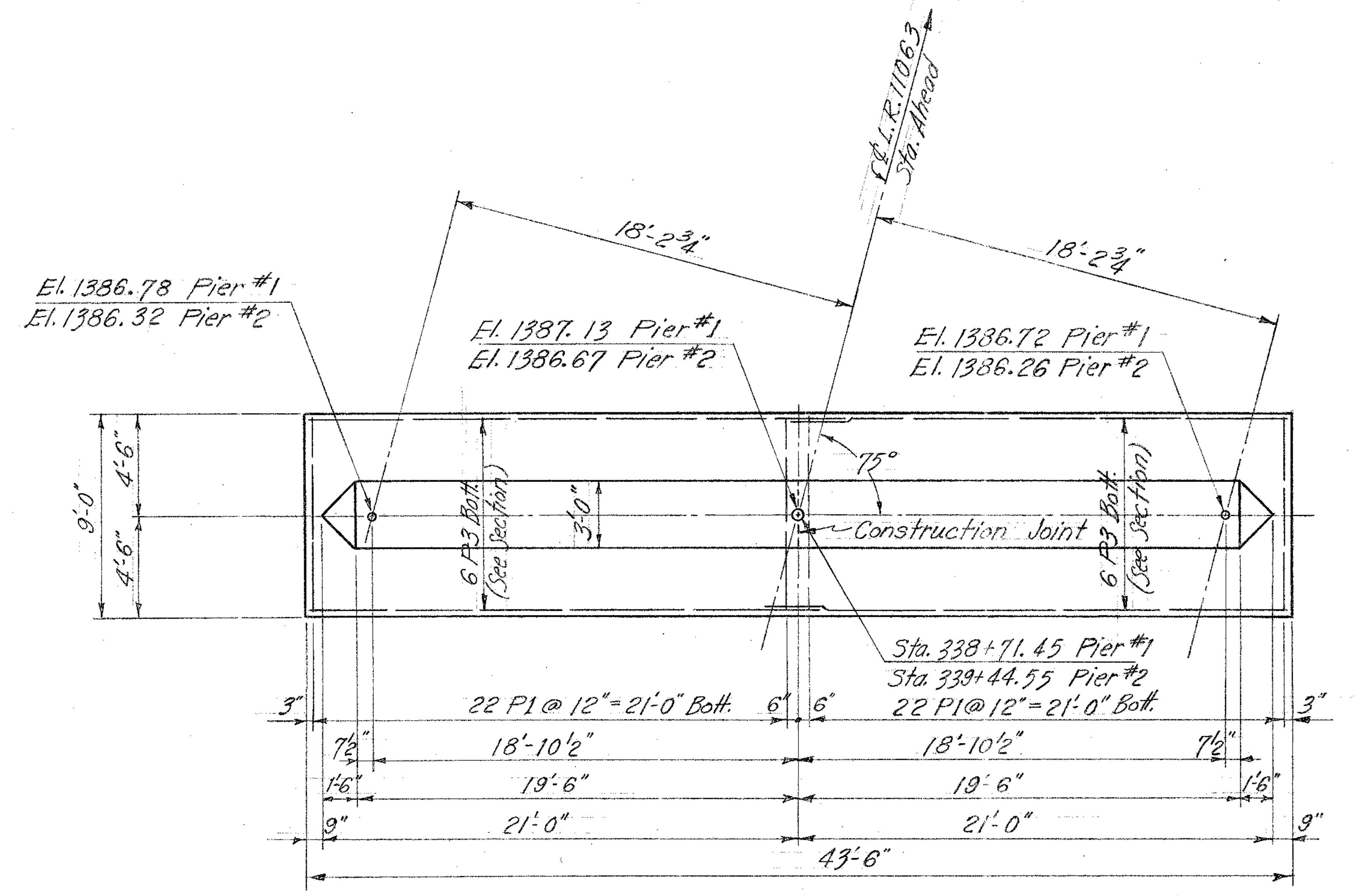
SKEW 75° R.H. Ahd.

ABUTMENTS #1 & #2

SCALE: AS NOTED

SHEET 2 OF 4

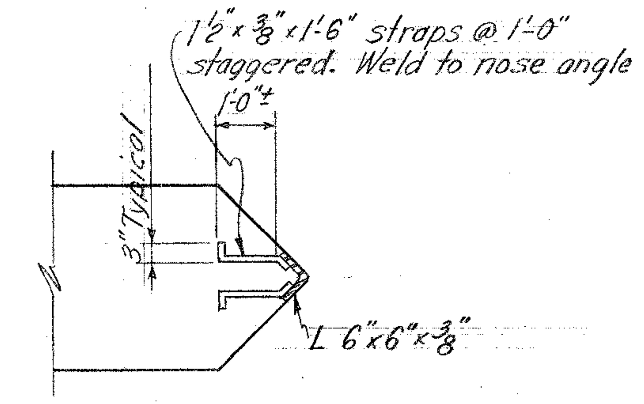
S-3346



PLAN PIER #1

Pier #2 Similar except as noted

Scale: 3/16" = 1'-0"



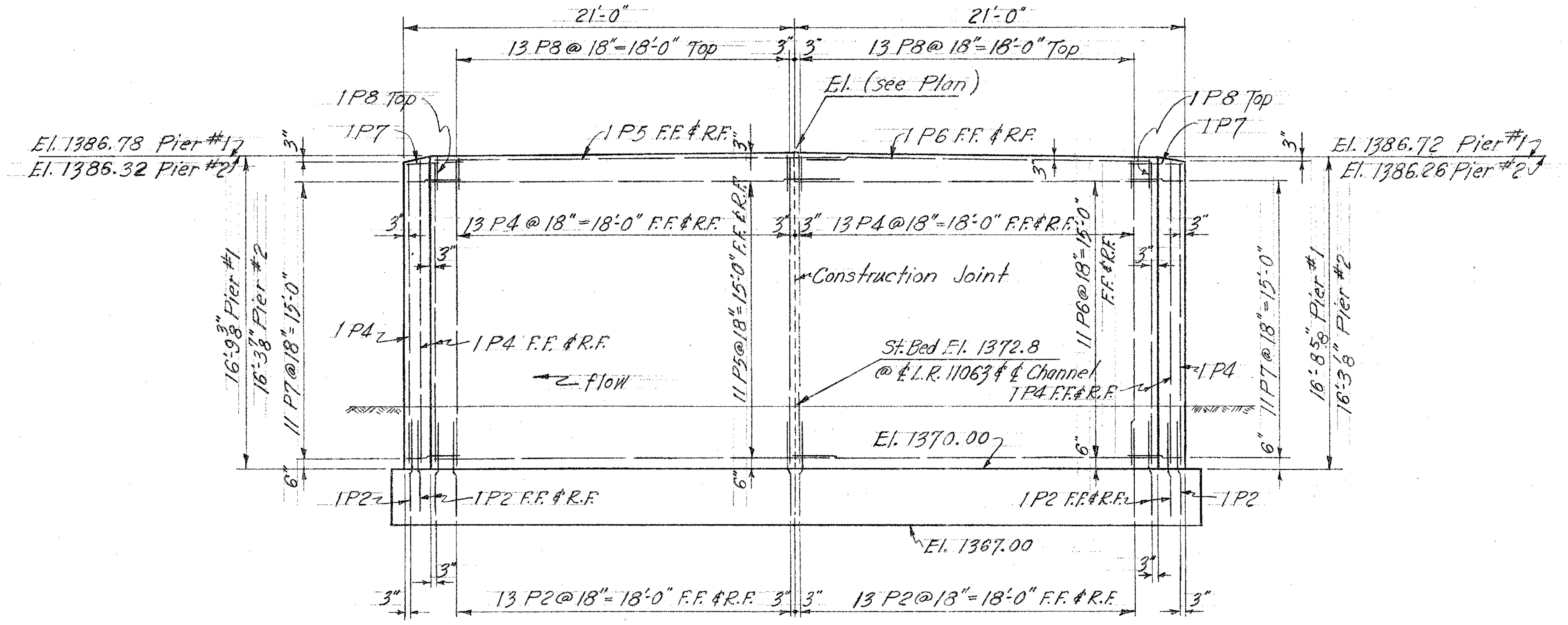
PIER NOSE DETAIL

NOTE: Place angle on upstream end of Piers only.

Scale: ⅜" = 1'-0"

MARK	No. PIER #1	No. PIER #2	SIZE	LENGTH	BENDING
P1	44	44	6	8'-6"	Straight
P2	62	62	5	5'-9"	5'-2"
P3	12	12	5	22'-6"	Straight
P4	62	62	5	15'-10"	Straight
P5	24	24	5	21'-6"	Straight
P6	24	24	5	19'-0"	Straight
P7	24	24	4	7'-6"	1'-3" 1'-3" 1'-9" 2'-6" 1'-3" 2'-0"
P8	28	28	4	6'-6"	2'-6" 2'-0"

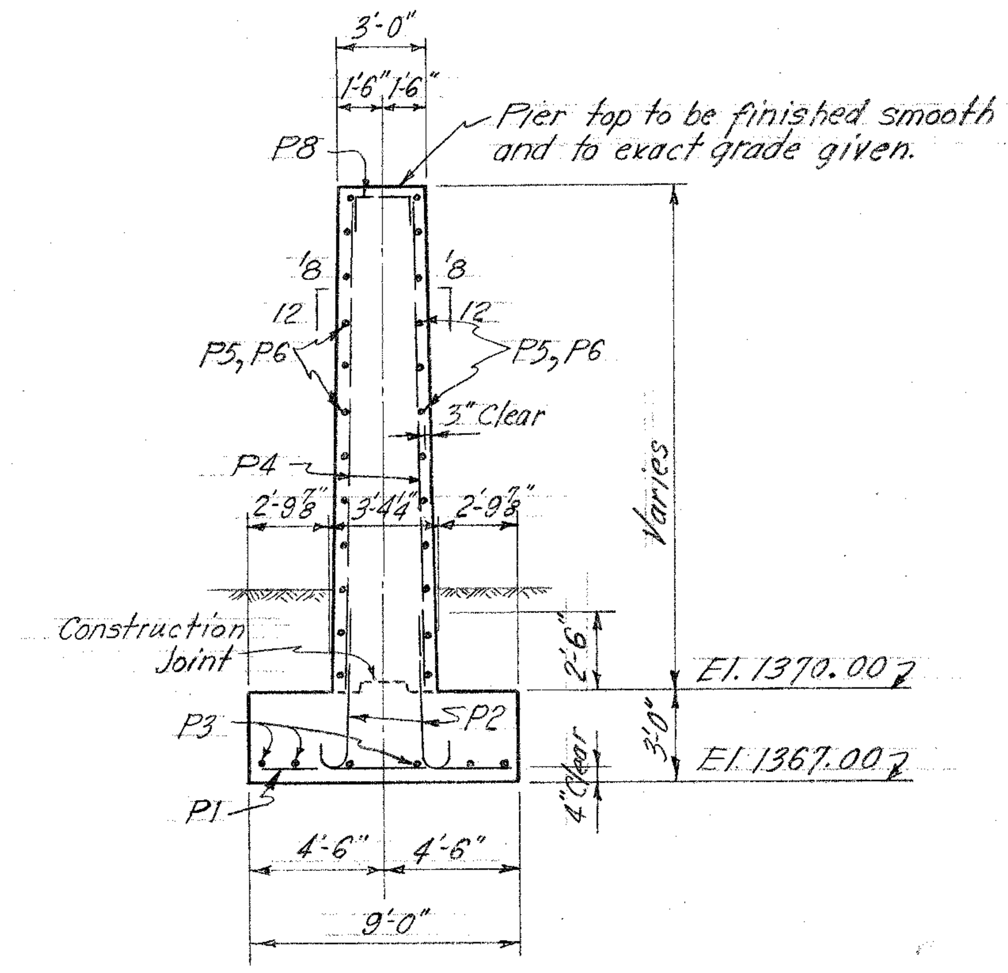
PIER BAR SCHEDULE



ELEVATION PIER #1

Pier #2 Similar except as noted.

Scale: 3/16" = 1'-0"



TYPICAL SECTION

Scale: 3/16" = 1'-0"

NOTES:
For Core Borings see Sheet #5
For Construction Joint Detail see Sheet #2
Max. Foundation Pressure 2.1 Tons / □ ft.

APPROVED APR 28 1958

BRIDGE ENGINEER H. H. Jensen

DESIGNED SAS
DRAWN HPK
TRACED ESH (4-30-58)
CHECKED

Commonwealth of Pennsylvania

Department of Highways

BRIDGE UNIT

CAMBRIA COUNTY

L.R. 11063-1 STATION 339+08

3 SPAN (70'-0" + 70'-0" + 70'-0") PRESTRESSED CONC. BRIDGE

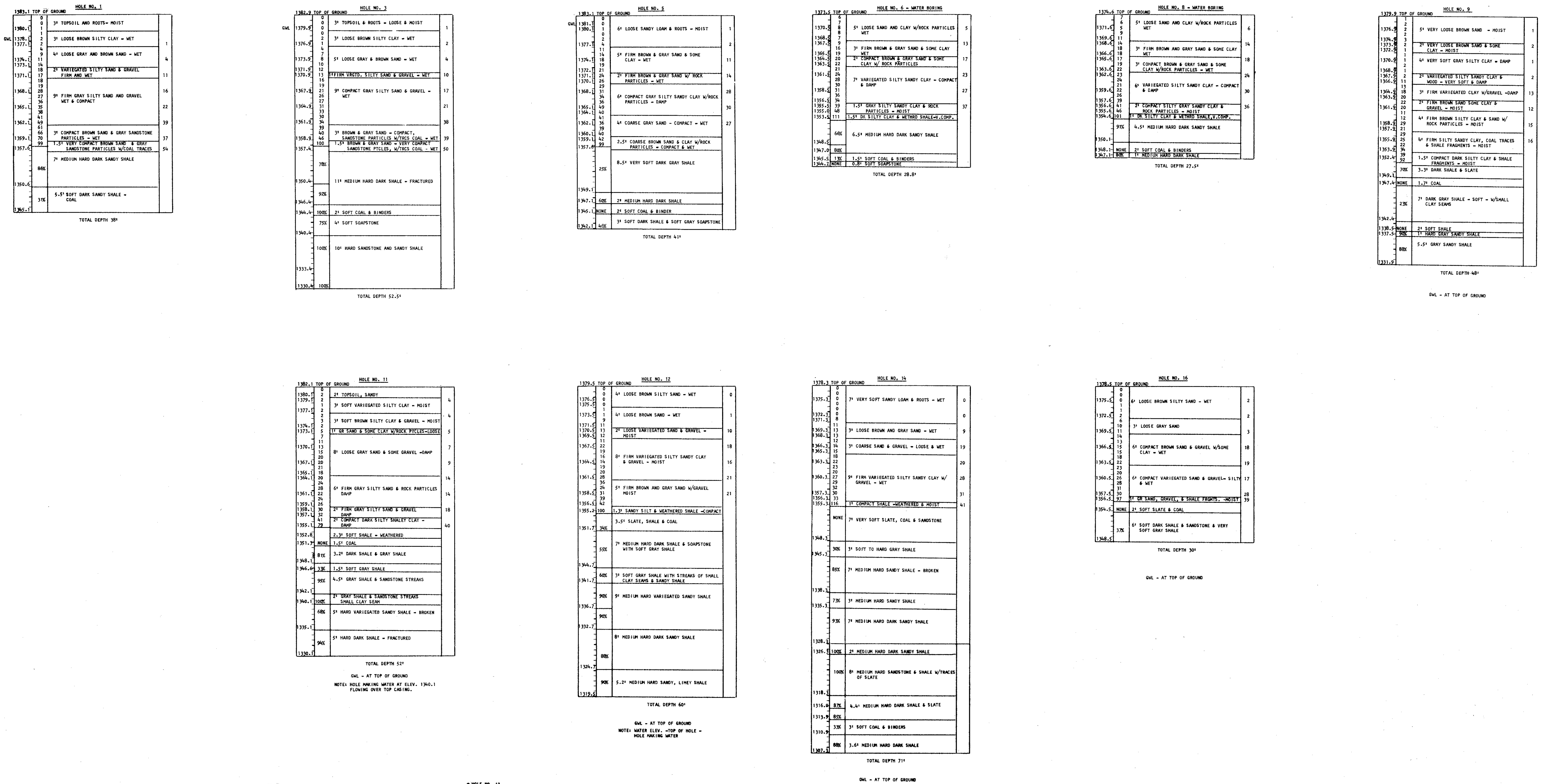
SKEW 75° R.H. Ahd.

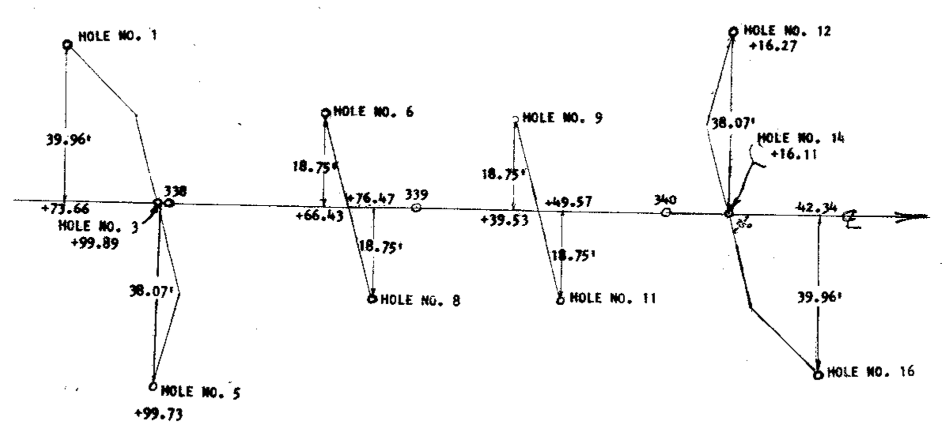
PIERS #1 & #2

SCALE: As Noted

SHEET 3 OF 4

S-3346





LOCATION OF HOLE BORINGS FOR BRIDGE AT STA. 339+

GENERAL NOTES

GWL	- WATER LEVEL
FIRST LEFT COLUMN	- ELEVATIONS IN FEET
SECOND LEFT COLUMN	- BLOWS PER FOOT ON 4" CASING, USING 380 LB. HAMMER, 16" DROP; AND PERCENTAGE OF CORE RECOVERY
RIGHT COLUMN	- BLOWS PER FOOT ON 2" SPLIT SPOON SAMPLER USING 140 LB. HAMMER, 30" DROP

TEST BORINGS

PENNA. DEPT. OF HIGHWAYS DISTRICT-9

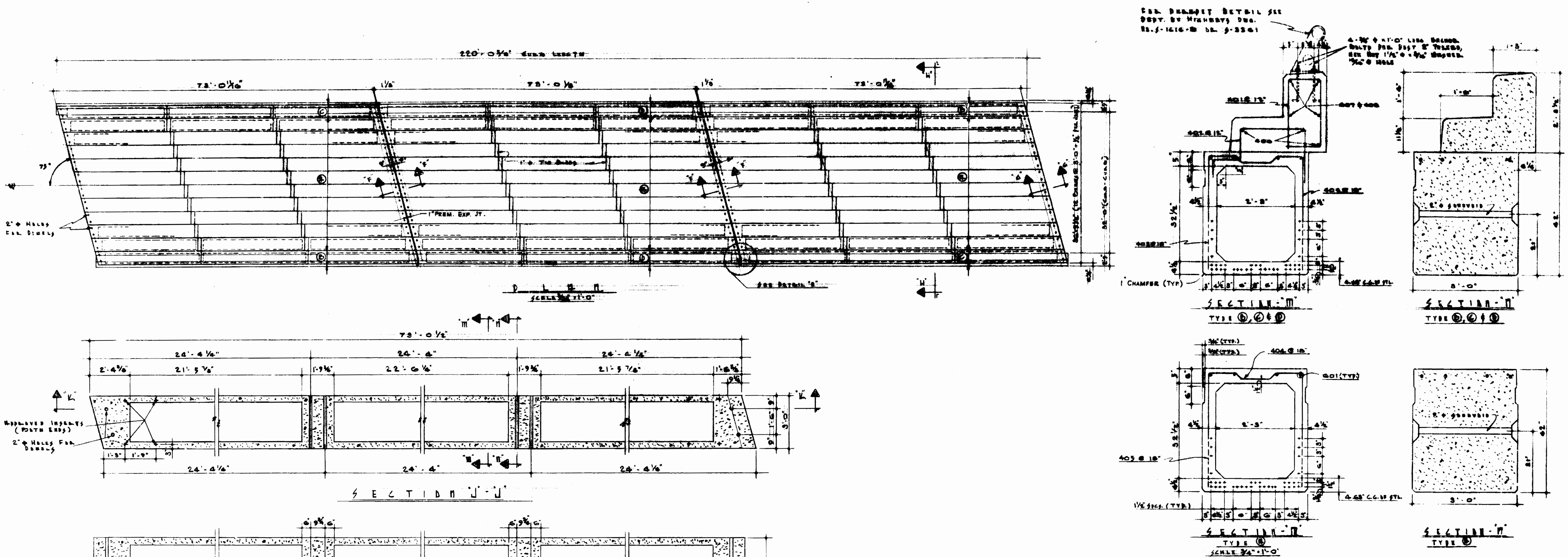
L.R. 11063, SEC. I CAMBRIA COUNTY

BRIDGE AT STA. 339+

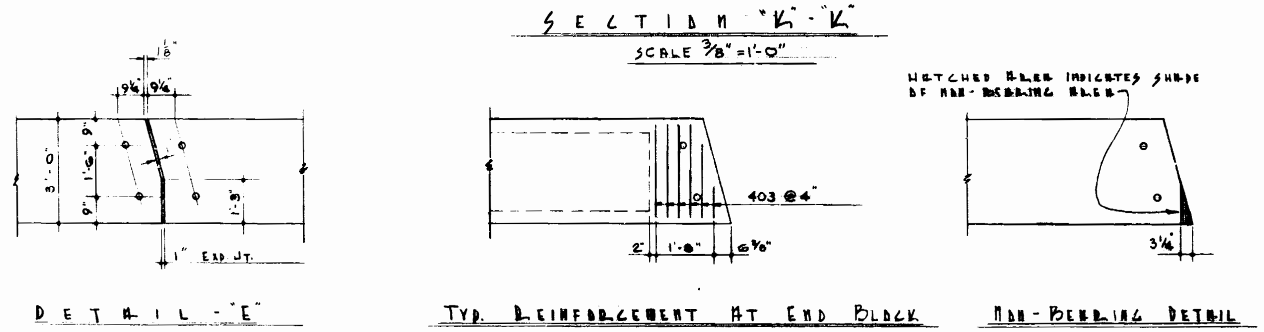
EQUIPMENT & SUPPLIES, INC.
PILE DRIVING & DRILLING CONTRACTORS

S-3346

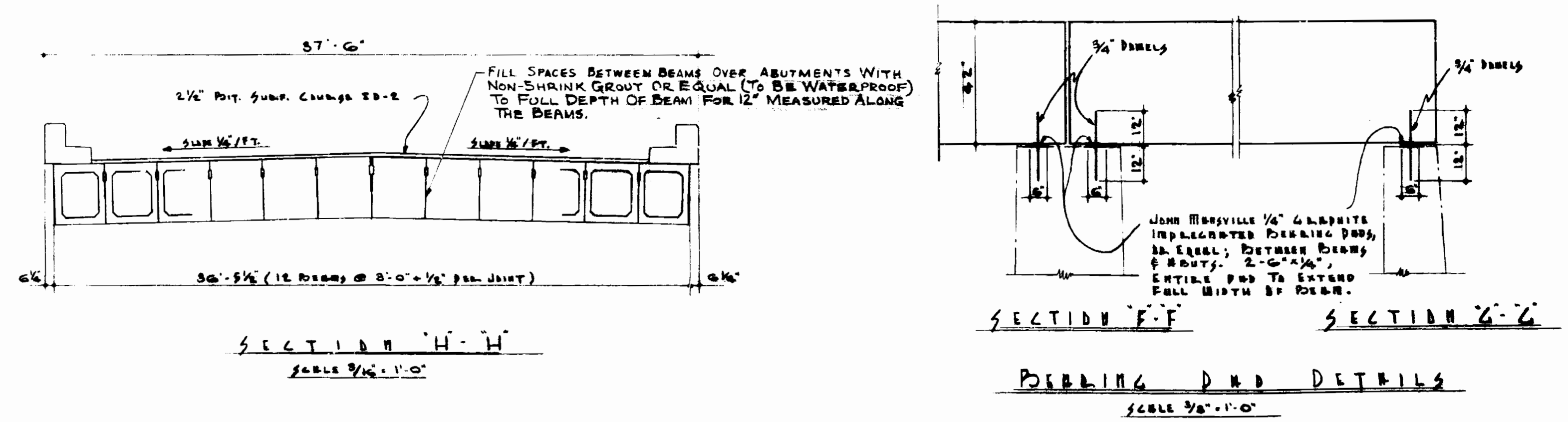
SHEET 4 OF 4 SCALE 1" = 10' JUNE 2, 1958



PLAN



DETAIL "E" — TYP. REINFORCEMENT AT END BLOCK (Scale 3/8" = 1'-0") — NON-BEARING DETAIL



SECTION "H"-"H" (Scale 3/16" = 1'-0")

SECTION "F"-"F" — SECTION "G"-"G"

BEARING PAD DETAILS (Scale 3/8" = 1'-0")

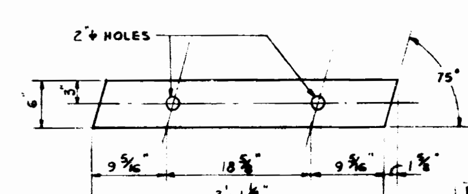
BAR SCHEDULE									
MARK NO.	401	402	403	404	405	406	407	408	601
SIZE ϕ	#4	#4	#4	#4	#4	#4	#4	#4	#6
NO. REQ'D.	442	450	432	1692	1656	36	66	8	144
LENGTH	5'-8"	9'-6"	Varies 8'-2" - 11'-10"	6'-6"	9'-4"	37'-0"	24'-0"	23'-7"	72'-9"
DIAGRAM						STRAIGHT BARS			

16. ALL Materials & Workmanship Shall be in accordance with P.D.H. Form 408-54 and Section 6.24 Supplement for Prestressed Concrete Bridge Superstructure Dated 1-10-58.

GENERAL NOTES

1. Design Load: H20-S16-44
2. Design Span: 70'-0"
3. Prestressed Units: 3/8" ϕ High Tensile Steel Strand Required: 37 Strands – A_s 2.96 in.^{2}
4. Concrete Strength 5,000 # @ 28 Days
5. Initial Prestressing 175,000 p.s.i.
6. Designed in Accordance with B.P.R. Criteria for Prestressed Concrete Bridge Superstructures (Section 6.24)
7. Tension at Bottom Fiber – 0
8. Maximum Compressive Stress 2000 p.s.i.
9. Principal Tension < 150 p.s.i. Standard Load; < 300 p.s.i. Ultimate Load
10. All Cables to Have 1 1/2" Min. Cover.
11. After the Prestressed Members are in place 1 1/2" holes 12" deep shall be drilled in the substructure under the 2" sleeves in the beams, 3/4" Anchor Dowels 30" long shall be placed in the holes, and the holes filled with Asphalt.
12. Seat Elevations & Slope to be verified in field prior to setting beams.
13. Camber to be checked in field.
14. Concrete Strength at Release of Prestress = 4000 P.S.I.
15. Minimum Ultimate Tensile Strength of Strands 250,000 p.s.i.

COMMONWEALTH OF PENNSYLVANIA — REGISTERED PROFESSIONAL ENGINEER



BEARING PAD DETAIL

SCALE 1" = 1'-0"

NOTE: USE CONTINUOUS BEARING PADS TO PREVENT LEAKAGE AT ABUTMENTS. BUTT SPLICES BETWEEN PADS WILL BE PERMITTED BUT MUST BE WATERPROOFED.

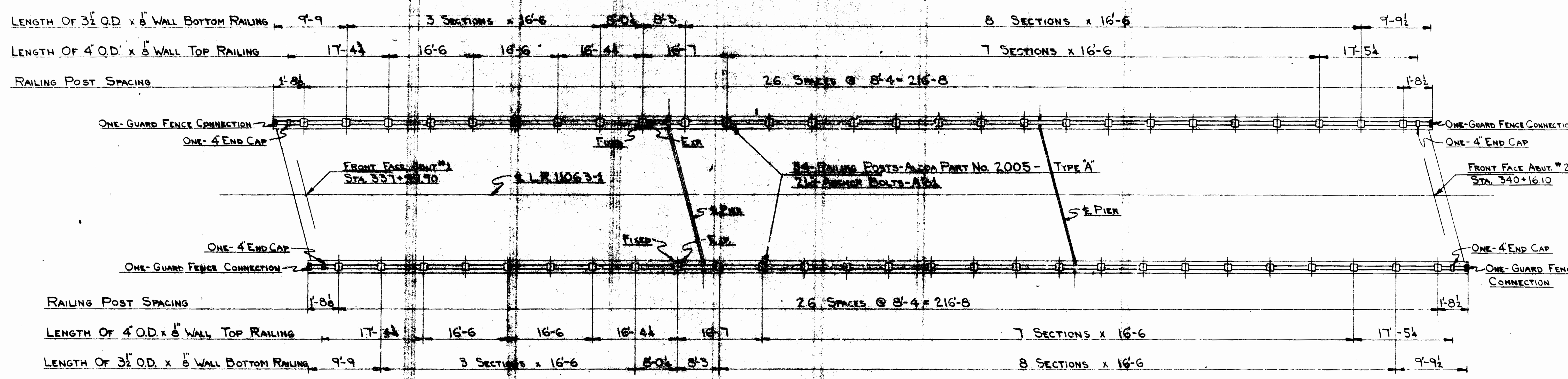
PENNA. DEPT. OF HWYS. APPROVED AS TO DESIGN NOV 19 1958

NEW ENTERPRISE STONE & LIME CO., INC.
PRESTRESSED CONCRETE DIVISION

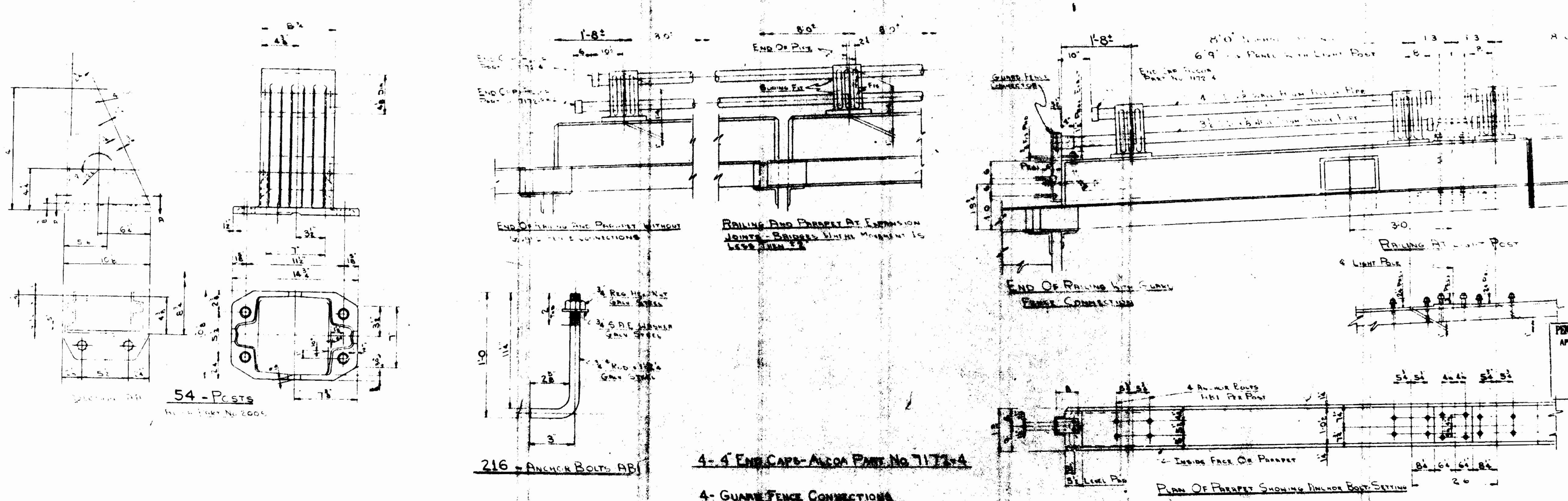
Frederic R. Harris & Co. Inc., Baltimore, Md. — Consulting Engrs., Chambersburg, Pa.

Route 11063 Sect. 1 Clarion Co.
Prestressed Bridge Members over Clearfield Creek
Sta. 339+08 — 3 Span 70'-0" - 70'-0" - 70'-0"

Job: 58-71	Drawn:	Scale - As Noted
Date: 10/6/58	Checked: R.A.K.	Sheet 1 of 1
Reference Dwg. S 3346		P-339



L.R. 11063-1 Sta. 339+08



Length Of Railing This Sheet 434 Lin. Ft.

GENERAL NOTES

1. GENERALLY TUBES SHALL BE TWO (2) PANELS LONG. JOINTS IN UPPER AND LOWER RAILS SHALL BE STAGGERED. JOINTS TO BE AT CENTERLINE OF POSTS WITH 2" CLEARANCE, EXCEPT AT EXPANSION JOINTS.
2. 1/2" HEADLESS SET SCREWS SHALL BE PROVIDED AT EACH POST FOR TOP AND BOTTOM RAILS TO PREVENT RATTLING.
3. ALL PARAPETS & RAILINGS ARE PARALLEL TO HORIZONTAL AND VERTICAL ALIGNMENT OF ADJACENT CURB.
4. ALL RAILING POSTS SHALL BE NORMAL TO GRADE. ALL RAILING POSTS AND LIGHT POLE BASES SHALL BE LEVEL.
5. FOR SPACING OF RAILING POSTS, REFER TO DETAIL DRAWINGS OF STRUCTURE INVOLVED OR RAILING ERECTION DIAGRAMS.
6. FOR LARGE SCALE DETAIL OF RAILING POST -- SEE SHEET 2, DWG. S-1614-8.
7. BASES SHALL BE GROUTED AFTER ERECTION AND ALIGNMENT TO SEAL OPENINGS BETWEEN BASE AND CONCRETE.
8. ANCHOR BOLTS FOR RAILING POSTS SHALL BE SET NORMAL TO GRADE.
9. GUARD FENCE CONNECTION ANCHOR BOLTS AND RAILING POST ANCHOR BOLTS SHALL BE GALVANIZED STEEL A.S.T.M. SPEC. A153.
10. RAILING TUBES, POSTS, SET SCREWS, AND GUARD FENCE CONNECTION SHALL BE ALUMINUM ALLOY
11. RAILING TO CONSIST OF ALUMINUM ALLOY (6061-T6) HIGHWAY RAILING ROUND TUBE AND SHALL CONFORM TO A.S.T.M. SPEC. B235-50T, ALLOY GS11A, CONDITION T6.
12. POSTS SHALL BE PENNA. HIGHWAY DEPT. S-1614-8, ALUMINUM ALLOY PERMANENT MOLD CASTINGS (356-T6) AND SHALL CONFORM TO A.S.T.M. SPEC. B108-50T, ALLOY SG70A, CONDITION T6.
13. GALV. STEEL WASHERS TO BE STEEL GALV. A.S.T.M. SPEC. A153.
14. ALUMINUM ALLOY PLATE SH [illegible] (6061-T6) SHALL CONFORM TO A.S.T.M. SPEC. B209-51T, ALLOY G[illegible]IA, CONDITION T6.
15. CAULKING COMPOUND SHALL BE EITHER 'ALUMILASTIC' AS MANUFACTURED BY PARR PAINT & COLOR CO. OF CLEVELAND, OHIO or 'PERMAGUM' AS MANUFACTURED BY THE PRESSTITE ENGINEERING CO. OF ST. LOUIS, MISSOURI, OR APPROVED EQUAL.

PENNA. DEPT. OF HWYS.
APPROVED AS TO DESIGN
OCT 28 1958
ENGINEER OF BRIDGES

APPROVED:

L. B. FOSTER CO.

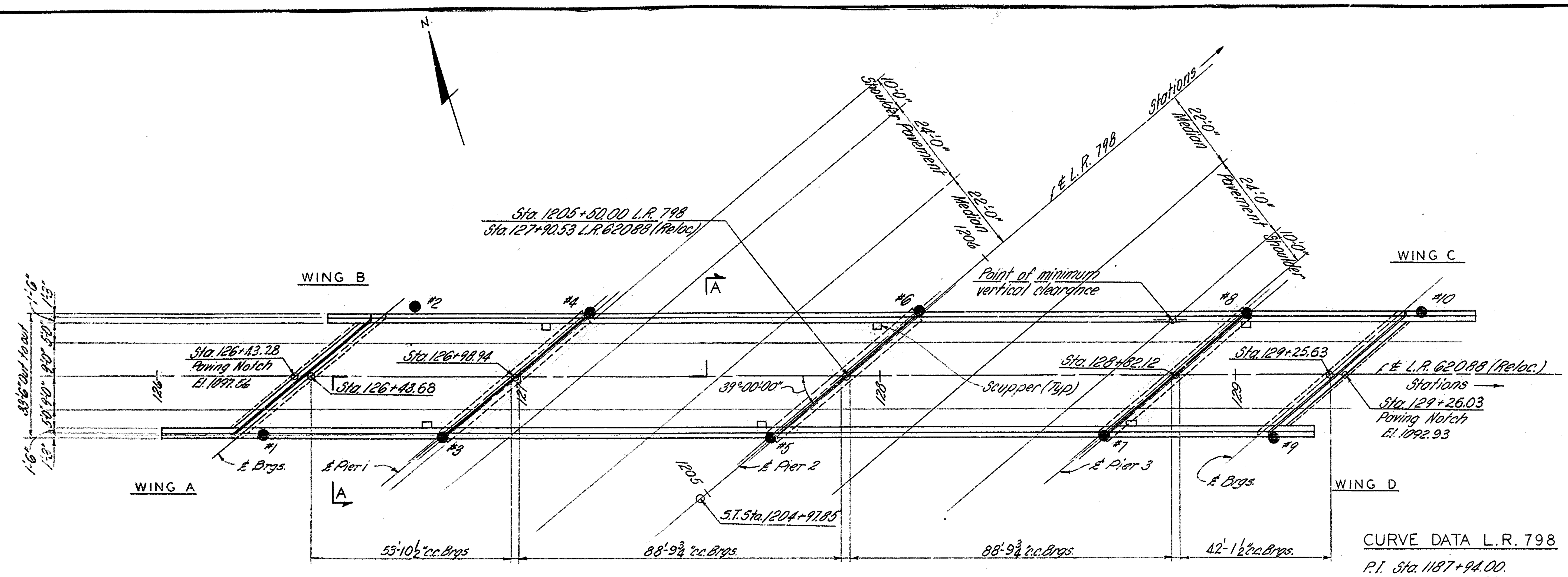
ATLANTA - CHICAGO - HOUSTON - LOS ANGELES - NEW YORK - PITTSBURGH

FOR: New Enterprise Stone & Lime Co.
Pennsylvania Dept. Of Highways
Project: L.R. 11063-(1) Cambria County
Alum. Parapet Railing, Gen. Notes & Details
For L.R. 11063-1 Sta. 339+08

MADE P.L. DATE 10-2-58 JOB NO AL-16358 ORDER NO P-21589
CHECK T.S. DATE 10-58 DRAWING LB-2 (LAST) CUSTOMER'S NO P-339

REVISIONS

INSPECTION BY

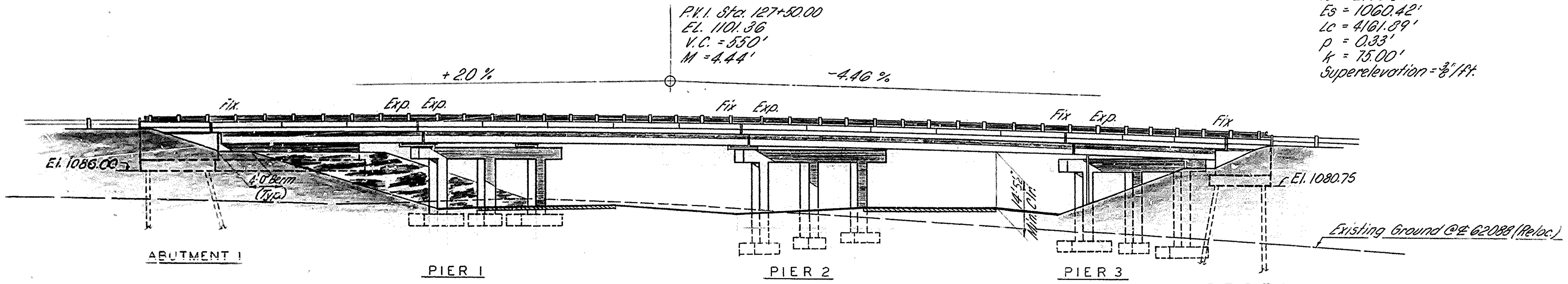


● — Indicates core boring location

PLAN
Scale 1" = 20'

CURVE DATA L.R. 798

P.I. Sta. 1187+94.00
Δ = 86°-14'-16" Lt.
D = 2°00'
Rc = 2864.93
Ls = 150.00'
θs = 1°30'
Ts = 2758.04'
Es = 1060.42'
Lc = 4161.89'
p = 0.33'
k = 75.00'
Superelevation = ⅜"/ft.



ELEVATION
Scale 1" = 20'

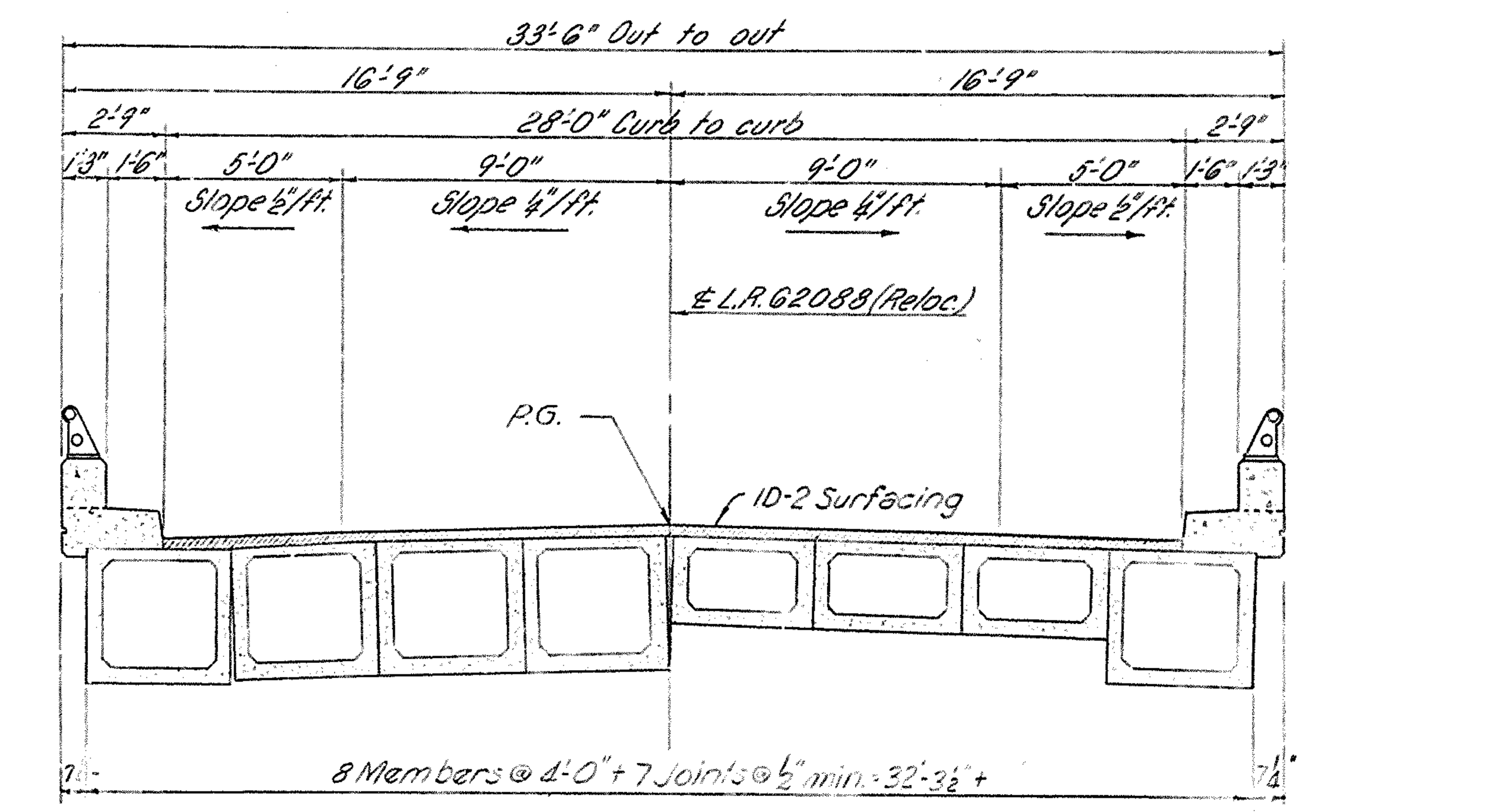
NOTES:

1 — Live load: H 20-S16-44.
2 — Design: - A.A.S.H.O. Standard Specifications for Highway Bridges (1953)
Earth pressure: 120 lbs./cu.ft. vertical; 35 lbs/cu.ft. fluid pressure horizontal

3 — Specifications: — Penna. Department of Highways Forms 408 (1954) and 409 (1949).
4 — All superstructure concrete shall be Class A. All abutment concrete shall be Class B except wingwall parapets which shall be Class A. All concrete in piers shall be Class A except footings which shall be Class B.
4a. Embankment shall be thoroughly compacted up to bottom elevation of Abutment before driving piles.
5 — All exposed concrete edges shall have a 1"×1" chamfer unless noted.
6 — Reinforcement bars are designed for fs = 18,000 psi & are detailed in accordance with A.C.I. recommendations.
7 — All reinforcement bars carrying design or temperature stresses shall be lapped at least 40 diameters where splicing is required.
8 — The bases of the structure may be ordered by the Engineer to be at any elevation or of any dimensions necessary to provide a proper foundation.
9 — Place two-coat painted waterproofing on top of footing heels and on rear face of abutment walls and wingwalls up to a line 1'-0" below top of finished grade.
10 — Fabricated structural steel shall be A.S.T.M. designation A7, current.
11 Provide 2" Cover on reinforcement bars unless otherwise Noted.

12 — W.P. - denotes Work Point.
13 — For parapet railing and guard fence connections see P.D.H. Std. Dwgs. S-1614-B and S-3361.
14. - Prestressed, Precast beams shall be designed in accordance with Pennsylvania Department of Highways Specifications Form 408 (1954) Supplement Section 6.24 (1958). Ultimate strength of strand equals 250,000 p.s.i.; initial prestress in strands equals 175,000 p.s.i.

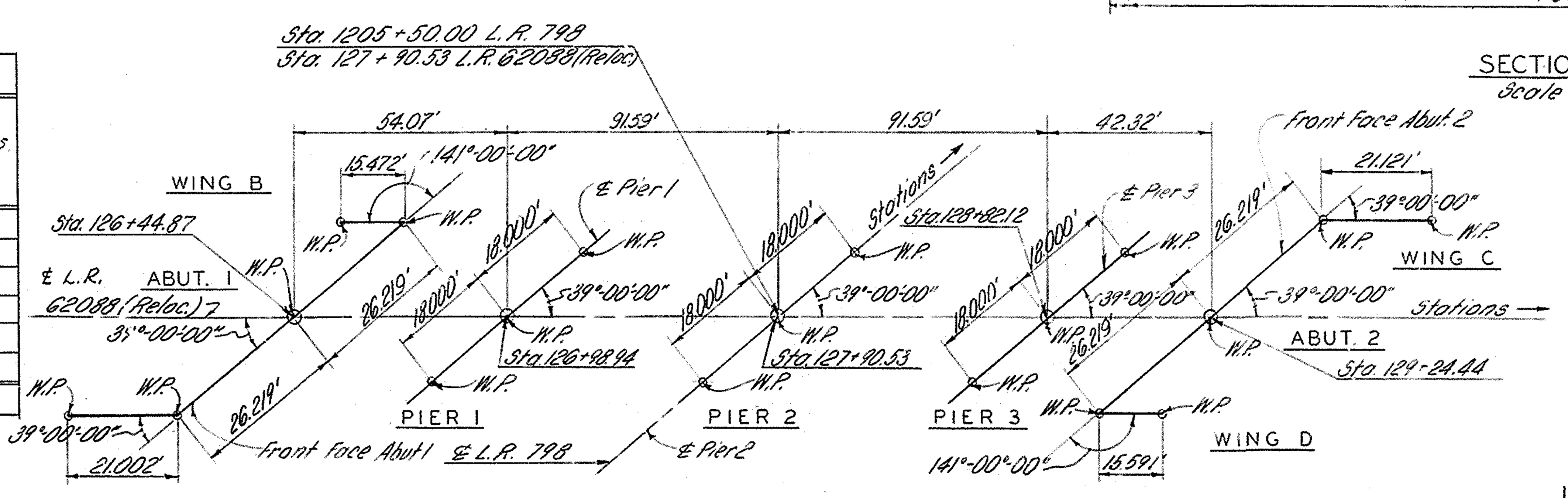
15. — All piles shall be steel beam piles 10BP42.
16. — Piles shall be driven to refusal or to practical refusal as directed by the Engineer but, in no case, to a bearing value less than 72 Tons for Test Piles and 60 Tons for 10BP42.
17. - All dowels in Pier Cap shall be set in preformed holes. Drilling of holes for dowels in Pier Cap will not be permitted.
18. - All Parapet, Parapet Railing will be considered a part of the Bridge Superstructure.



SECTION A-A
Scale ¼" = 1'-0"

QUANTITIES											
Item	Cu. Yds Class 3 Excavation	Cu. Yds. Class A Concrete	Sq. Ft. Prestressed Precast Conc. Box Beam	Cu. Yds. Class B Concrete	Lbs. Reinforcement Bars	Lbs Fabricated Structural Steel	Lin. Ft. Parapet Railing	Steel Beam Piles 10 BP 42		Each Guard Fence Connections	Sq. Yds. Bituminous Surface ID-2
								Test Piles	Lin. Ft.		
Abutment 1	—	4	—	104	5,530	—	27	1 @ 20'	300	2	—
Pier 1	98	46	—	25	13,840	—	—	—	—	—	—
Pier 2	89	49	—	18	17,440	—	—	—	—	—	—
Pier 3	74	53	—	25	15,310	—	—	—	—	—	—
Abutment 2	—	4	—	108	5,590	—	27	1 @ 32'	470	2	—
Superstructure	—	114	9,143	—	8,770	6450	570	—	—	—	878
Totals	261	270	9,143	280	66,480	* 6450	624	Lump Sum	770	4	878

* Includes: 3720 Lbs. Malleable iron or Cast Steel
2340 Lbs Wrought iron



STAKE-OUT PLAN
Not to Scale

These drawings supersede S-3661 Approved June 16, 1959

DESIGNED F.S.F.
DRAWN W.W.W.
TRACED W.W.W.
CHECKED E.R.L.

PRESTRESS ALTERNATE REVISIONS
BY
MACOMBER & FABER
CONSULTING ENGINEERS
HARRISBURG, PENNA.

PREPARED BY
GROF & MYERS
CONSULTING ENGINEERS
HARRISBURG, PENNA.

APPROVED JUN 13 1960
BRIDGE ENGINEER

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

WASHINGTON COUNTY

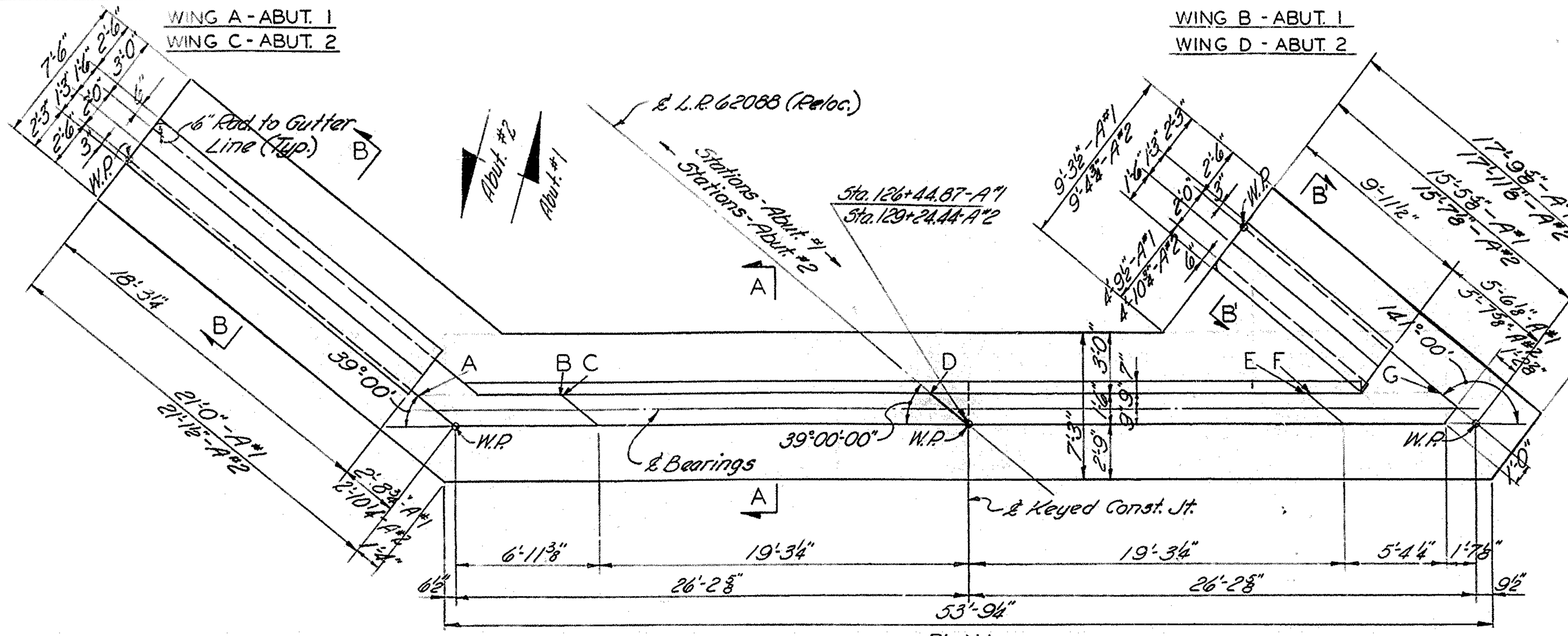
L.R. 798 SECTION 1

L.R. 798 UNDER L.R. 62088 (RELOC)

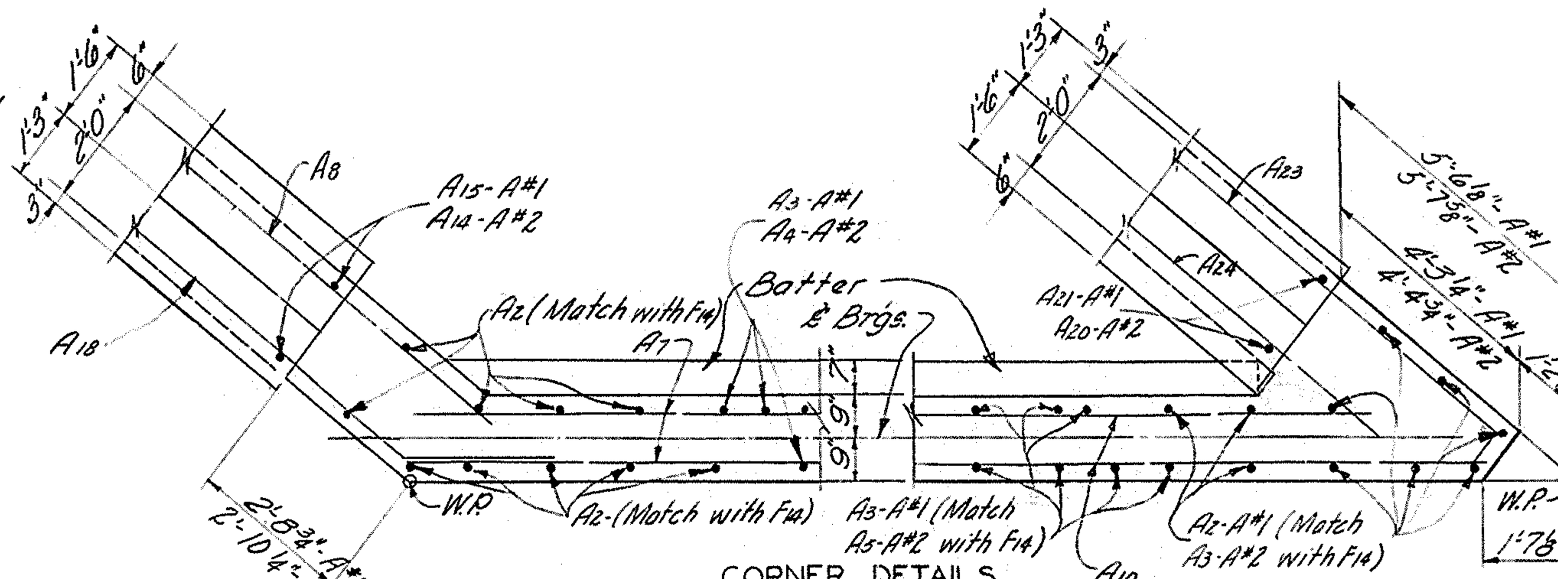
GENERAL PLAN
STA. 1205+50.00

SCALE: AS NOTED
SHEET 1 OF 8 + S-1614 B, S-3361

S-3661 A



PLAN
Scale: 3/16"=1'-0"



CORNER DETAILS
Scale: 3/8"=1'-0"

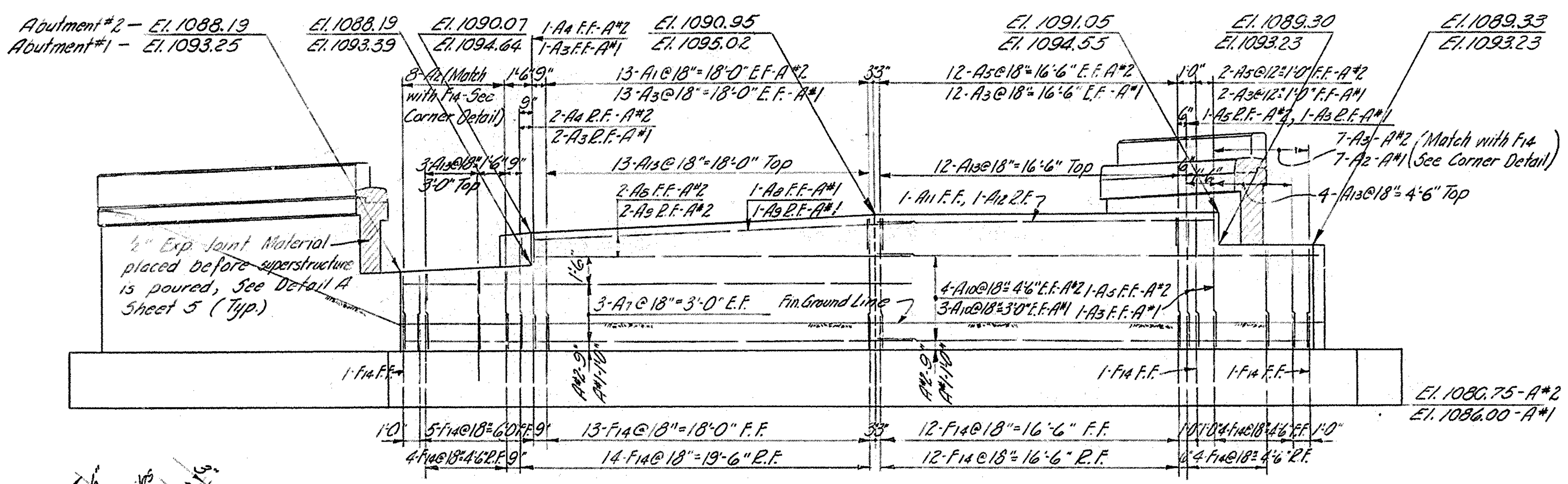
ELEVATION	A	B	C	D	E	F	G
ABUTMENT 1	1093.26	1093.40	1094.65	1095.03	1094.56	1093.24	1093.24
ABUTMENT 2	1088.12	1088.12	1090.00	1090.87	1090.99	1089.24	1089.28

Note "A": See note 8 sheet 5 for possible adjustment of bearing elevations due to beam camber.

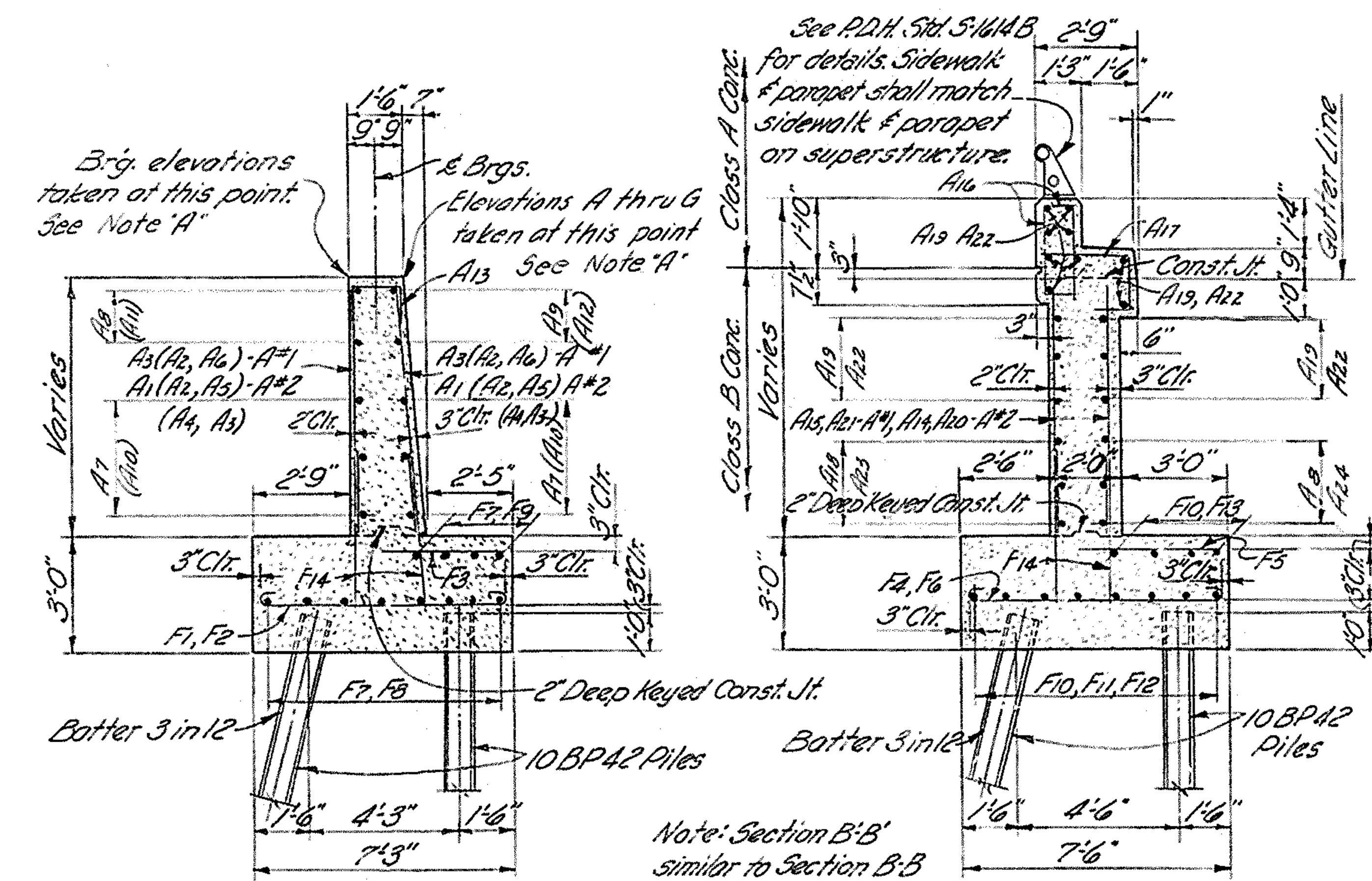
BAR SCHEDULE										
MK.	NUMBER A*1	NUMBER A*2	SIZE	LENGTH	TYPE	A	B	C	D	REMARKS
F1	76	38	5	8'-0"	1	7"	6'-10"	7"		
F2	38	70	5	4'-6"	1	7"	3'-11"	—		
F3	31	31	5	3'-6"	Str.					
F4	30	30	6	8'-3"	1	8"	6'-11"	8"		
F5	21	21	5	4'-0"	Str.					
F6	24	24	6	4'-9" to 10'-6"	1	8"	4'-1" to 9'-10"			vary 2 ea. by 3"
F7	12	12	5	28'-0"	Str.					
F8	8	8	5	15'-9" to 28'-0"	Str.					vary 2 ea. by 1'-9"
F9	4	4	5	18'-0"	Str.					
F10	12	12	5	22'-3"	Str.					
F11	4	4	5	17'-3"	Str.					
F12	4	4	5	13'-0" to 16'-9"	Str.					vary 2 ea. by 1'-3"
F13	4	4	5	8'-6"	Str.					
F14	118	118	5	4'-6"	Str.					
A1	—	26	5	6'-0" to 7'-0"	Str.					vary 2 ea. by 1"
A2	20	10	5	4'-0"	Str.					
A3	54	10	5	5'-4"	Str.					
A4	—	3	5	6'-0"	Str.					
A5	—	27	5	7'-0"	Str.					
A6	10	10	5	5'-0"	2-A	2'-0½"	11	2'-0½"	0	
A7	6	6	5	28'-0"	Str.					
A8	4	5	5	21'-3"	Str.					
A9	1	2	5	23'-0"	Str.					
A10	6	8	5	24'-3"	Str.					
A11	1	1	5	19'-0"	Str.					
A12	1	1	5	17'-3"	Str.					
A13	32	32	5	5'-3"	2-A	2'-1"	1'-1"	2'-1"	0	
A14	—	26	5	7'-2"	Str.					
A15	26	—	5	8'-0"	Str.					
A16	42	42	5	3'-3"	2-A	11"	2'-4"	0	0	
A17	21	21	5	4'-7"	2-A	2'-4"	1'-3"	1'-0"	0	
A18	3	3	5	23'-0"	2-A	2'-3"	20'-9"	0	10	
A19	15	15	5	17'-9"	Str.					
A20	—	16	5	8'-9"	Str.					
A21	16	—	5	8'-0"	Str.					
A22	15	15	5	9'-6"	Str.					
A23	3	4	5	14'-6"	2-A	9"	13'-9"	0	0	
A24	3	4	5	12'-3"	Str.					

BAR TYPES

TYPE 1 TYPE 2-A



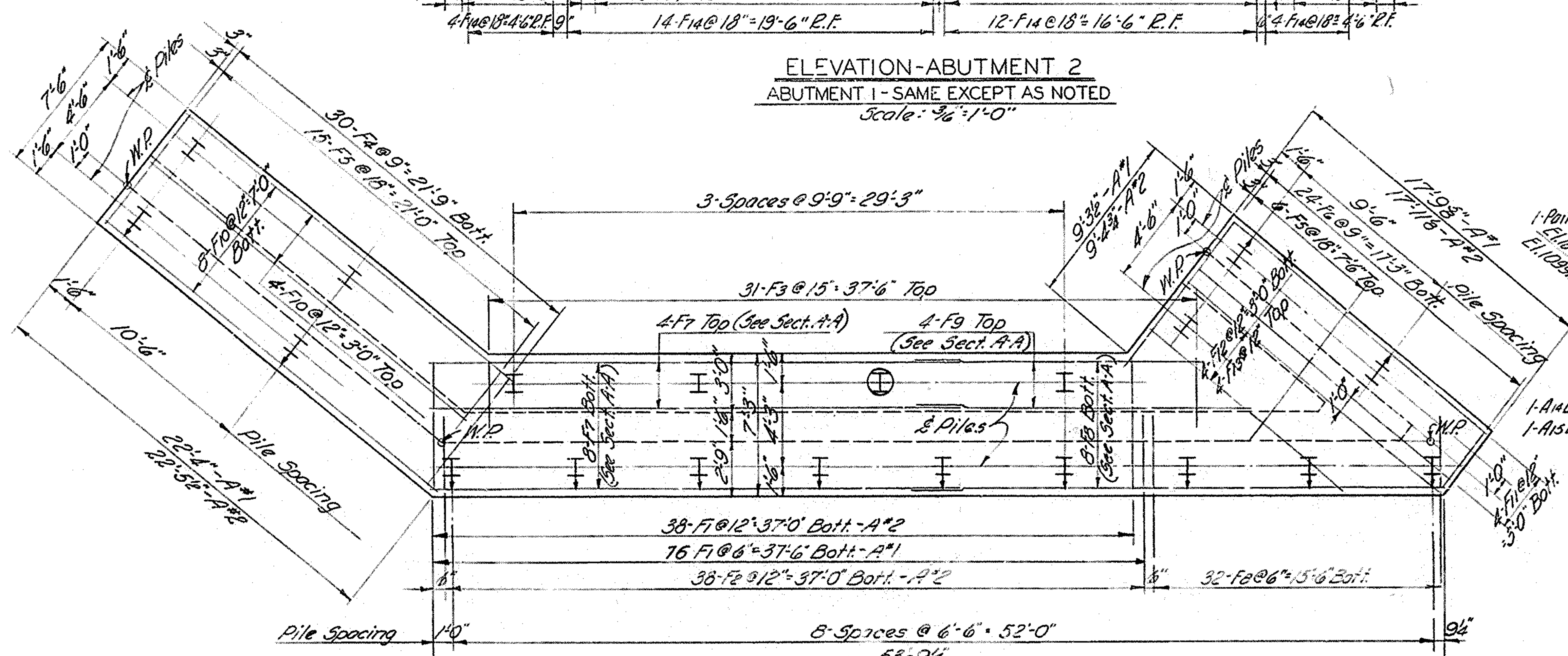
ELEVATION-ABUTMENT 2
ABUTMENT 1 - SAME EXCEPT AS NOTED
Scale: 3/16"=1'-0"



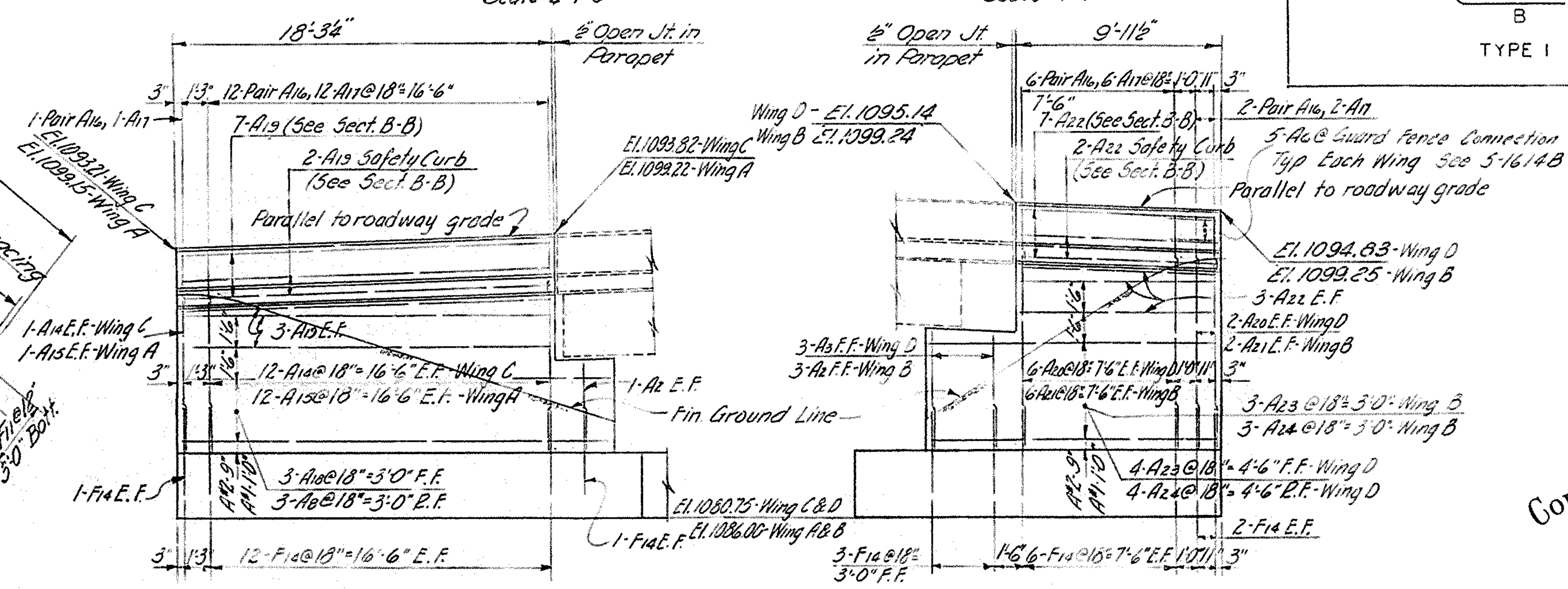
SECTION A-A
Scale: 1/4"=1'-0"

Note: Section B'B' similar to Section B-B except for footing dimensions.

SECTION B-B
Scale: 1/4"=1'-0"



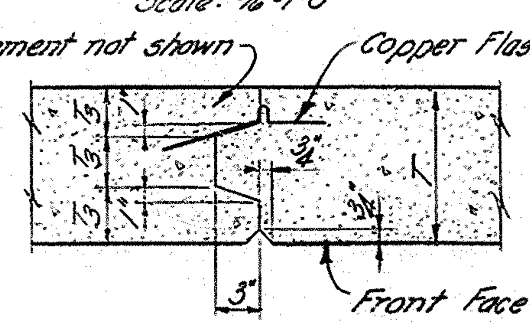
FOOTING PLAN
Scale: 3/16"=1'-0"



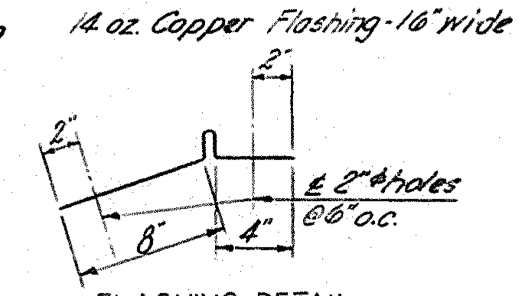
ELEVATION - WING C
WING A - SAME EXCEPT AS NOTED
Scale: 3/16"=1'-0"

ELEVATION - WING D
WING B - SAME EXCEPT AS NOTED
Scale: 3/16"=1'-0"

Note: Keys of vertical construction joints shall be carried up to a line one foot below top of wall. From that line butt joints shall extend to the top of wall.



CONSTRUCTION JOINT DETAIL
No Scale



FLASHING DETAIL

NOTES

1. For General Notes & Stake-Out Plan see Sheet 1.
2. Maximum design pile load = 32½ Tons per pile
3. Maximum design foundation pressure = 1½ Tons per sq. ft.
4. For railing post spacing see sheet 5.
5. Legend: E.F. denotes Each Face W.P. denotes Work Point
   R.F. denotes Rear Face A*1 denotes Abutment 1
   F.F. denotes Front Face A*2 denotes Abutment 2
6. ⊤ Indicates Battered pile and direction of 3 in 12 batter. Ⓣ Indicates Test Pile
7. Fill to be compacted to bottom of footing before driving of piles.

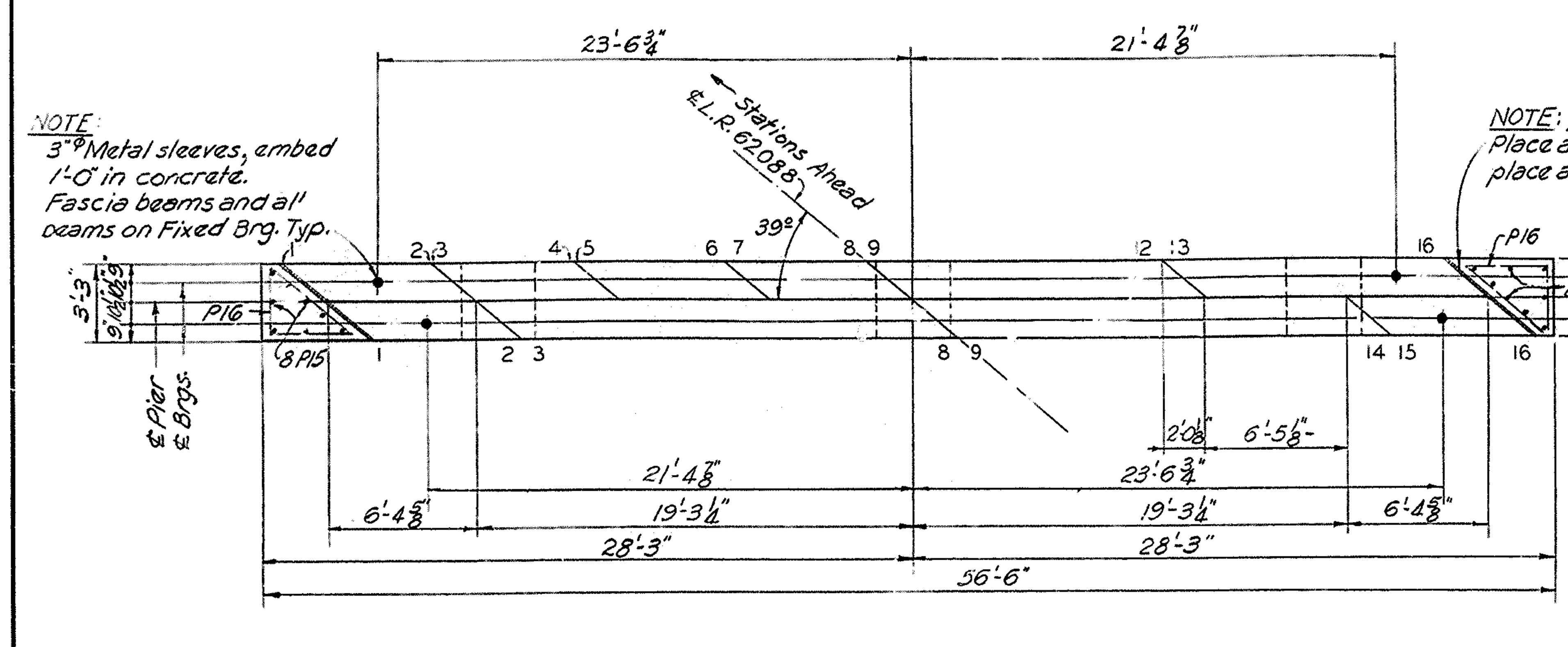
JUN 13 1960
Approved: K.H. Jensen
Bridge Engineer

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

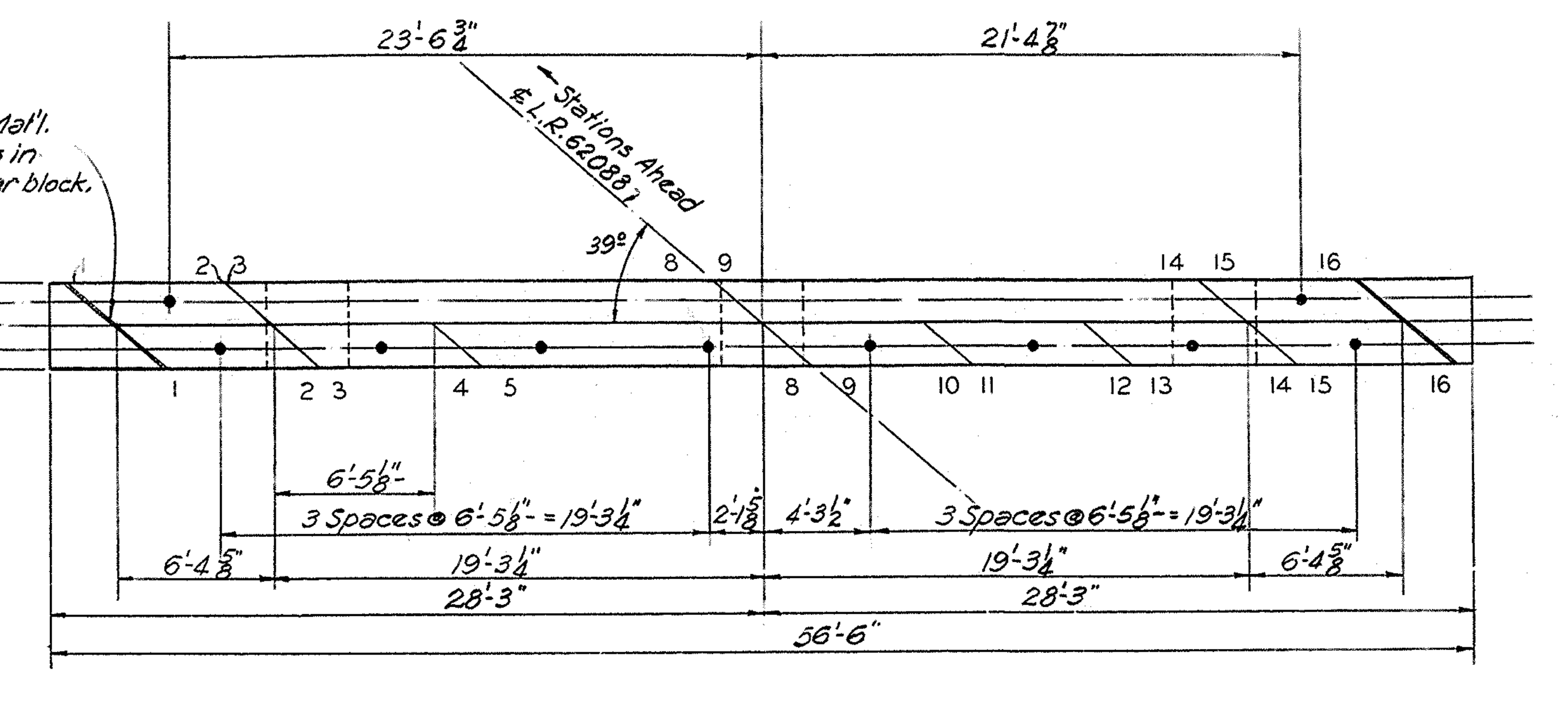
WASHINGTON COUNTY
L.R. 798 SECTION 1
L.R. 798 UNDER L.R. 62088 (RELOC)
ABUTMENTS
STA. 1205 + 50.00

SCALE: AS NOTED
SHEET 2 OF 8

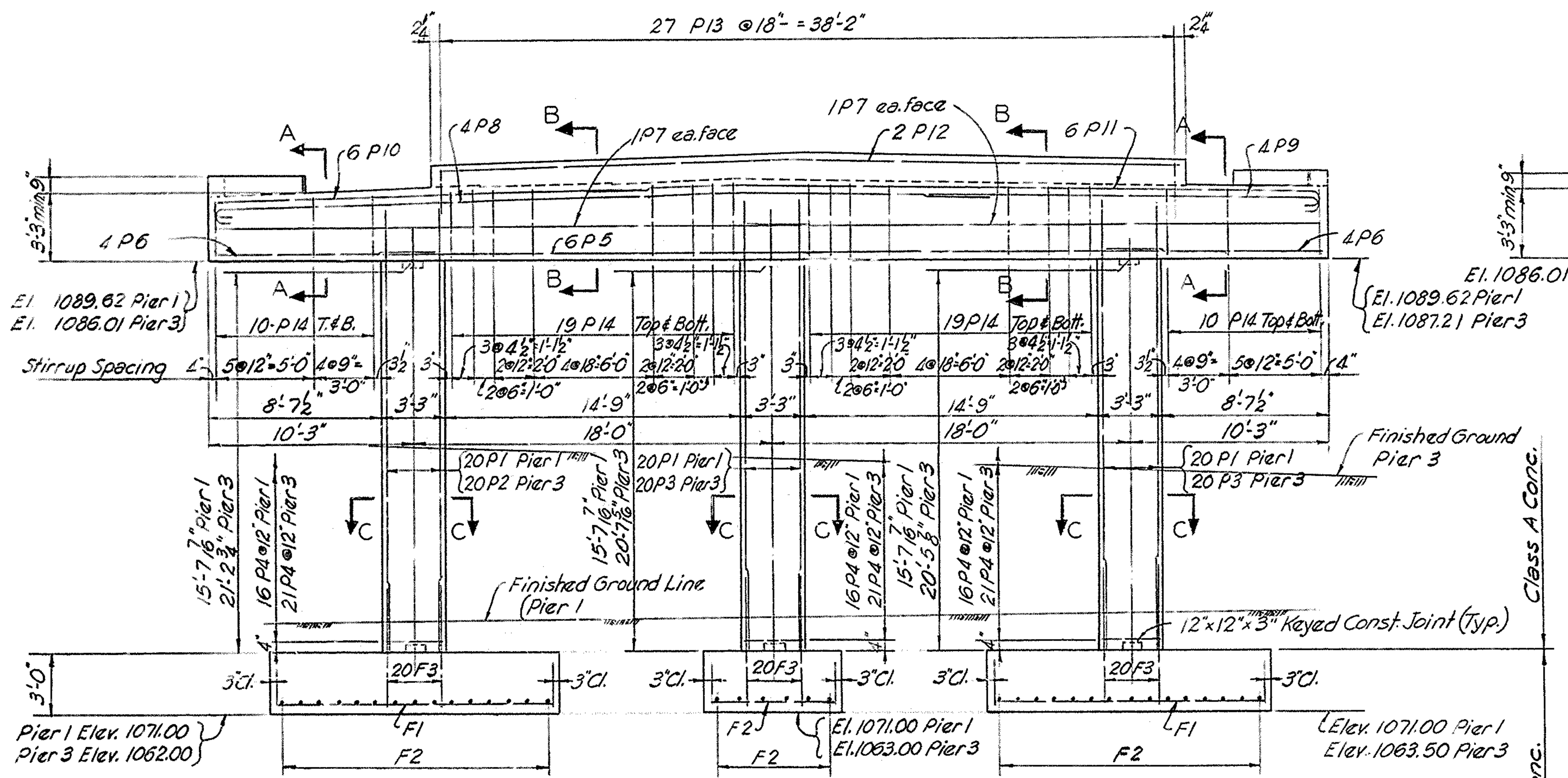
S-3661 A



PLAN - PIER 1
Scale 3/16" = 1'-0"



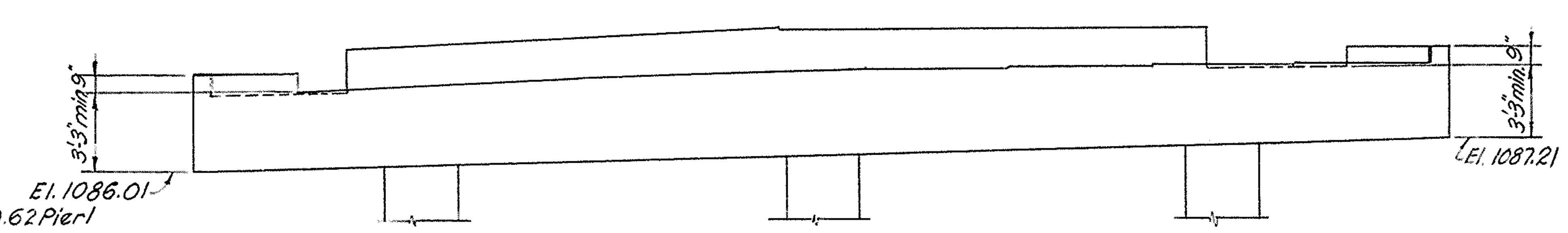
PLAN - PIER 3
Scale 3/16" = 1'-0"



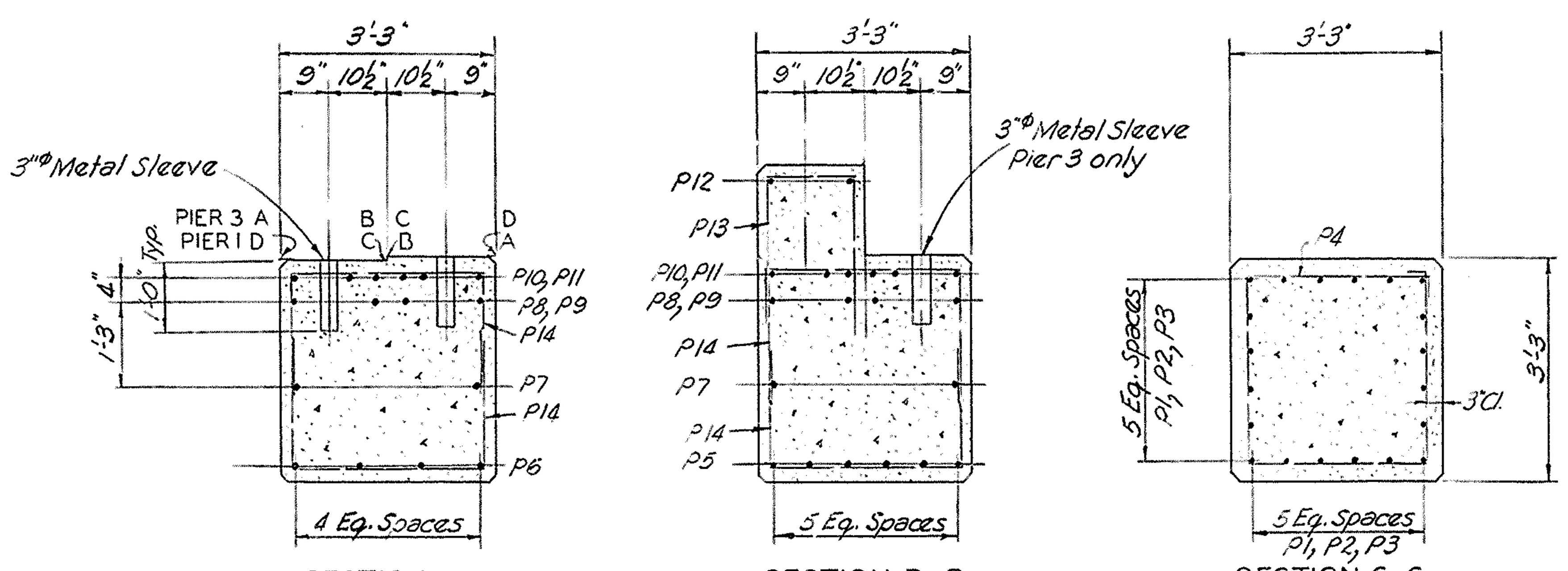
ELEVATION - PIER 1 SHOWN
Scale 3/16" = 1'-0"
Pier 3 Similar as noted

NOTE: F3 dowels in footing to match P1, P2, P3 bars in columns.

NOTE: For Reinf. Bars in Pier Cap, see Pier 1.



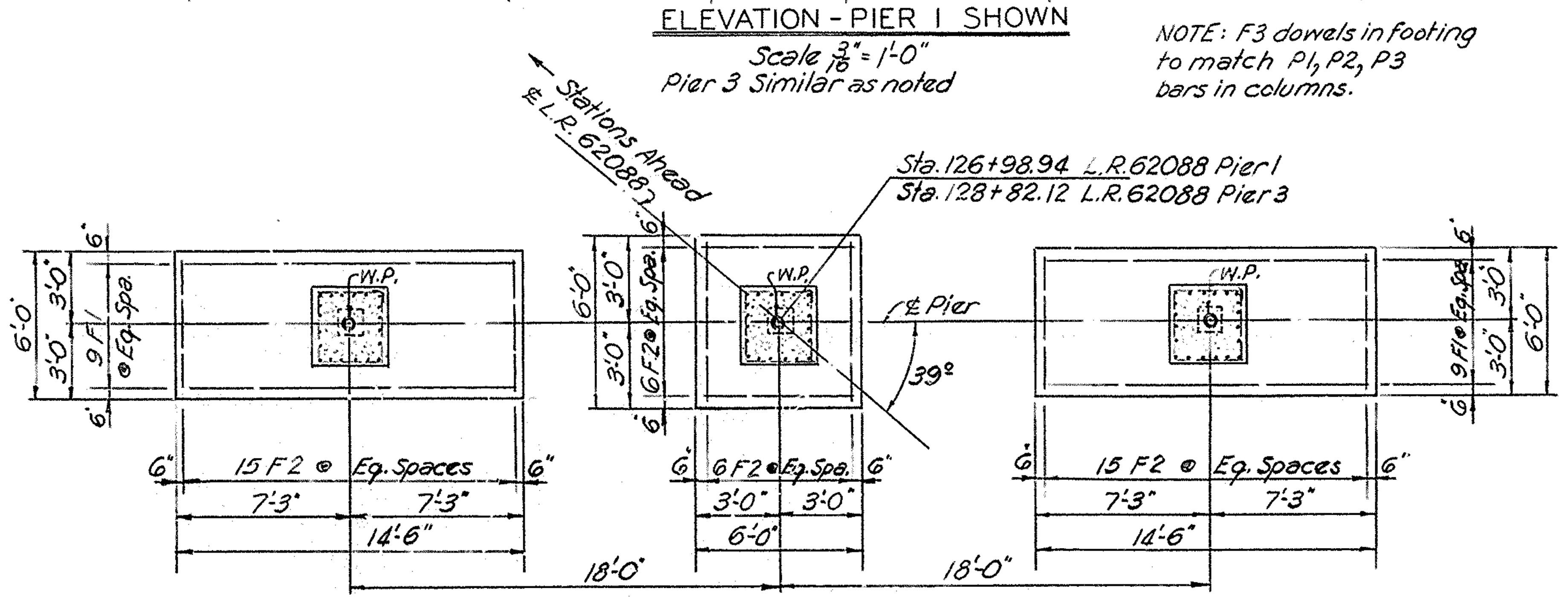
ELEVATION PIER 3
Scale 3/16" = 1'-0"
See Elevation View - Pier 1 for columns and footing details.



SECTION A-A
Scale ½" = 1'-0"

SECTION B-B
Scale ½" = 1'-0"

SECTION C-C
Scale ½" = 1'-0"



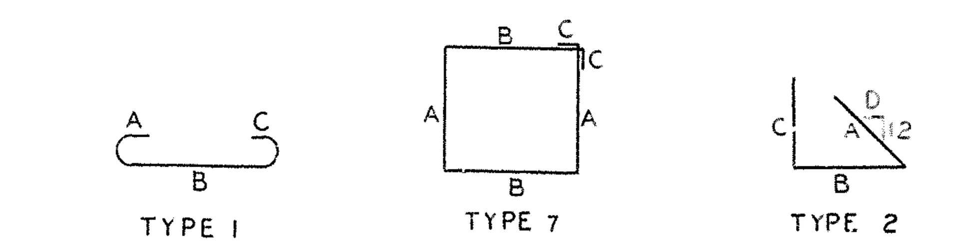
FOOTING PLAN - PIERS 1 & 3
Scale 3/16" = 1'-0"

TABLE OF BEARING ELEVATIONS																	
	BRG.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
PIER 1	A	1093.01	1093.01	1093.06	1093.31	1093.26	1093.43	1093.38	1093.55	1093.55	—	—	1093.44	1093.44	—	—	1093.17
	B	1093.04	1093.04	1093.09	1093.34	1093.29	1093.46	1093.41	1093.58	1093.58	—	—	1093.47	1093.47	—	—	1093.20
	C	1093.06	1093.06	1094.38	—	—	—	—	1094.85	1094.85	—	—	—	—	1094.47	1093.22	1093.09
	D	1093.07	1093.07	1094.39	—	—	—	—	1094.86	1094.86	—	—	—	—	1094.48	1093.23	1093.10
PIER 3	A	1089.41	1089.41	1091.29	—	—	—	—	1092.16	1092.06	—	—	—	—	1092.17	1090.42	1090.45
	B	1089.48	1089.48	1091.36	—	—	—	—	1092.23	1092.13	—	—	—	—	1092.24	1090.49	1090.52
	C	1089.56	1089.56	1089.70	1090.00	1090.00	—	—	1090.46	1090.46	1090.45	1090.51	1090.51	1090.56	1090.48	1090.54	1090.46
	D	1089.62	1089.62	1089.76	1090.06	1090.06	—	—	1090.52	1090.52	1090.51	1090.57	1090.58	1090.62	1090.54	1090.60	1090.52

N.H.U

BAR SCHEDULE

MK	NUMBER PIER 1	NUMBER PIER 3	SIZE	LENGTH	TYPE	A	B	C	D	REMARKS
F1	18	18	10	14'-0"	Str.					
F2	42	42	5	5'-6"	Str.					
F3	60	60	10	7'-0"	Str.					
P1	60		10	18'-0"	Str.					
P2		20	10	23'-9"	Str.					
P3		40	10	23'-0"	Str.					
P4	48	63	4	11'-6"	7	2'-9"	2'-9"	3		
P5	6	6	10	39'-3"	Str.					
P6	8	8	5	10'-6"	Str.					
P7	4	4	5	29'-2"	Str.					
P8	4	4	10	40'-0"	1	1'-5"	38'-7"	—		
P9	4	4	10	23'-3"	1	1'-5"	21'-10"	—		
P10	6	6	11	24'-1"	1	1'-7"	22'-6"	—		
P11	6	6	11	40'-0"	1	1'-7"	38'-5"	—		
P12	2	2	5	38'-2"	Str.					
P13	27	27	5	7'-3"	2	3'-0"	1'-3"	3'-0"	0	
P14	116	116	5	7'-9"	2	2'-5"	2'-11"	2'-5"	0	
P15	16	16	6	2'-0"	Str.					
P16	2	2	4	10'-9"	2	4'-3"	3'-9"	2'-9"	15	

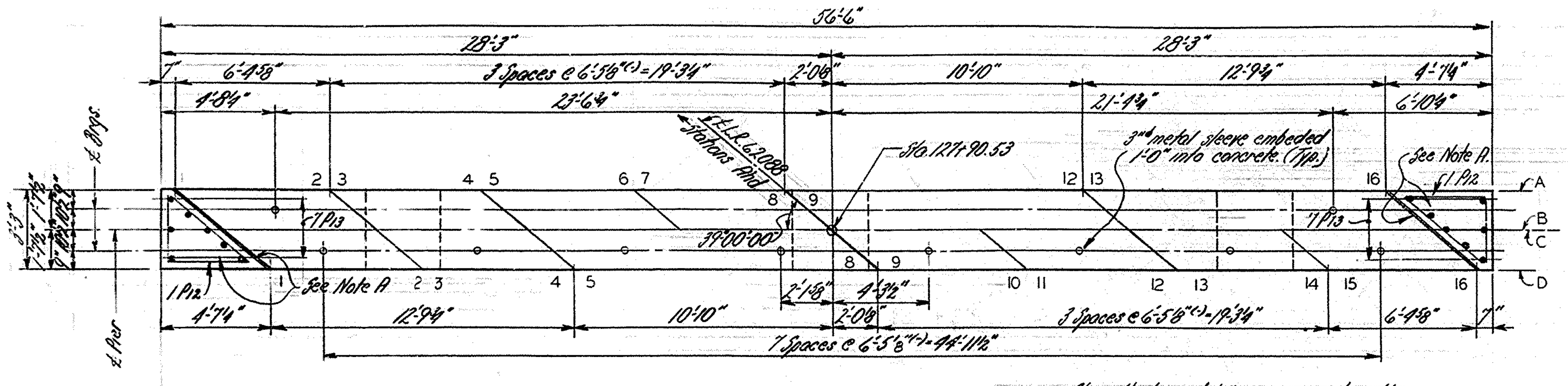


NOTES
1. For additional notes, see sheet 1.
2. Maximum design foundation pressure = 9½ Tons/Sq. Ft.
3. Legend: E.F. denotes Each Face
W.P. denotes Work Point.
4. For superstructure drain pipe connections to piers, see sheet 7.
5. 3"Φ Metal Sleeves to be included in the price for Class A Conc.
6. See note 8 sheet 5 for possible adjustment of bearing elevations due to beam camber.

Approved: JUN 13 19__
K.H. Jensen
Bridge Engineer

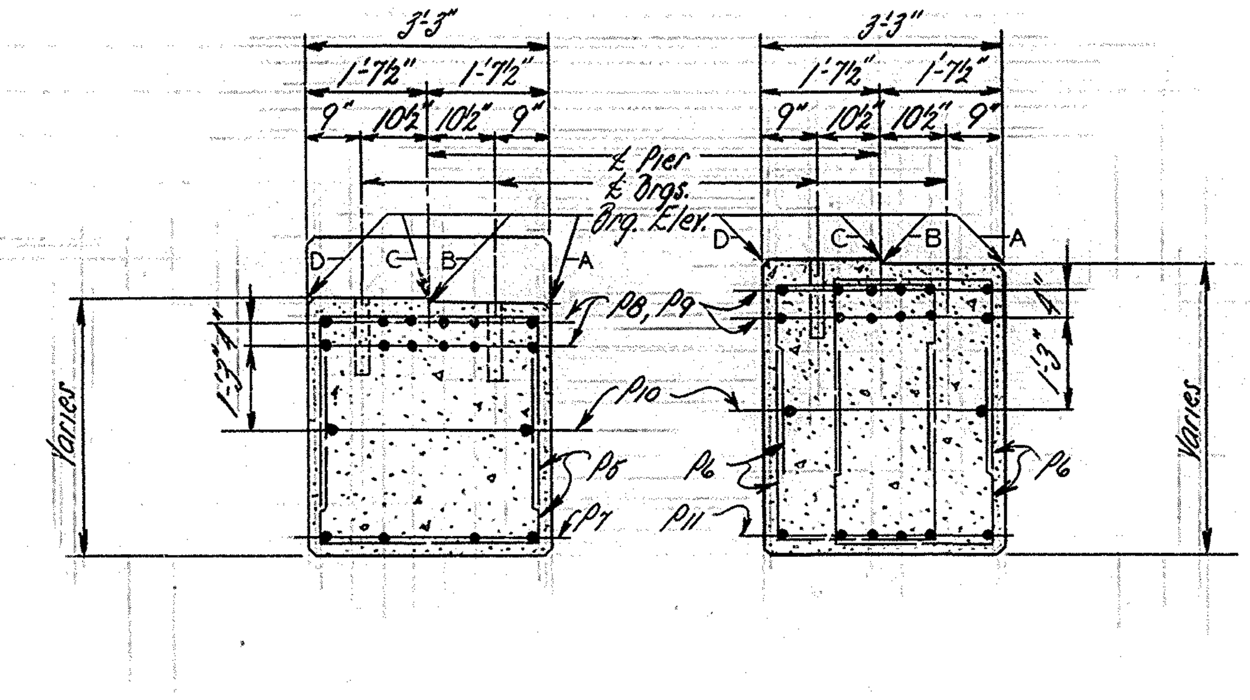
Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

WASHINGTON COUNTY
L.R. 798 SECTION 1
L.R. 798 UNDER L.R. 62088 (RELOC.)
PIERS 1 & 3
STA. 1205+50.00
SCALE: AS NOTED
SHEET 3 OF 8
S-3661 A



PLAN
Scale: 1/4" = 1'-0"

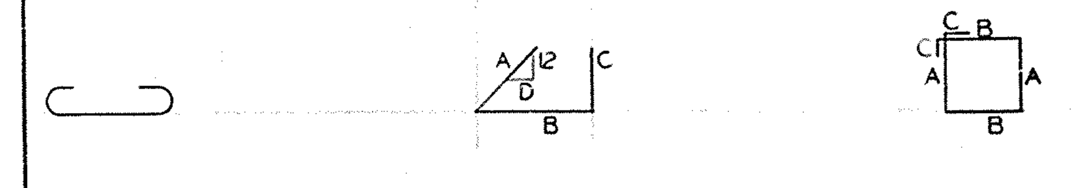
Note A: Shear block and 1/2" prem. exp. jt. mat'l. to be constructed after beams are set in place



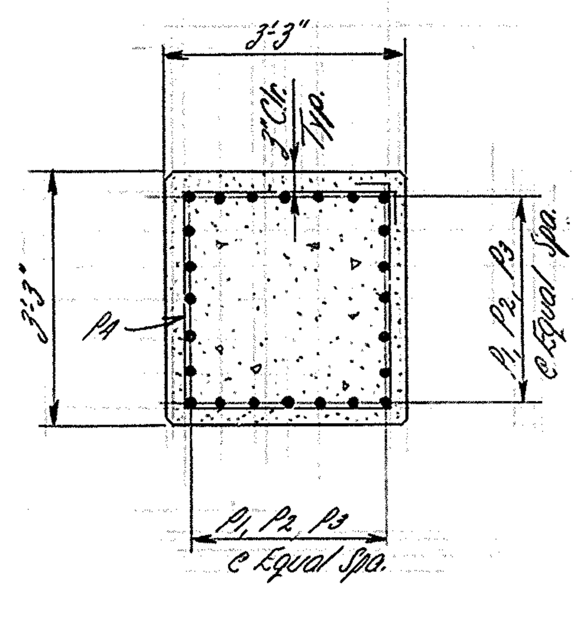
SECTION A-A
Scale: 1/2" = 1'-0"

SECTION B-B
Scale: 1/2" = 1'-0"

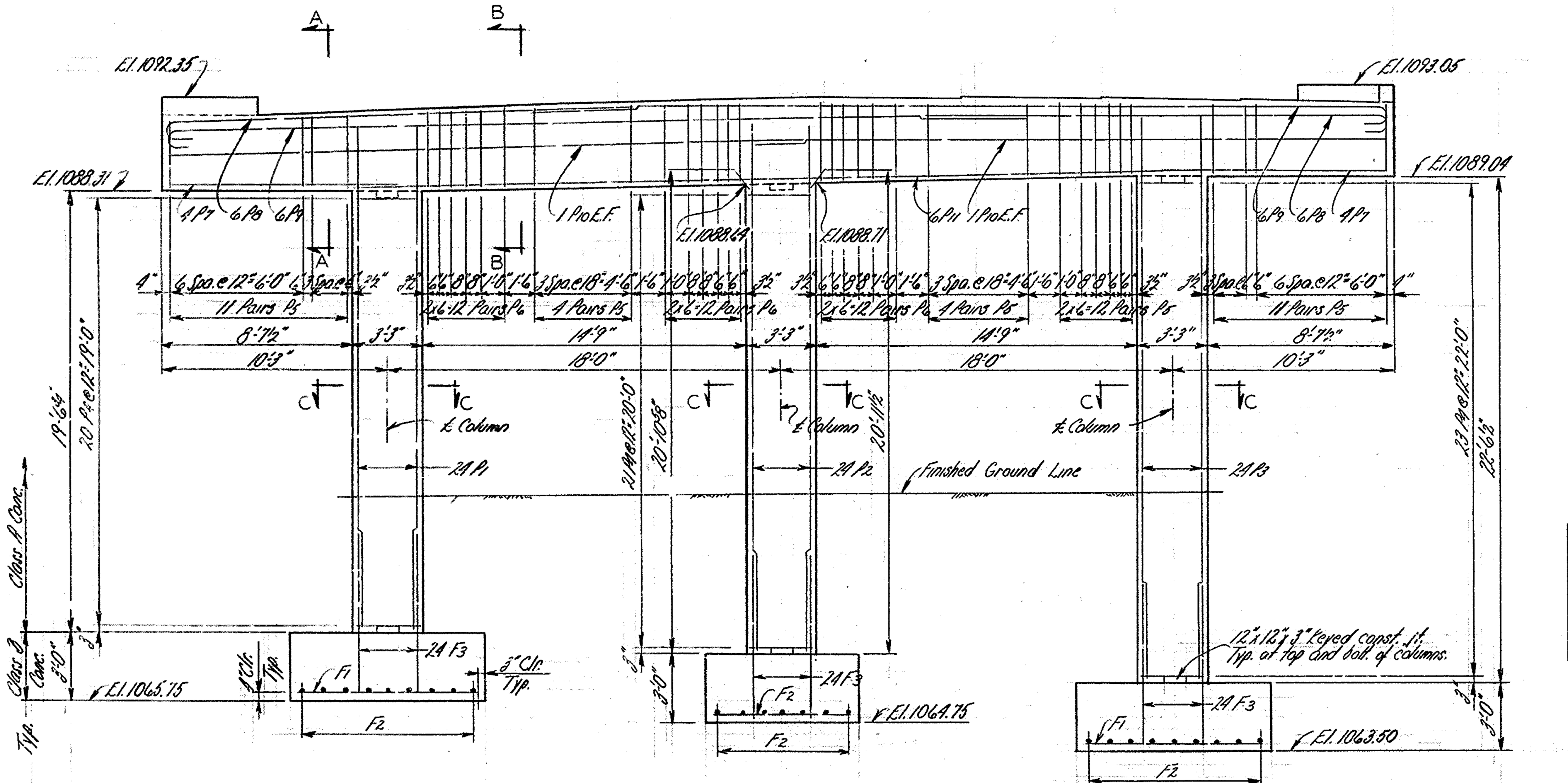
REINF. BAR SCHEDULE									
MK	NUMBER	SIZE	LENGTH	TYPE	A	B	C	D	REMARKS
F1	16	8	8'-6"	Str.					
F2	32	5	5'-6"	Str.					
F3	72	10	7'-0"	Str.					
P1	24	10	22'-3"	Str.					
P2	24	10	23'-6"	Str.					
P3	24	10	25'-0"	Str.					
P4	64	4	11'-6"	7	2'-9"	2'-9"	3"		
P5	60	5	7'-9"	2	2'-5"	2'-11"	2'-5"	0	
P6	96	5	7'-0"	2	2'-5"	2'-2"	2'-5"	0	
P7	8	5	10'-9"	Str.					
P8	12	11	23'-0"	1	1'-7"	21'-5"	0		
P9	12	11	41'-3"	1	1'-7"	39'-8"	0		
P10	4	5	29'-3"	Str.					
P11	6	11	39'-3"	Str.					
P12	2	6	11'-3"	2	4'-6"	3'-10"	2'-11"	15	
P13	14	6	2'-3"	Str.					



TYPE 1 TYPE 2 TYPE 7

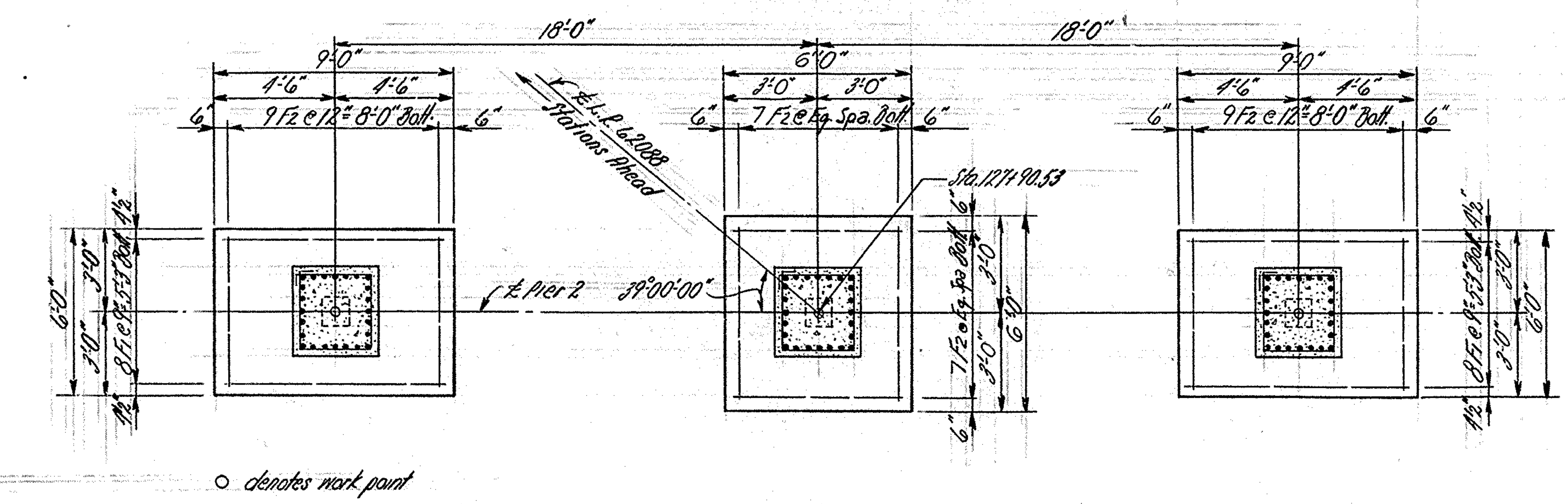


SECTION C-C
Scale: 1/2" = 1'-0"



ELEVATION
Scale: 1/4" = 1'-0"

NOTE: F3 dowels in footing to match P1, P2, P3 bars in column.



○ denotes work point

FOOTING PLAN
Scale: 1/4" = 1'-0"

TABLE OF BEARING ELEVATIONS																	
	BRG.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
PIER 2	A	1091.73	1091.73	1091.80	1092.11	1092.05	1092.28	1092.23	1092.45	1092.45	—	—	1092.45	1092.45	—	—	1092.28
	B	1091.79	1091.79	1091.86	1092.17	1092.11	1092.34	1092.29	1092.51	1092.51	—	—	1092.51	1092.51	—	—	1092.34
	C	1091.80	1091.80	1091.91	1092.17	1092.17	—	—	1092.51	1092.51	1092.46	1092.51	1092.46	1092.51	1092.37	1092.44	1092.30
	D	1091.83	1091.83	1091.94	1092.20	1092.20	—	—	1092.54	1092.54	1092.49	1092.54	1092.49	1092.54	1092.40	1092.47	1092.33

Approved: JUN 13 1960

F. H. Jensen
Bridge Engineer

NOTES

1. For additional notes, see sheet 1.
2. Maximum design foundation pressure = 12 tons per sq. ft.
3. Legend: E.F. denotes Each Face.
4. For superstructure drain pipe connections to pier, see sheet 7.
5. 3"ϕ metal sleeves to be included in the price paid for Class A Conc.
6. See note 8 sheet 5 for possible adjustment of bearing elevations due to beam camber.

Commonwealth of Pennsylvania

Department of Highways
BRIDGE UNIT

WASHINGTON COUNTY

L.R. 798 SECTION 1

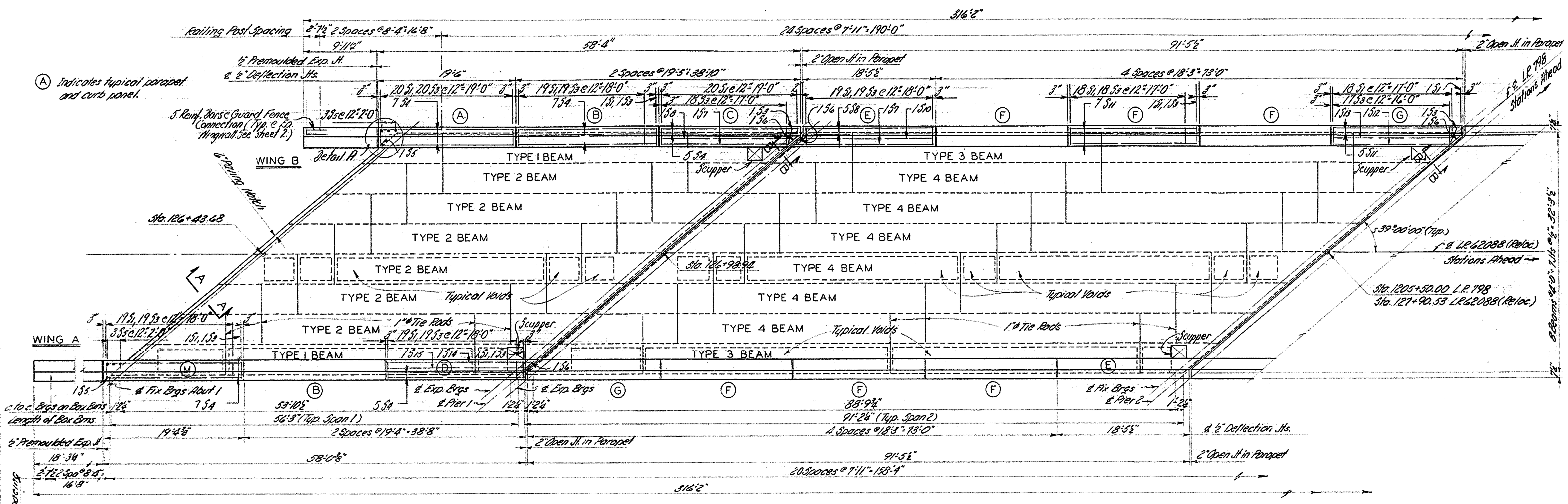
L.R. 798 UNDER L.R. 62088 (RELOC.)

PIER 2

STA. 1205+50.00

SCALE: AS NOTED
SHEET 4 OF 8

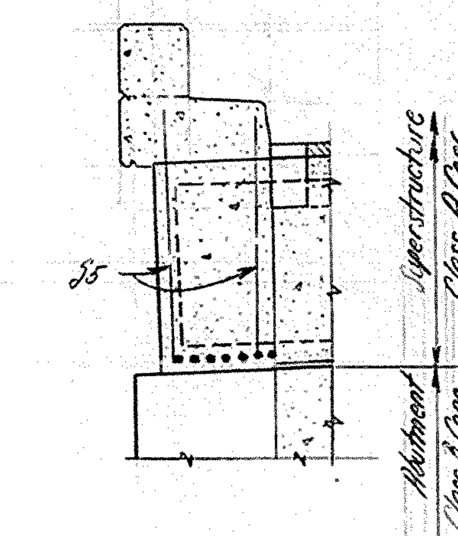
S-3661-A



PLAN - SPANS 1 & 2

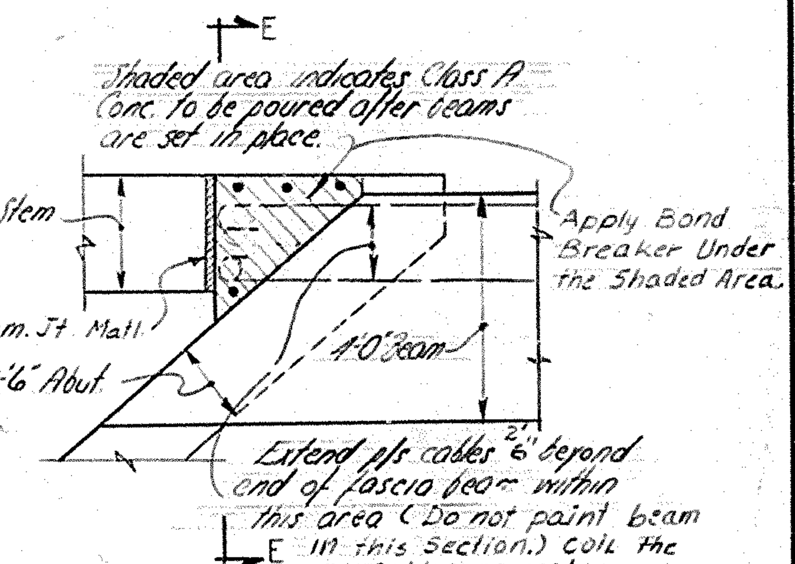
Scale: ⅛"=1'-0"

For additional reinf. see Sects. A-A & B-B, sheet 6.



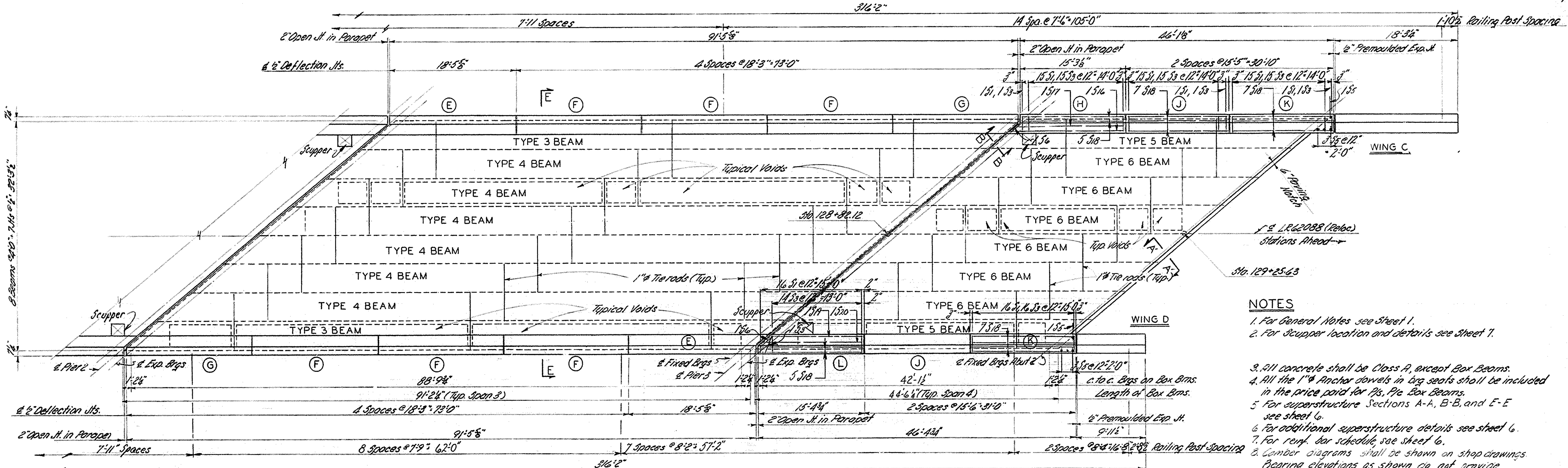
SECTION E-E

Scale: ⅜"=1'-0"



DETAIL A

Scale: ⅜"=1'-0" (Typical Detail at Abut.)



PLAN - SPANS 3 & 4

Scale: ⅛"=1'-0"

Approved: JAN 13 1960

K H Jensen
Bridge Engineer

NOTES

1. For General Notes see Sheet 1.
2. For Scupper location and details see Sheet 7.
3. All concrete shall be Class A, except Box Beams.
4. All the 1"ϕ Anchor dowels in brg seats shall be included in the price paid for P/s, P/c Box Beams.
5. For superstructure Sections A-A, B-B, and E-E see sheet 6.
6. For additional superstructure details see sheet 6.
7. For reinf. bar schedule, see sheet 6.
8. Camber diagrams shall be shown on shop drawings. Bearing elevations as shown do not provide for beam camber. The beam camber shall be checked by the contractor. If the beam camber exceeds the vertical curve ordinate for its span, the bearing elevations or slab thickness shall be adjusted accordingly at no additional cost to P.D.H.

1	Revised Detail 'A' To Clarify.	N.H.U.	W.E.L.	8-15-60
NO	REVISION	BY	CH'KD	DATE

Commonwealth of Pennsylvania

Department of Highways

BRIDGE UNIT

WASHINGTON COUNTY

L.R. 798 — SECTION 1

L.R. 798 UNDER L.R. 62088 (RELOC)

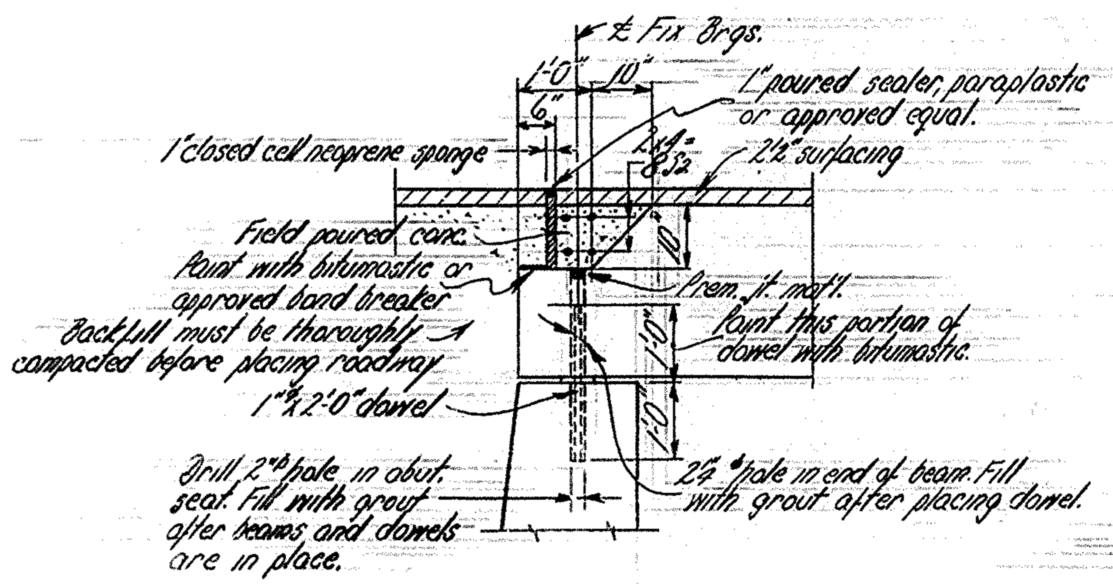
SLAB PLAN

STA. 1205+50.00

SCALE: AS NOTED

SHEET 5 OF 8

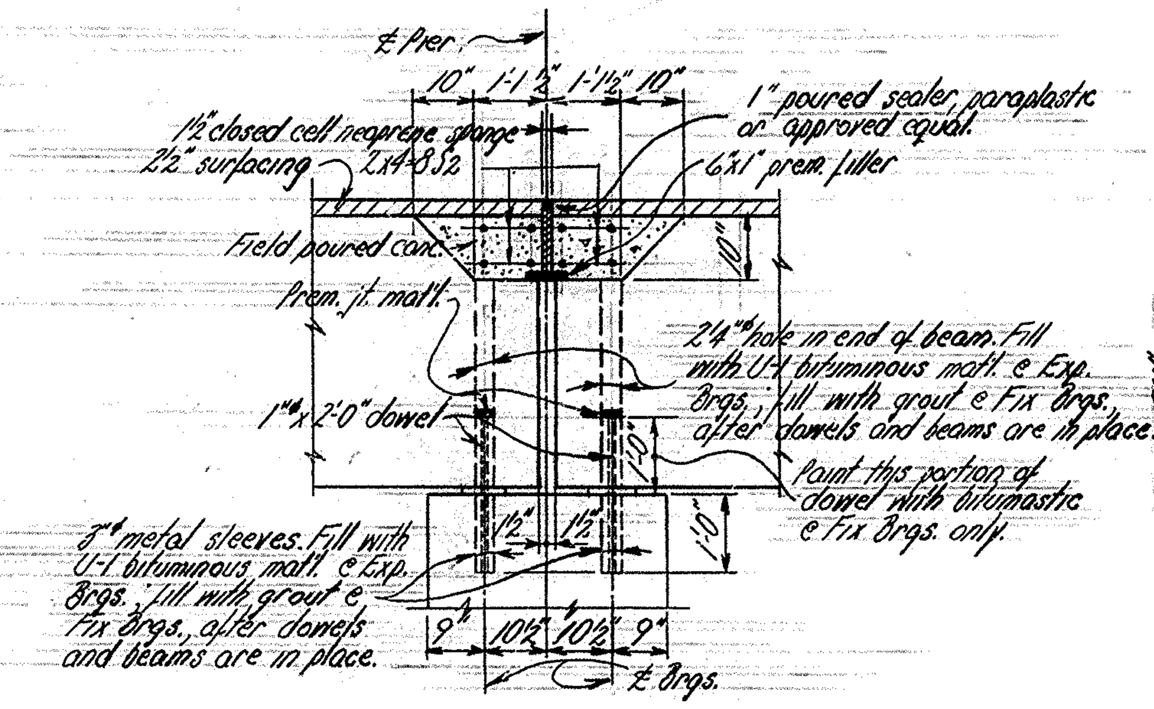
S-3661-A



SECTION A - A

SECTION A'-A' SIMILAR

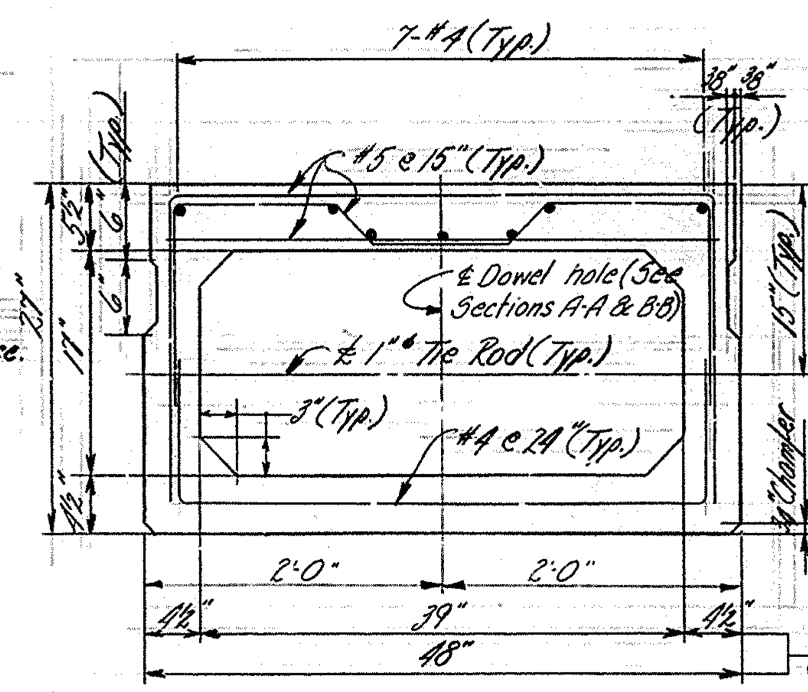
Scale: ½" = 1'-0"



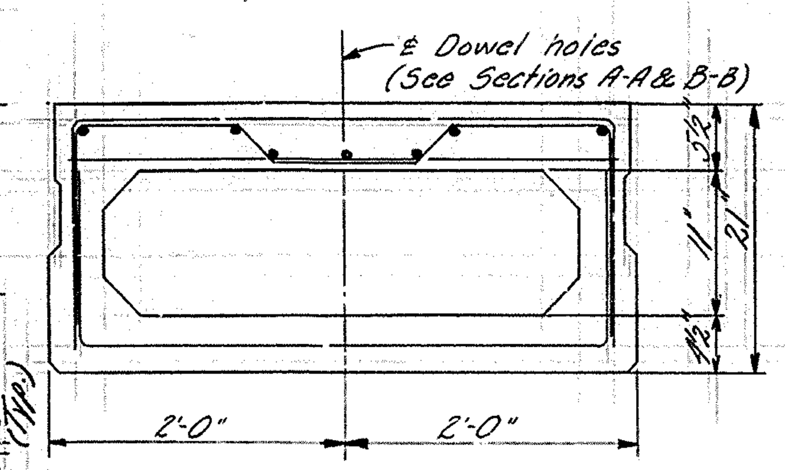
Note: Dowels, dowel holes and sleeves occur in all beams at Fix Brgs. and in fascia beams only at Exp. Brgs.

SECTION B - B

Scale: ½" = 1'-0"

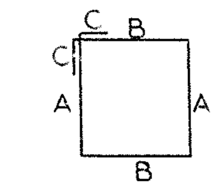


TYPE 2 BEAM

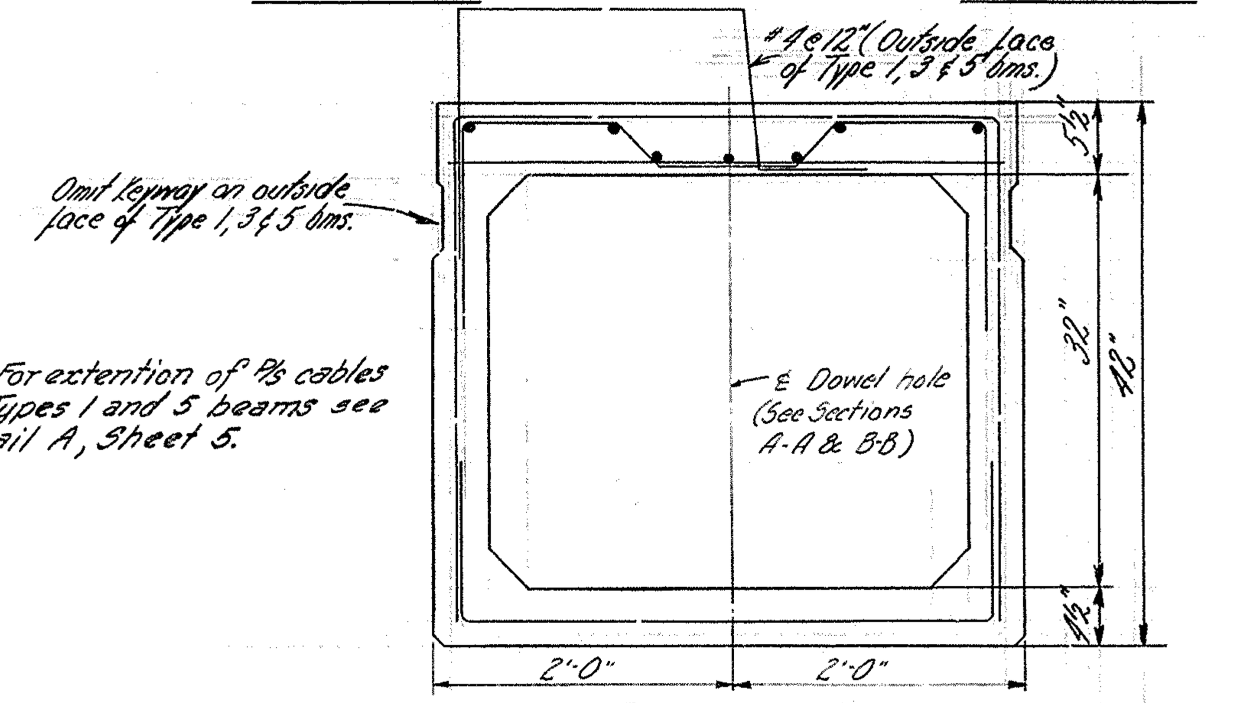


TYPE 6 BEAM

REINF. BAR SCHEDULE									
MK	NUMBER	SIZE	LENGTH	TYPE	A	B	C	D	REMARKS
S1	596	4	6'-3"	7	1'-11½"	11"	3"		
S2	64	6	27'-3"	Str.					
S3	590	4	2'-6"	Str.					
S4	38	4	19'-0"	Str.					
S5	16	4	3'-9"	Str.					
S6	12	4	2'-0"	Str.					
S7	1	4	17'-3"	Str.					
S8	21	4	18'-0"	Str.					
S9	4	4	19'-6"	Str.					
S10	4	4	18'-9"	Str.					
S11	104	4	17'-9"	Str.					
S12	4	4	16'-0"	Str.					
S13	4	4	16'-9"	Str.					
S14	1	4	20'-6"	Str.					
S15	1	4	19'-9"	Str.					
S16	1	4	16'-6"	Str.					
S17	1	4	15'-9"	Str.					
S18	38	4	15'-0"	Str.					
S19	1	4	13'-3"	Str.					
S20	1	4	14'-0"	Str.					



TYPE 7



Note: For extention of P/S cables on Types 1 and 5 beams see Detail A, Sheet 5.

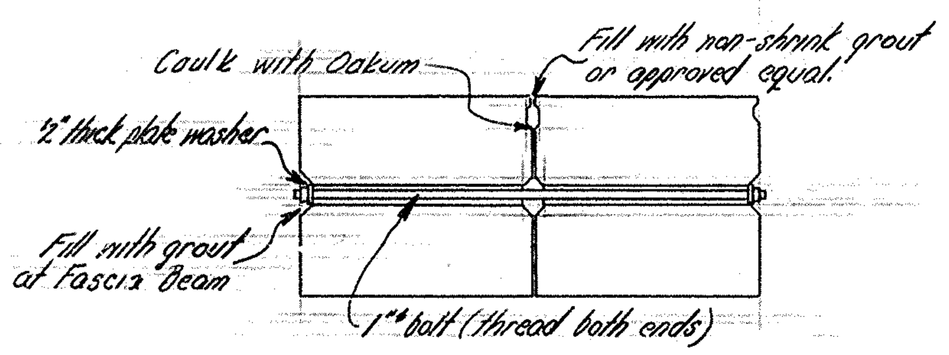
TYPE 1, 3, 4 & 5 BEAM

Number and size of strands, location of c.g. and concrete stress at release of prestress to be shown on shop detail drawings.

TYPICAL SECTIONS - PRESTRESSED MEMBERS

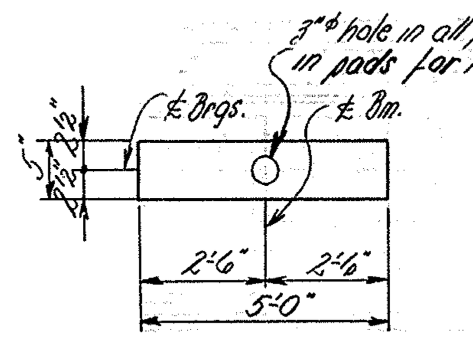
Scale: 1" = 1'-0"

FOR NOTES SEE SHEETS 1 & 5.



INTERBEAM CONNECTION

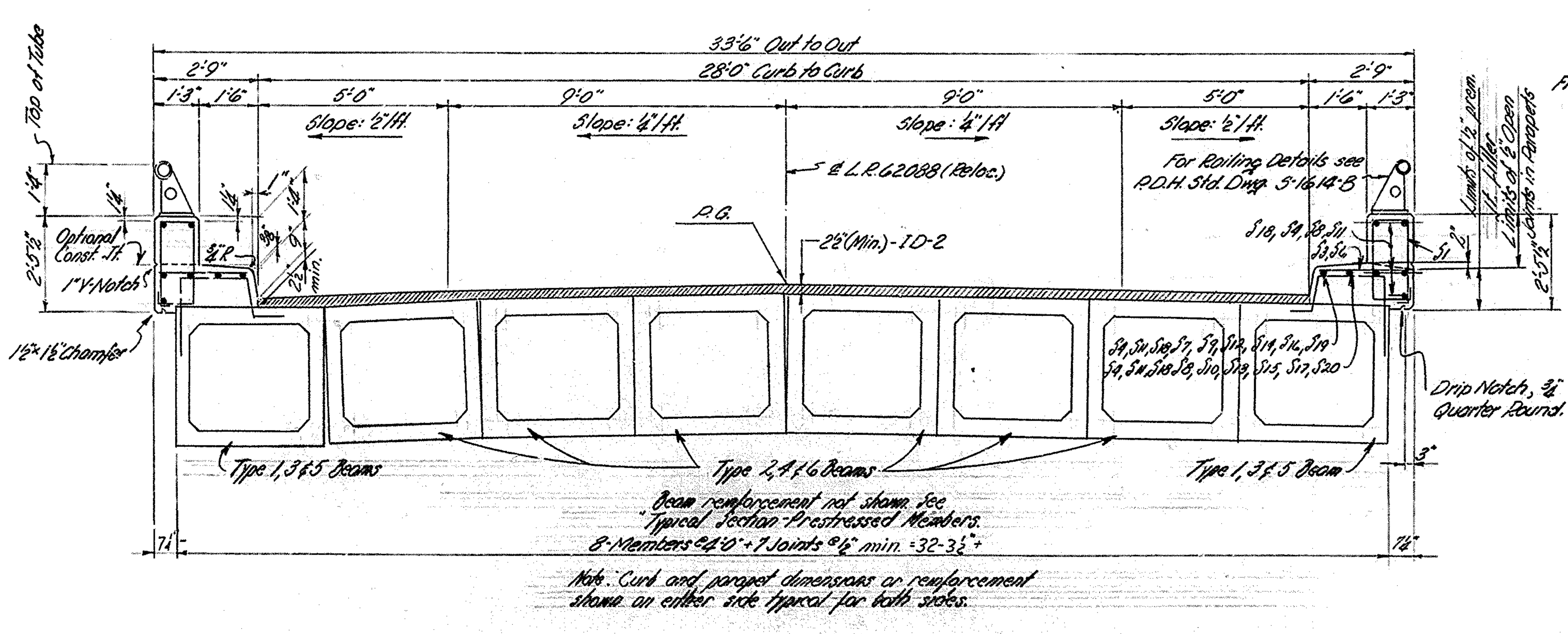
Not to Scale



Neoprene Bearing pads to be durometer hardness = 60
Spans 1 & 4 - 1" thick pads.
Spans 2 & 3 - 1¼" thick pads.
12 pads 5" x 1" x 5'-0" without holes.
20 pads 5" x 1" x 5'-0" with holes.
12 pads 5" x 1¼" x 5'-0" without holes.
20 pads 5" x 1¼" x 5'-0" with holes.

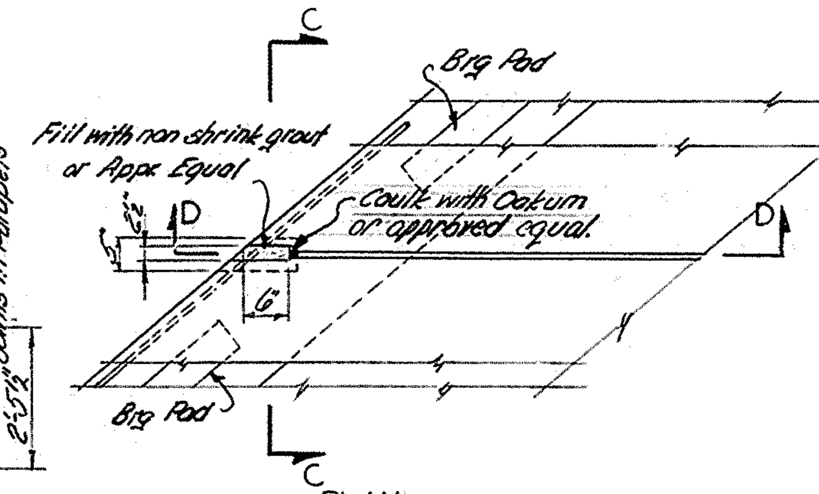
BEARING PADS

Not to Scale

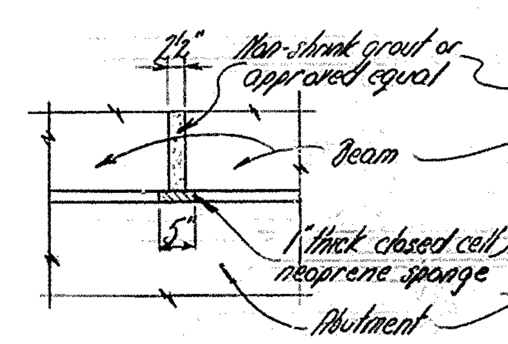


SECTION E-E

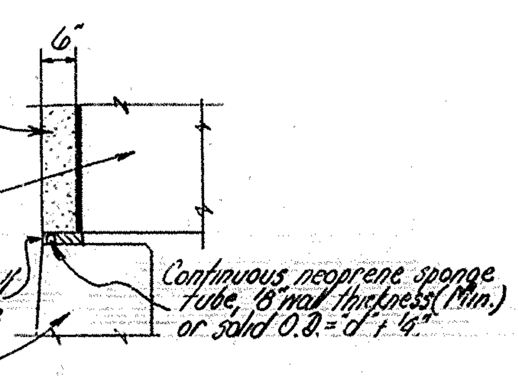
Scale: ⅜" = 1'-0"



PLAN



SECTION C-C



SECTION D-D

WATERPROOFING DETAIL

Scale: ½" = 1'-0"

Approved: JUN 13 1960

K. H. Jensen
Bridge Engineer

Commonwealth of Pennsylvania

Department of Highways

BRIDGE UNIT

WASHINGTON COUNTY

L.R. 798 SECTION 1

L.R. 798 UNDER L.R. 62088 (RELOC)

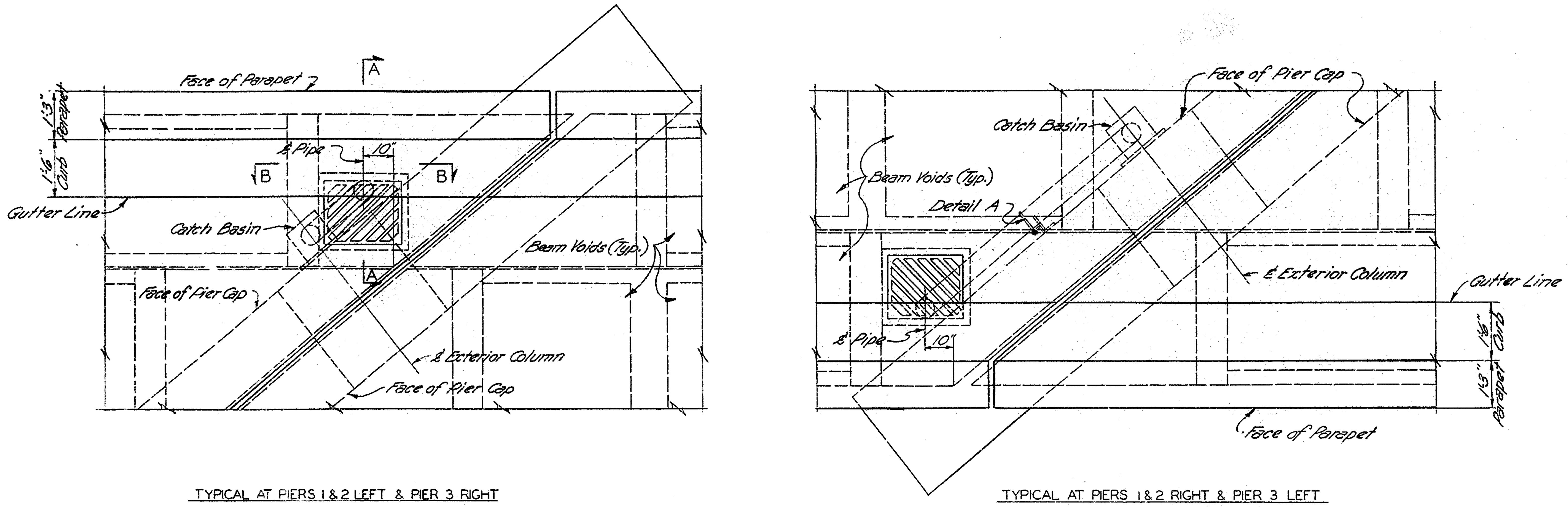
SUPERSTRUCTURE DETAILS

STA. 1205 + 50.00

SCALE: AS NOTED

SHEET 6 OF 8

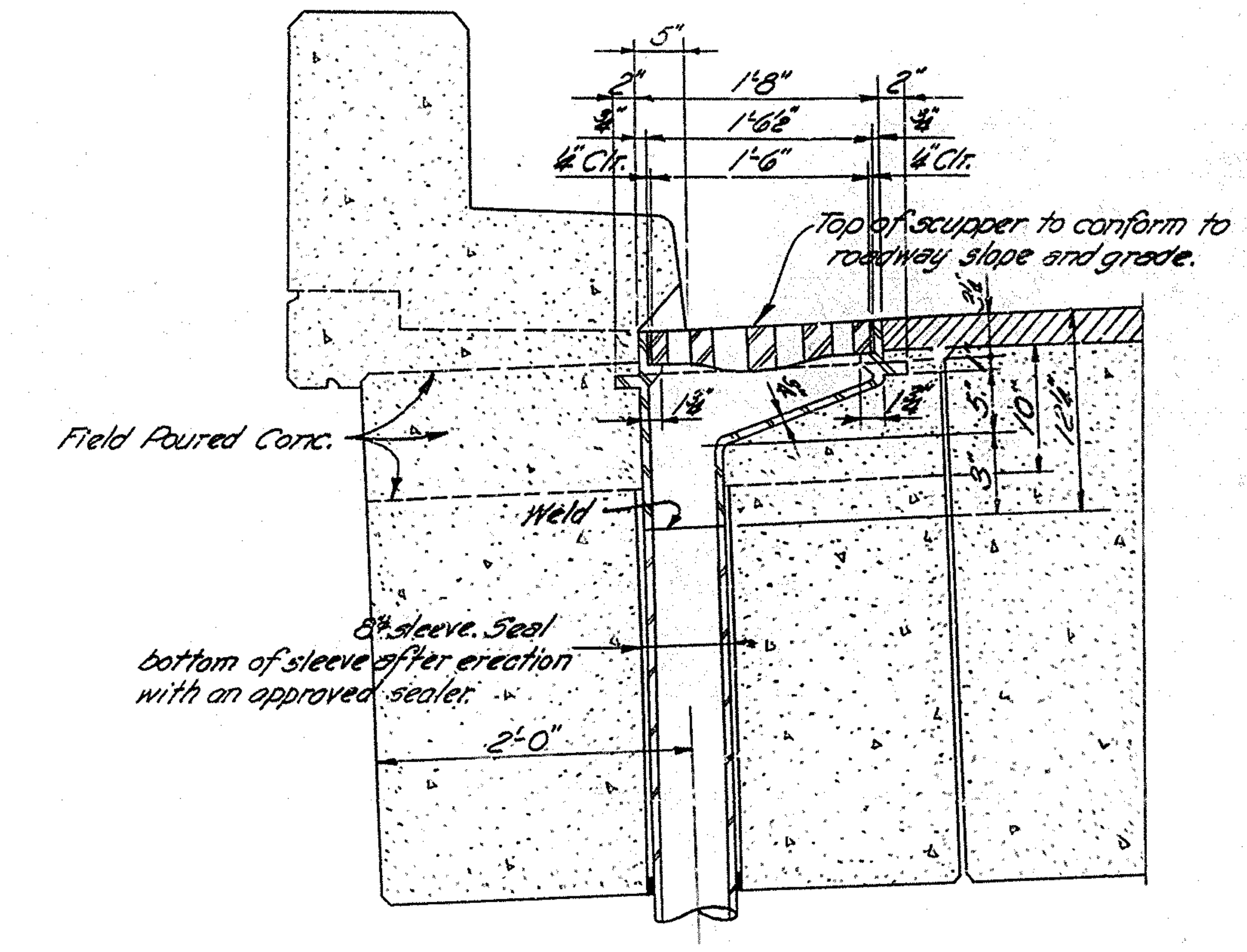
S-3661-A



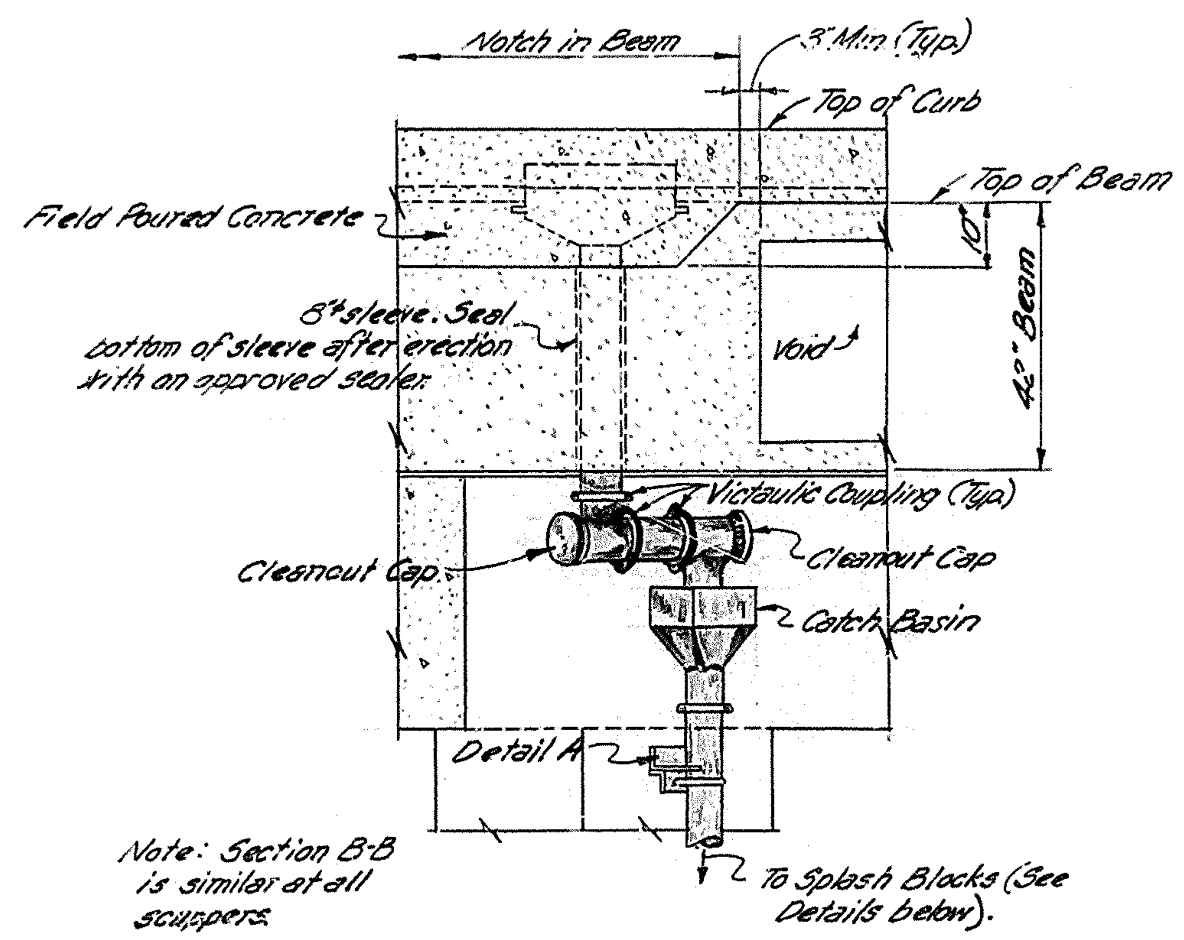
TYPICAL AT PIERS 1 & 2 LEFT & PIER 3 RIGHT

TYPICAL AT PIERS 1 & 2 RIGHT & PIER 3 LEFT

SCUPPER PLAN AT PIERS
Scale: ½"=1'-0"

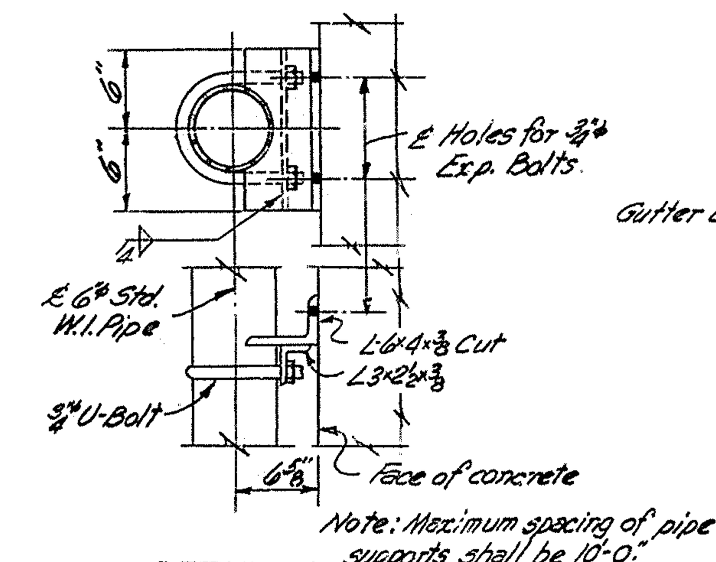


SECTION A-A
Scale: 1"=1'-0"



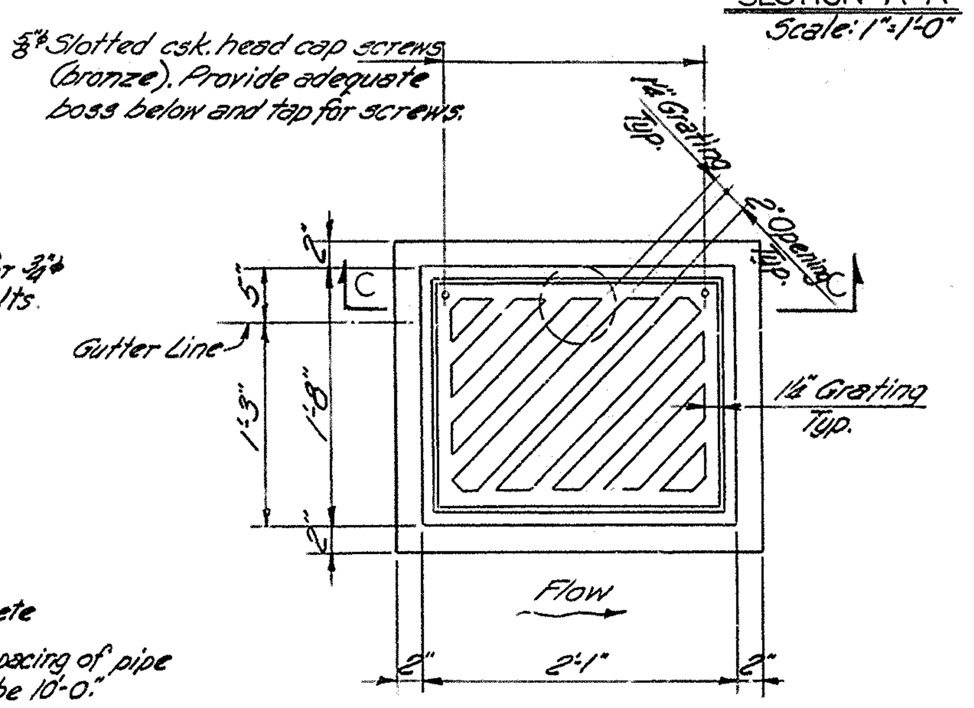
Note: Section B-B is similar at all scuppers.

SECTION B-B
Scale: ½"=1'-0"

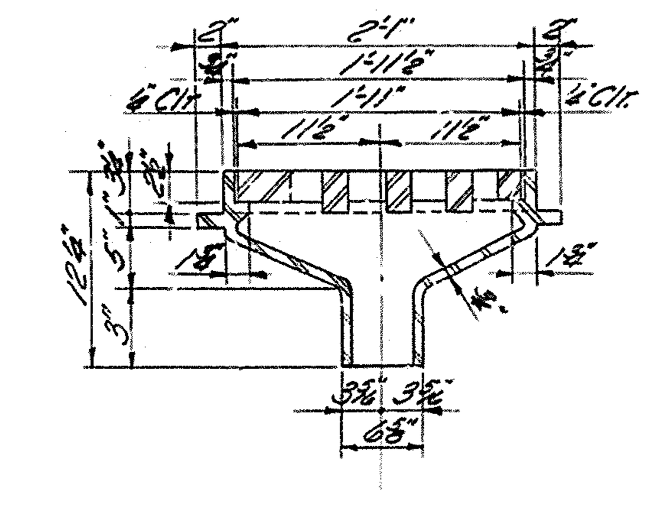


Note: Maximum spacing of pipe supports shall be 10'-0".

DETAIL A
Scale: 1"=1'-0"



PLAN OF SCUPPER
Scale: 1"=1'-0"

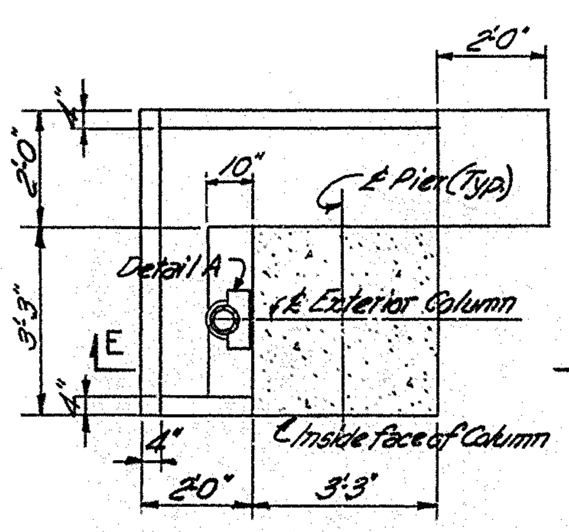


SECTION C-C
Scale: 1"=1'-0"

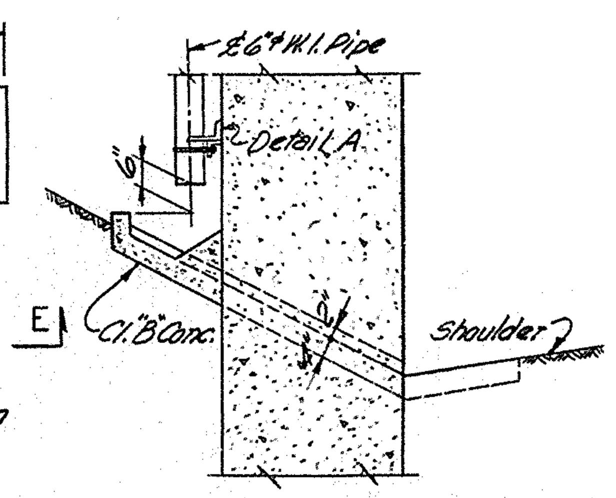
NOTES

1. All pipe is to be Std. 6"ϕ wrought iron.
2. Scuppers and catch basin material to be malleable iron or cast steel.
3. All couplings to be Victaulic or approved equal.
4. Weight of scuppers, catch basin, pipe and pipe supports is included in Fabricated Structural Steel Item.
5. For additional notes, see sheet 1.

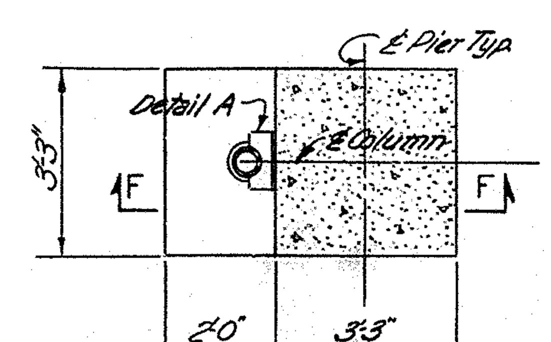
Approved: JUN 13 1980
K.H. Jensen
Bridge Engineer



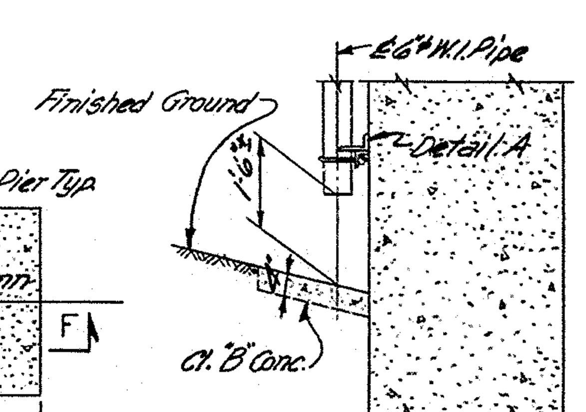
SPLASH BLOCK PLAN
PIERS 1 & 3 - LT. & RT.
Scale: ⅜"=1'-0"



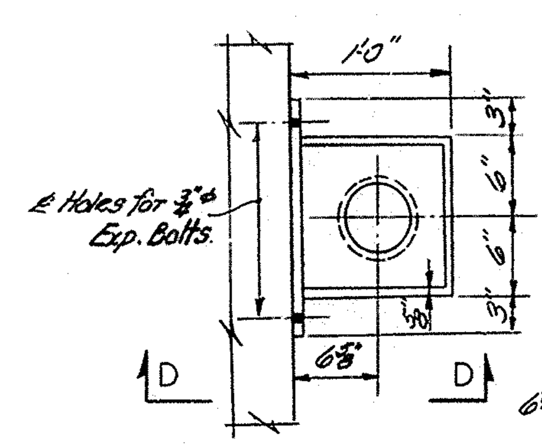
SECTION E-E
Scale: ⅜"=1'-0"



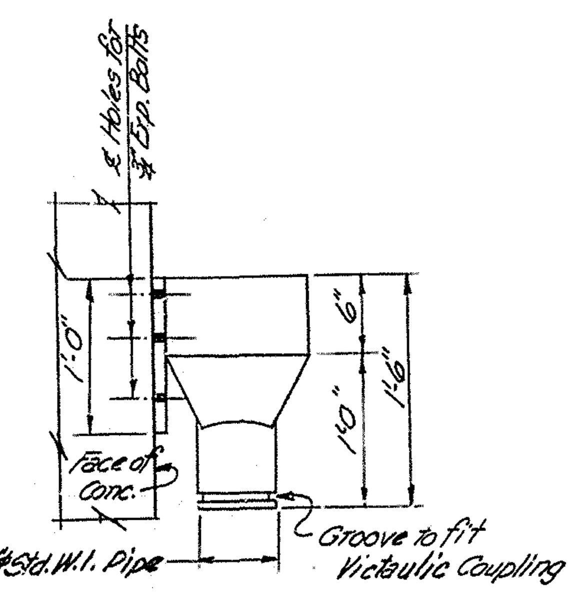
SPLASH BLOCK PLAN
PIER 2 LT. & RT.
Scale: ⅜"=1'-0"



SECTION F-F
Scale: ⅜"=1'-0"



CATCH BASIN
Scale: 1"=1'-0"



VIEW D-D
Scale: 1"=1'-0"

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT

WASHINGTON COUNTY

L.R. 798 SECTION 1

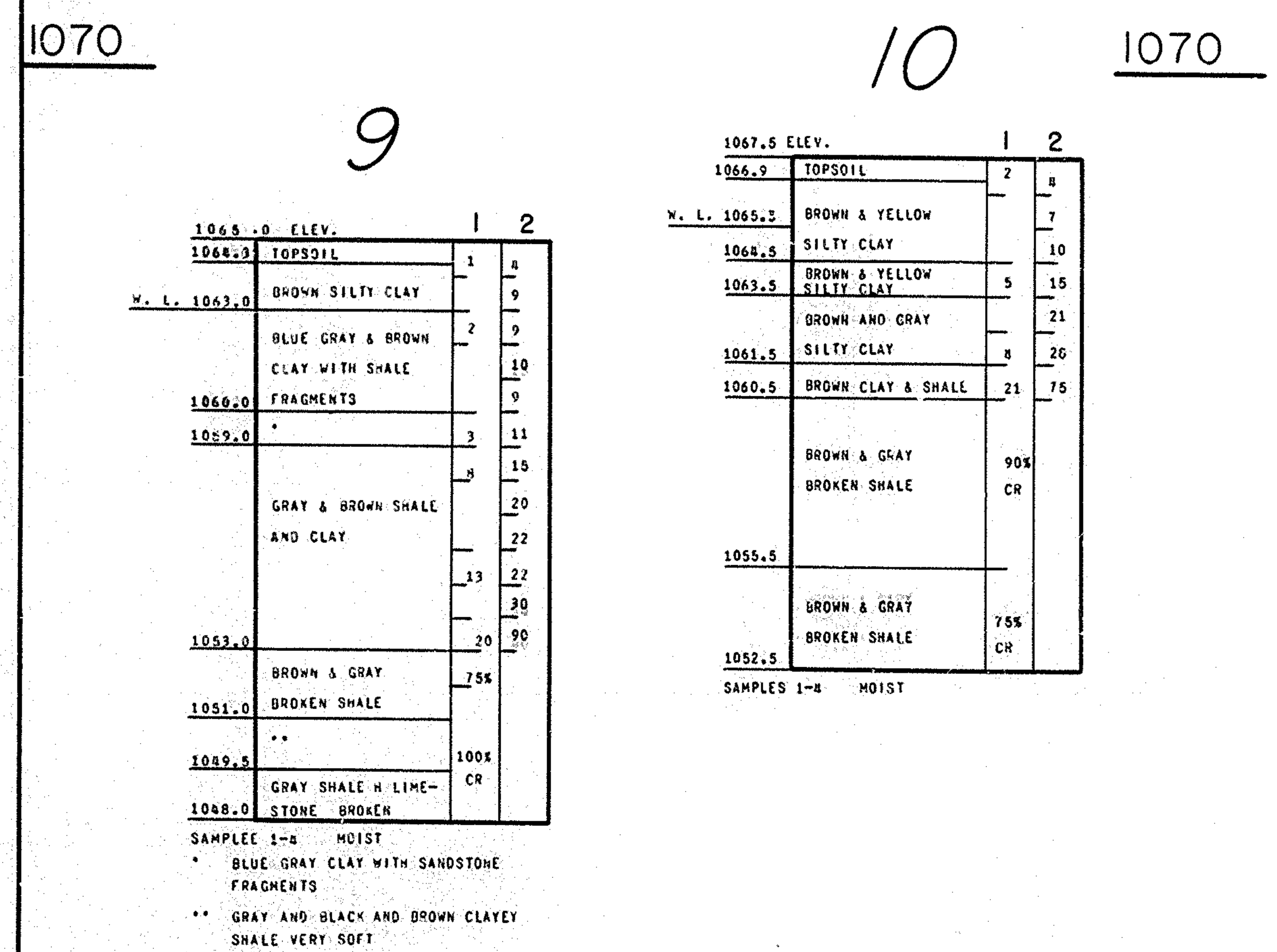
L.R. 798 UNDER L.R. 62088 (RELOC)

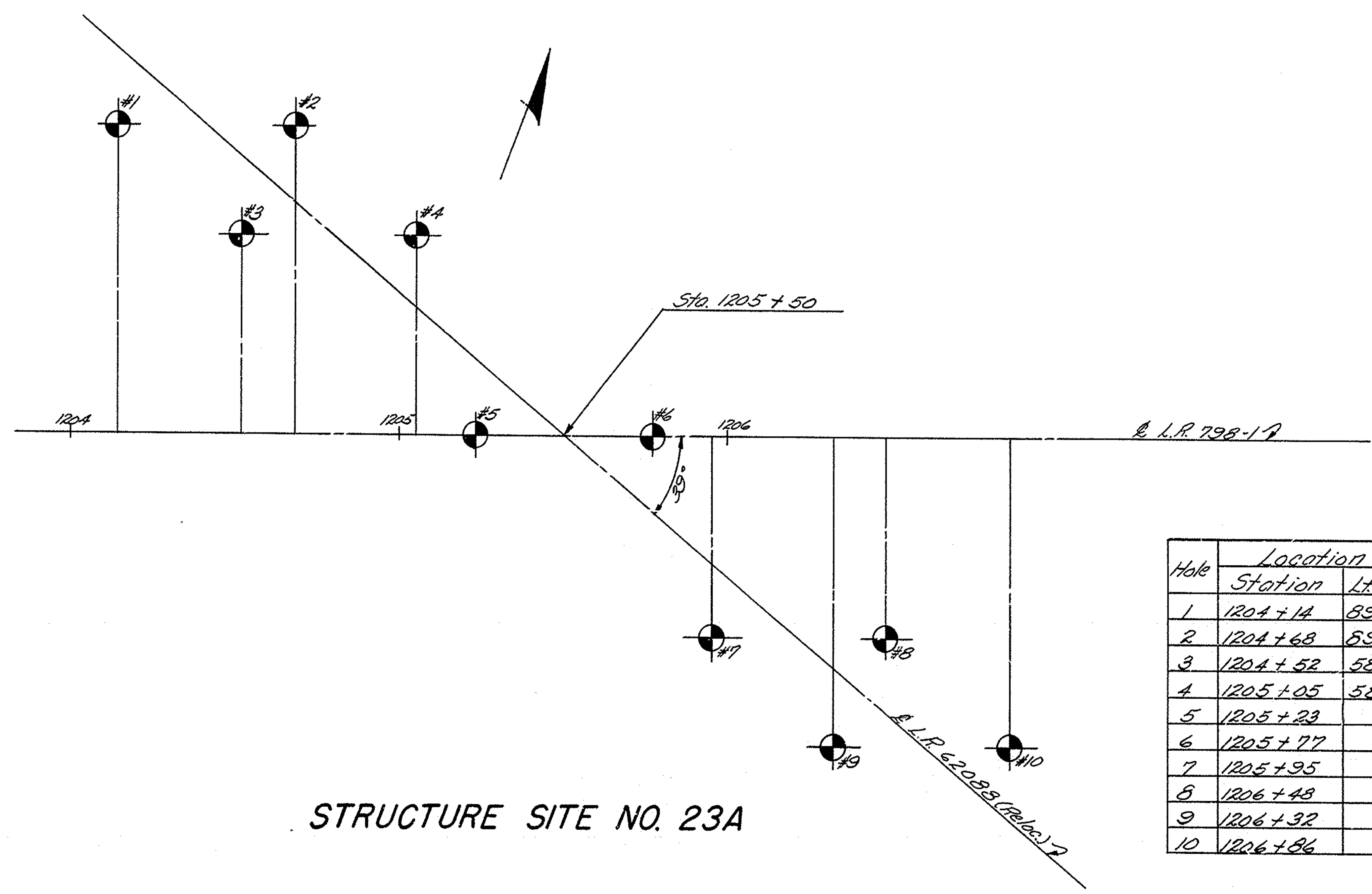
DRAINAGE DETAILS

STA. 1205+50.00

SCALE: AS NOTED
SHEET 7 OF 8

S-3661A





STRUCTURE SITE NO. 23A

Hole	Location Station	Lt.	Rt.
1	1204 + 14	89'	
2	1204 + 68	89'	
3	1204 + 52	58'	
4	1205 + 05	58'	
5	1205 + 23	℄	
6	1205 + 77	℄	
7	1205 + 95		58'
8	1206 + 48		58'
9	1206 + 32		89'
10	1206 + 86		89'

L E G E N D

DISTANCE HAMMER DROP	24 INCH
DRIVE HAMMER	300 LBS.
SAMPLER HAMMER	300 LBS.
CASING SIZE	4 INCH
SAMPLER SIZE	2 INCH
SIZE OF CORE BIT	NX INCH

COLUMN 1 DENOTES SAMPLE BLOWS AND PERCENTAGE OF CORE RECOVERY

COLUMN 2 DENOTES CASING BLOWS

VERTICAL SCALE	1" - 4'0"
W. L.	WATER LEVEL
CR	CORE RECOVERY

CLASSIFICATION OF SOIL HAS BEEN MADE BY THE DRILLER AND HAS NOT BEEN CHECKED BY A SOILS ENGINEER. CLASSIFICATION OF ROCK HAS BEEN MADE BY THE DRILLER AND HAS NOT BEEN CHECKED BY A GEOLOGIST.

S-3661-A

STA. 1205 + 50
SHEET 8 OF 8

TEST BORINGS
FOR
GROF & MYERS
HARRISBURG, PA.
OF
L.R. 798-1 WASHINGTON CO., PA.
MADE BY
SPRAGUE & HENWOOD, INC.
SCRANTON, PA.

APPROVED BY [signature]
DRAWN BY D.M. CHECKED BY D.M. DATE 5-19-58

TYPE 2 BEAM

NOTE:
EVEN STRAND OFF FLUSH EXCEPT WHERE INDICATED AND PAINT WITH TWO COATS OF BITUMASTIC PAINT.

TYPE 1, 1A, 3, 3A, 3B, 4, 5, 5A BEAMS

TYPICAL BEAM SECTIONS
1" = 1'-0"

TYPE 6 BEAM

NO.	REVISION	BY	CH'KD	DATE
1	INCREASED THE EXTENDED LENGTH OF STRANDS.	M.H.J.	W.E.L.	9-15-60

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

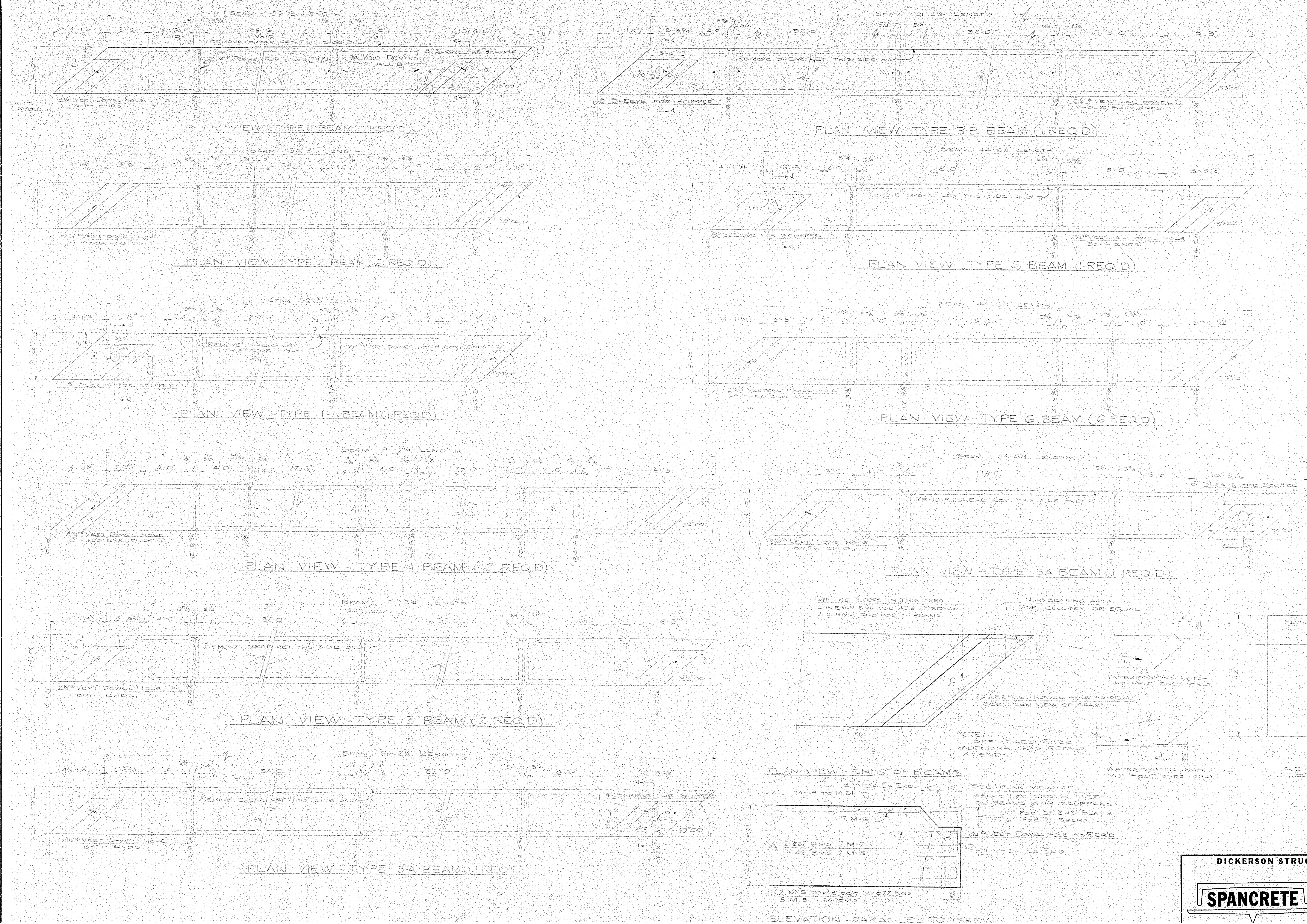
JOB: WASHINGTON CO. LR 798-1 BRIDGE @ STA. 1205+50.00

FOR: MARSOLINO CONST. CO. PA. DEPT. OF HIGHWAYS

SHEET 1 OF 5

JOB NO. 6014

DATE: 6-28-60 | ENGR.: C.G.B. | APPROVED:



A. ESTIMATED CAMBER(S) BE CHECKED BY CONTRACTOR IN FIELD
B. DEFLECTION DUE TO PARAPET & WEARING SURFACE

SPANS	1	2 & 3	4
A	0.89	1.39	0.85
B	0.21	0.52	0.16

PENNA. DEPT. OF HWYS.
APPROVED AS TO DESIGN
AUG 10 1960

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

JOB: WASHINGTON CO. LR 798-1 BRIDGE @ STA 1205+50.00

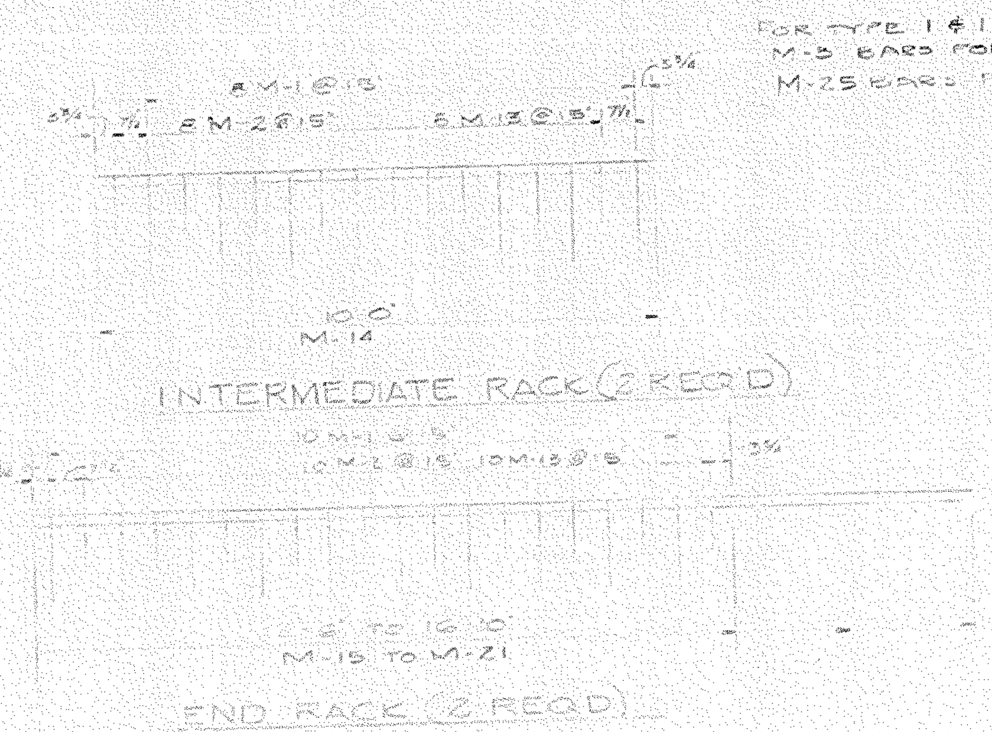
FOR: MARSOLINO CONST. CO
PA. DEPT. OF HIGHWAYS

SHEET 2 OF 3

JOB NO.

DATE: | ENGR.: | APPROVED:

For type 1 & 1A bms substitute
M-3 bars for M-1 bars &
M-25 bars for M-2 bars

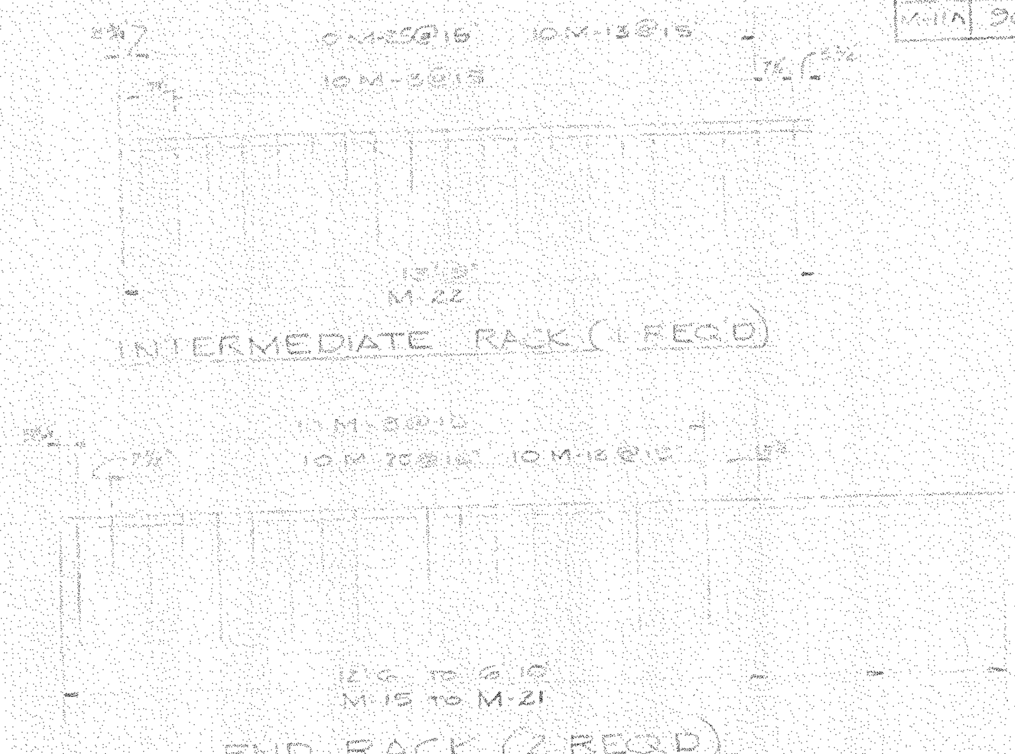


REINFORCING RACK LAYOUT
BEAM TYPES 1 & 2

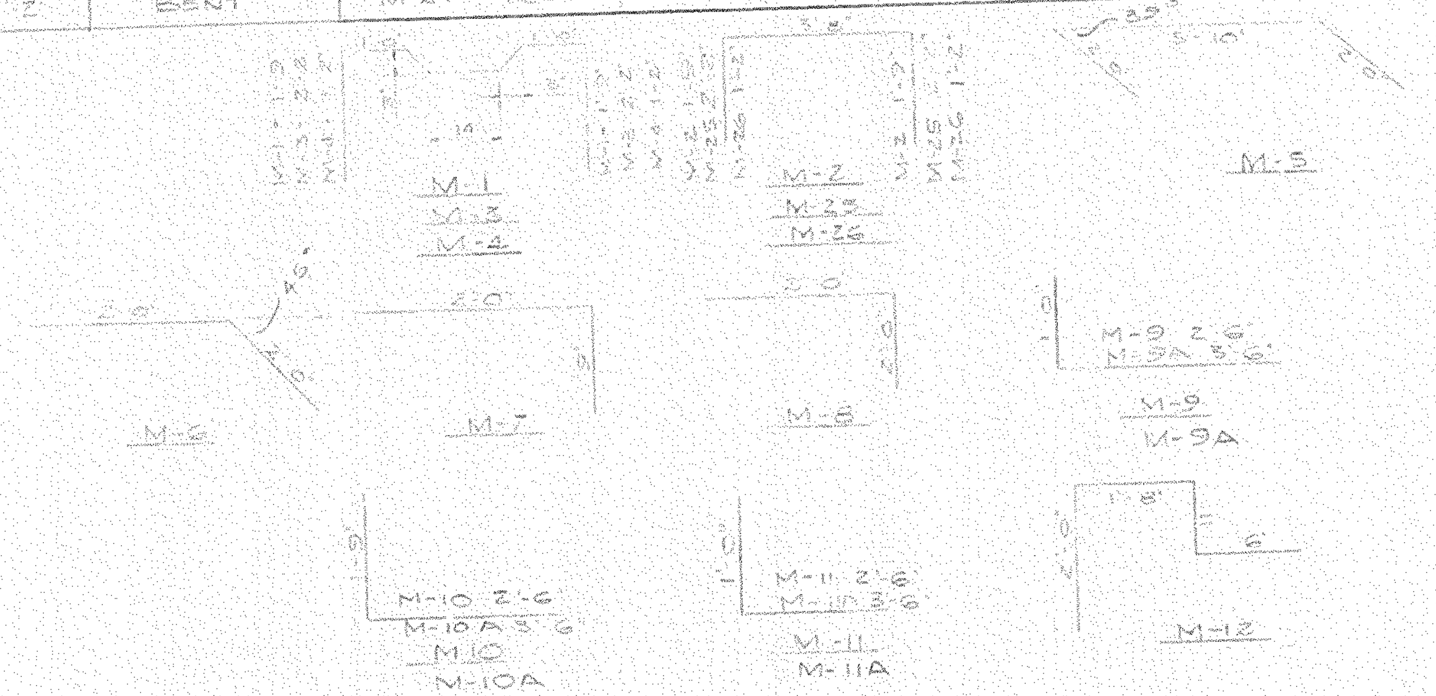
BAR LIST				
MARK	NUMBER	SIZE	LENGTH	DESCRIPTION
M-1	216	5	7'-4"	Bent
M-2	216	5	7'-2"	"
M-3	1086	5	8'-2"	"
M-4	180	5	6'-2"	"
M-5	224	6	9'-10"	"
M-6	448	6	4'-6"	"
M-7	224	6	2'-9"	"
M-8	224	6	4'-0"	"
M-9	360	4	3'-6"	"
M-10	1576	4	2'-3"	"
M-11	234	4	3'-2"	"
M-12	520	4	5'-1"	Bent
M-13	1464	5	5'-9"	Straight
M-14	112	4	10'-0"	"
M-15	64	4	12'-6"	"
M-16	64	4	13'-7"	"
M-17	64	4	13'-11"	"
M-18	64	4	14'-6"	"
M-19	64	4	15'-5"	"
M-20	64	4	15'-9"	"
M-21	64	4	16'-10"	"
M-22	448	4	13'-9"	"
M-23	56	4	8'-11½"	"
M-24	1252	4	2'-0"	Straight

M-25	1064	5	8'-2"	Bent
M-26	162	5	6'-6"	"
M-9A	360	4	4'-6"	"
M-10A	3200	4	5'-5"	"
M-11A	360	4	4'-2"	Bent

LIST OF MATERIALS		
NUMBER	UNITS	DESCRIPTION
118,200	LF	3/8" Prestressing Strand
24	Pcs	39x17" Box Voids 4'-0" Lg
4	Pcs	39x17" 24'-0" Lg
68	Pcs	39x32" 4'-0" Lg
8	Pcs	39x32" 2'-0" Lg
1	Pcs	39x32" 7'-0" Lg
3	Pcs	39x32" 9'-0" Lg
2	Pcs	39x32" 29'-6" Lg
2	Pcs	39x32" 8'-0" Lg
2	Pcs	39x32" 5'-6" Lg
24	Pcs	39x32" 27'-0" Lg
8	Pcs	39x32" 32'-0" Lg
24	Pcs	39x11" 4'-0" Lg
6	Pcs	39x11" Box Voids 31'-0" Lg
712	LF	2¼" Type 'A' Sonovoid
70	Pcs	1⅛" Transverse Rods 7'-11½" Lg
140	Pcs	3x3x½" ℄ Washers
140	Pcs	1" Heavy Hex Nuts
190	Pcs	½" Fascia Forming Inserts
36	Sq.Ft	½" Thick Celotex or Equal

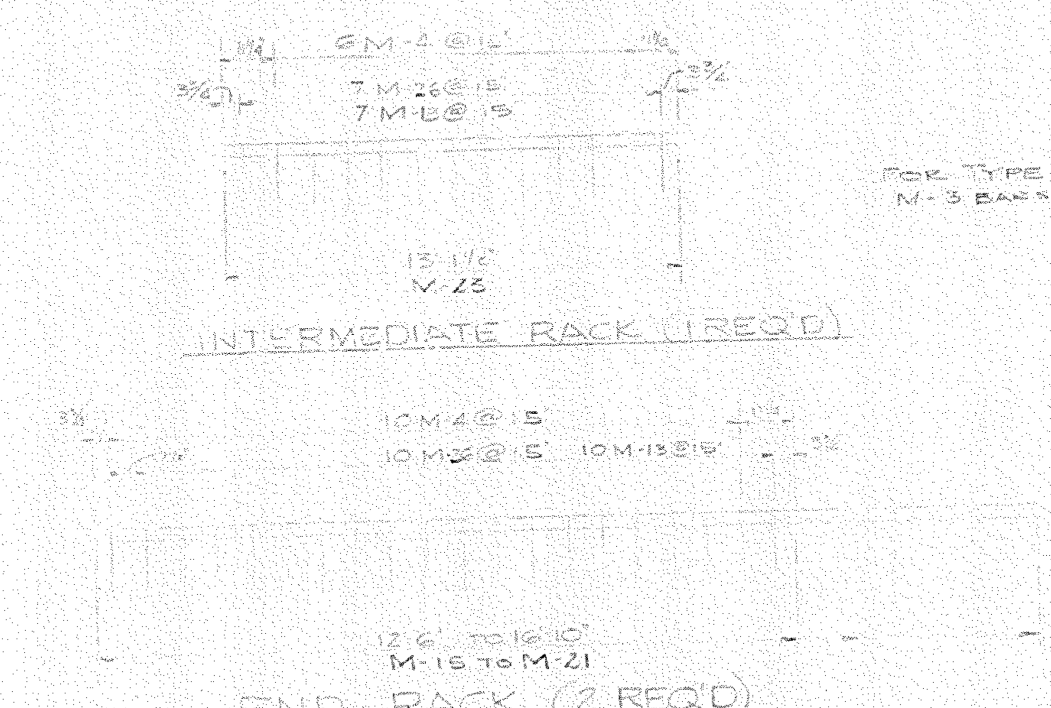


REINFORCING RACK LAYOUT
BEAM TYPES 3 & 4



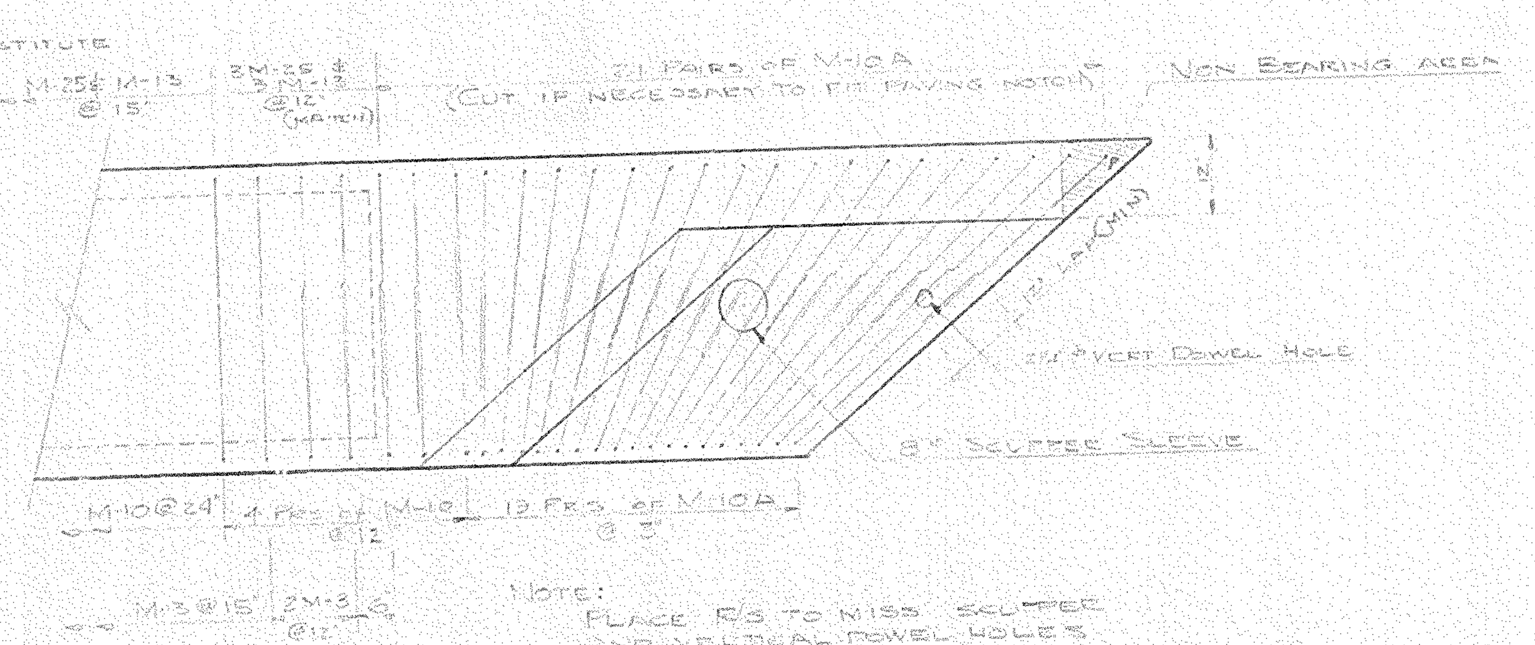
NOTE:
CUT STEEL TO FIT IN BEAMS
WITH LARGE PAVING NOTCHES

BEAM CONCRETE QUANTITIES AND WEIGHTS		
BEAM TYPE	CONCRETE PER BEAM (CU YDS)	SHIPPING WEIGHT (TONS)
1 & 1A	16.1	33.8
2	11.9	24.9
3	22.6	47.4
3A & 3B	23.4	49.0
4	23.9	50.0
5 & 5A	12.4	25.9
6	8.2	17.2



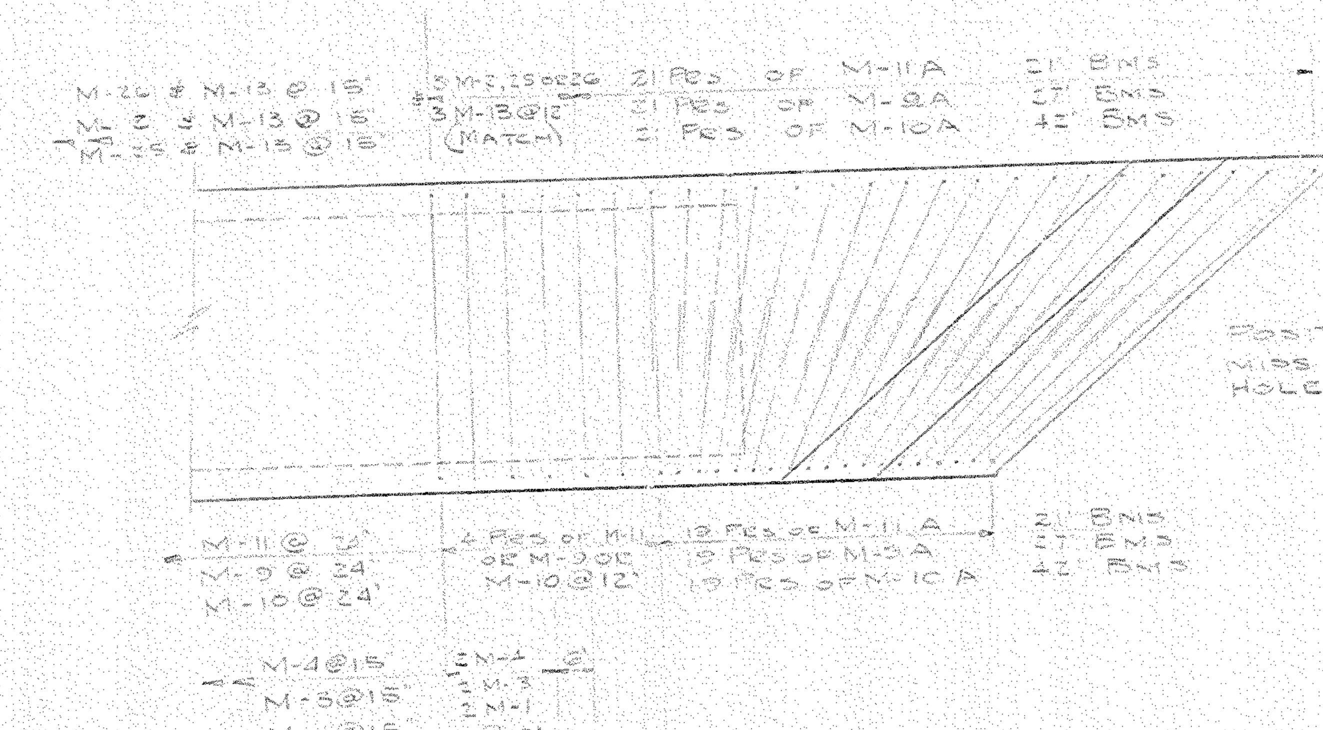
REINFORCING RACK LAYOUT
BEAM TYPES 5 & 6

For Type 5 & 5A bms substitute
M-3 bars for M-4 bars



Note:
Place R/S to miss scupper
and vertical dowel holes

REINFORCING DETAILS ON
BEAM ENDS WITH SCUPPERS
½" = 1'-0"



Position as req'd to
miss lifting inserts & dowel
holes

REINFORCING DETAILS ON
BEAM ENDS WITHOUT SCUPPERS

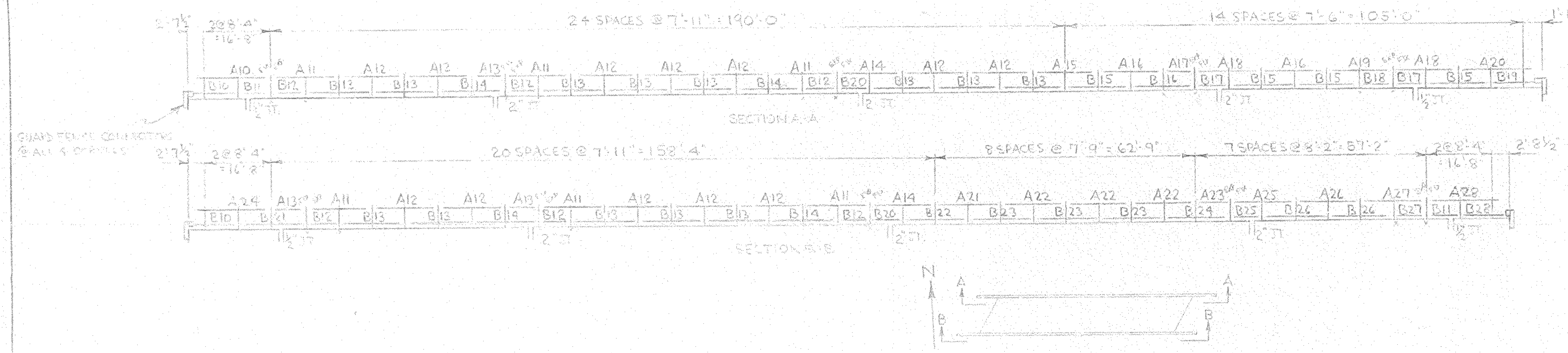
PENNA. DEPT. OF HWYS
APPROVED AS TO DESIGN
AUG 10 1960

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

JOB: WASHINGTON CO. LR 798-1 BRIDGE @ STA 1205+50.00		SHEET 3 OF 3
FOR: MARSOLINO CONST. CO. PA. DEPT. OF HIGHWAYS		JOB NO. 6014
DATE: 7-5-60	ENGR.: C.G.B.	APPROVED:

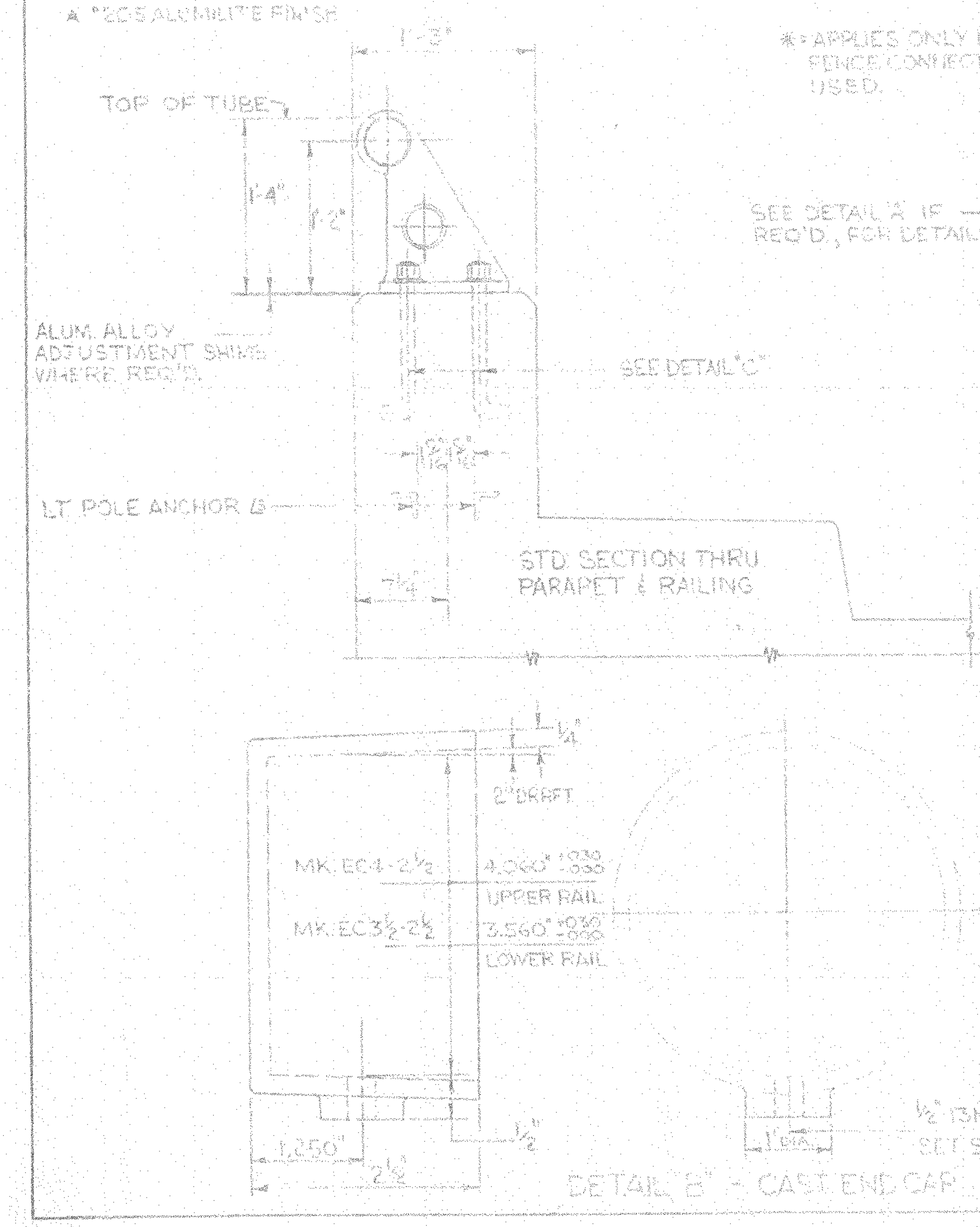
Ref. Dwg. S-3661A
P-3661

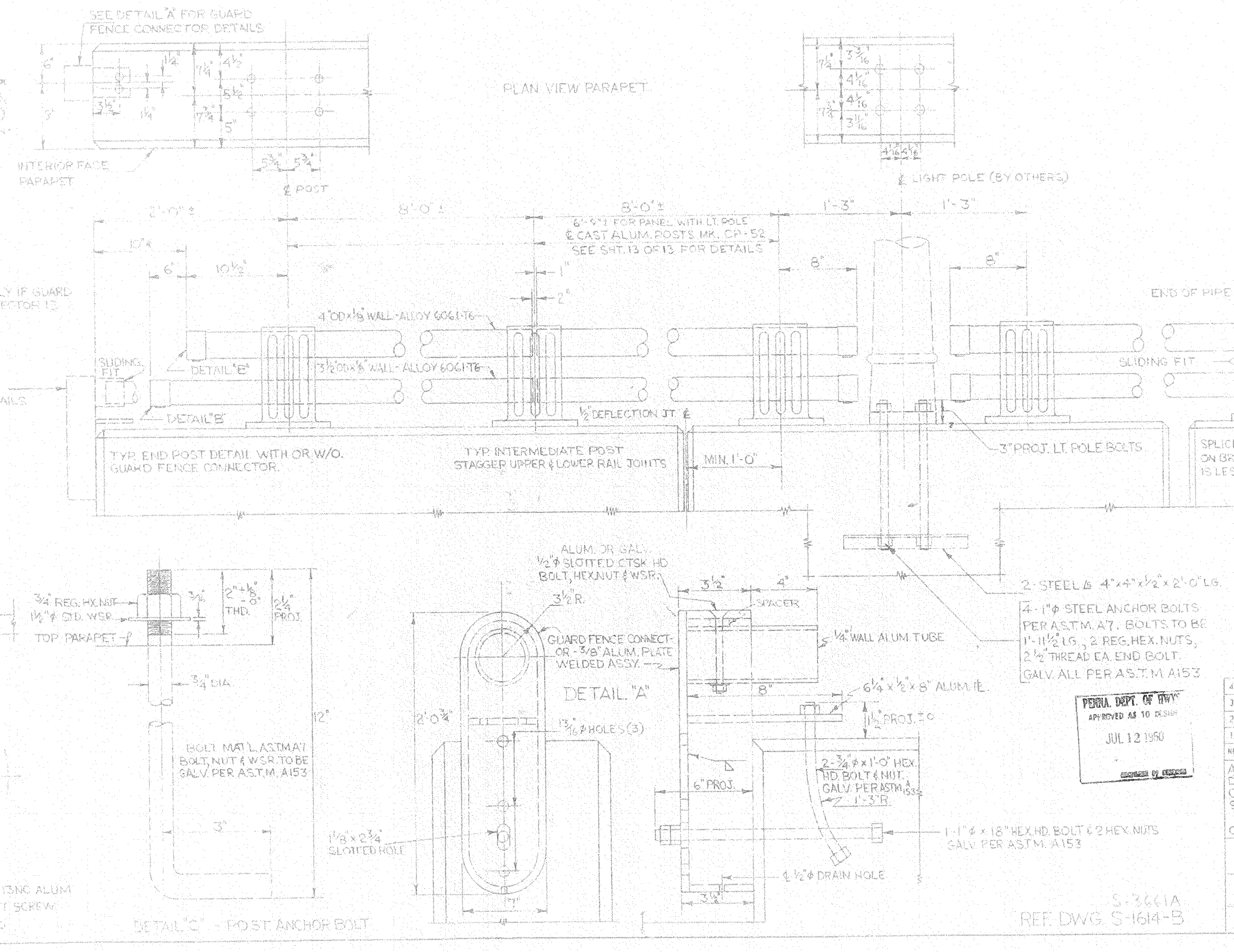


GEN. NOTES & SPECS.

CAST POSTS TO BE PER A.S.T.M. SPEC. B108, ALLOY SG 70A, CONDITION T6.
TUBING DO B235, DO 50 HA, DO T6.
SHIMS DO B209, DO 55CA, DO [illegible]
SET SCREWS DO B211, DO 24S, DO T4 ✱
ANCHOR BOLTS, NUTS, WASHERS TO BE HOT DIP GALV. PER A.S.T.M. [illegible] A153.
BOLT MAT'L. TO BE PER A.S.T.M. SPEC. A7 (FORM 408-11.24, PA. DEPT. HWYS.)
POSTS TO SET NORMAL TO GRADE - POST ANCHOR BOLTS TO BE SET PERPENDICULAR TO TOP OF PARAPET.
LT. POLE ANCHOR BOLTS TO BE SET TRULY VERTICAL.
TOLERANCES TO BE IN ACCORD. WITH MANUFACTURER'S AND INDUSTRY'S STANDARDS.
SURFACES OF ALUM. WHICH COME IN CONTACT WITH CONCRETE OR STEEL SHALL BE GIVEN A COAT OF "ALUMILASTIC" OR EQUAL. (NOT BY M.F.M.)
UPPER & LOWER RAIL JOINTS SHALL BE STAGGERED.

✱ "205 ALUMILITE FINISH





BILL OF MATERIAL CONTINUED			
QTY.	DESCRIPTION	MARK	LENGTH
1	3½" OD × ⅛" TUBE	B19	9'-1½"
2	DO	B20	7'-7¼"
1		B21	15'-11¼"
1		B22	15'-6"
3		B23	15'-4"
1		B24	15'-10"
1		B25	7'-10¼"
2		B26	16'-2"
1		B27	8'-1"
1		B28	10'-9½"

BILL OF MATERIAL			
QTY.	DESCRIPTION	MARK	LENGTH
81	CAST POST	CP-52	
4	TOP RAIL CAP	EC4-2½	
[illegible]	BOT. RAIL CAP	EC3½-2½	
324	POST ANCHOR	BY FOOT	FIT
4	GUARD FEN. CON.	BY FOOT	FIT
1	4" OD × ⅛" TUBE	A10	18'-2¼"
6	DO	A11	15'-9"
12		A12	15'-8"
3		A13	7'-7¼"
2		A14	15'-6¼"
1		A15	15'-3"
2		A16	14'-[illegible]
1		A17	7'-5"
2		A18	14'-5⅝"
1		A19	14'-11"
1		A20	15'-11½"
1		A21	15'-4"
3		A22	15'-4"
1		A23	8'-1"
1		A24	19'-4"
1		A25	16'-6"
1		A26	16'-2"
1		A27	16'-2"
1		A28	18'-2¼"
2	3½" OD × ⅛" TUBE	B10	10'-8¼"
2	DO	B11	8'-2¼"
6		B12	7'-10"
13		B13	15'-8"
4		B14	15'-6¼"
4		B15	14'-10"
1		B16	14'-11"
2		B17	7'-2¼"
1		B18	7'-5"

PENNA. DEPT. OF HWYS.
APPROVED AS TO DESIGN
JUL 12 1950

NO.	DATE	BY	REVISIONS
4			
3			
2			
1			

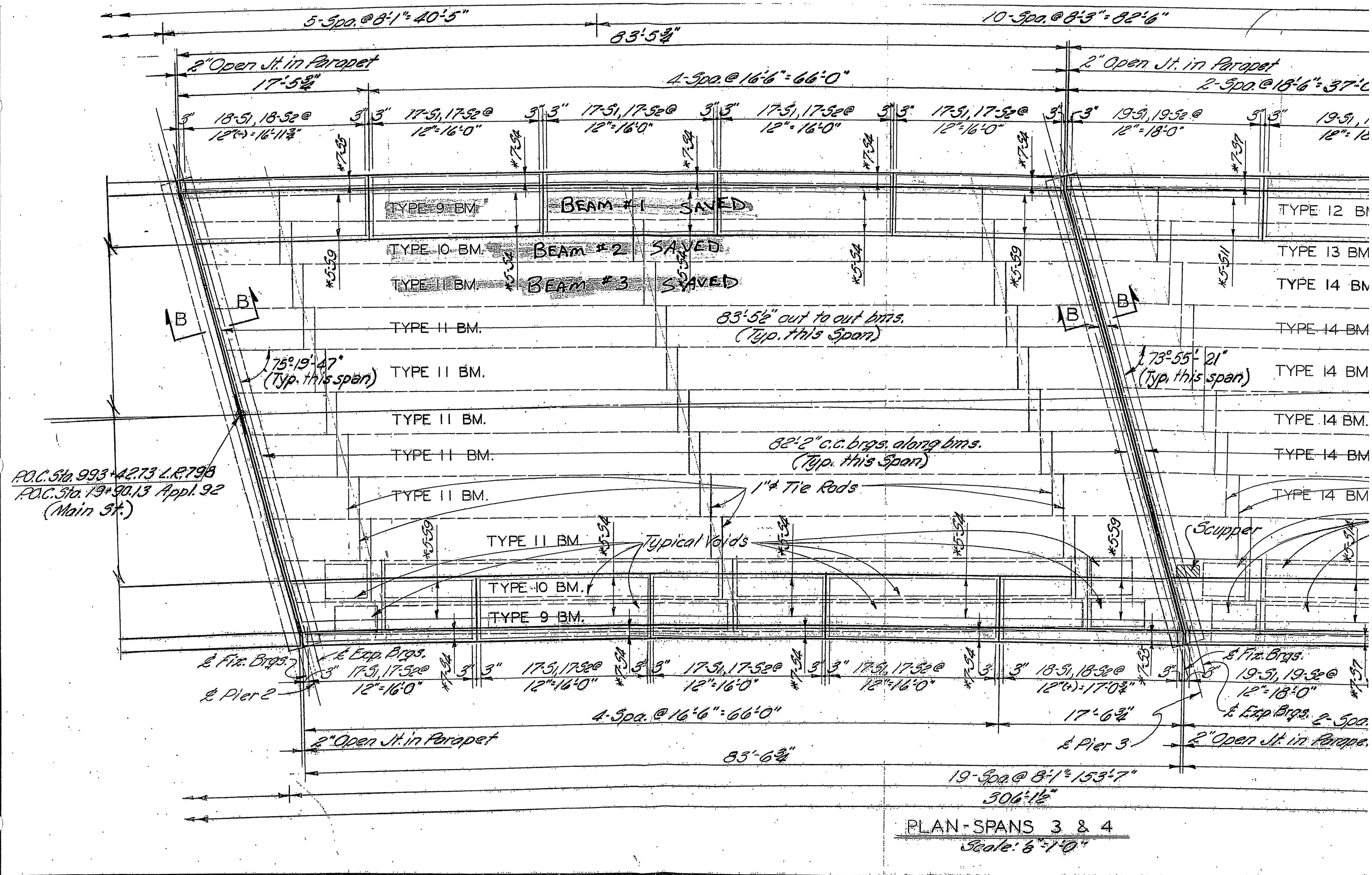
ALUMINUM HANDRAIL DETAILS FOR PENNA. DEPT. HWYS. L.R. 798 SECT. 1
COUNTY WASHINGTON
STA. 1205+50.00
L.R. 798 UNDER L.R. 62093 (RE.DC.)
CONTRACTOR: - W.P. DICKERSON & SON, INC.

MICHAEL FLYNN MANUFACTURING COMPANY
PHILADELPHIA 24, PENNSYLVANIA

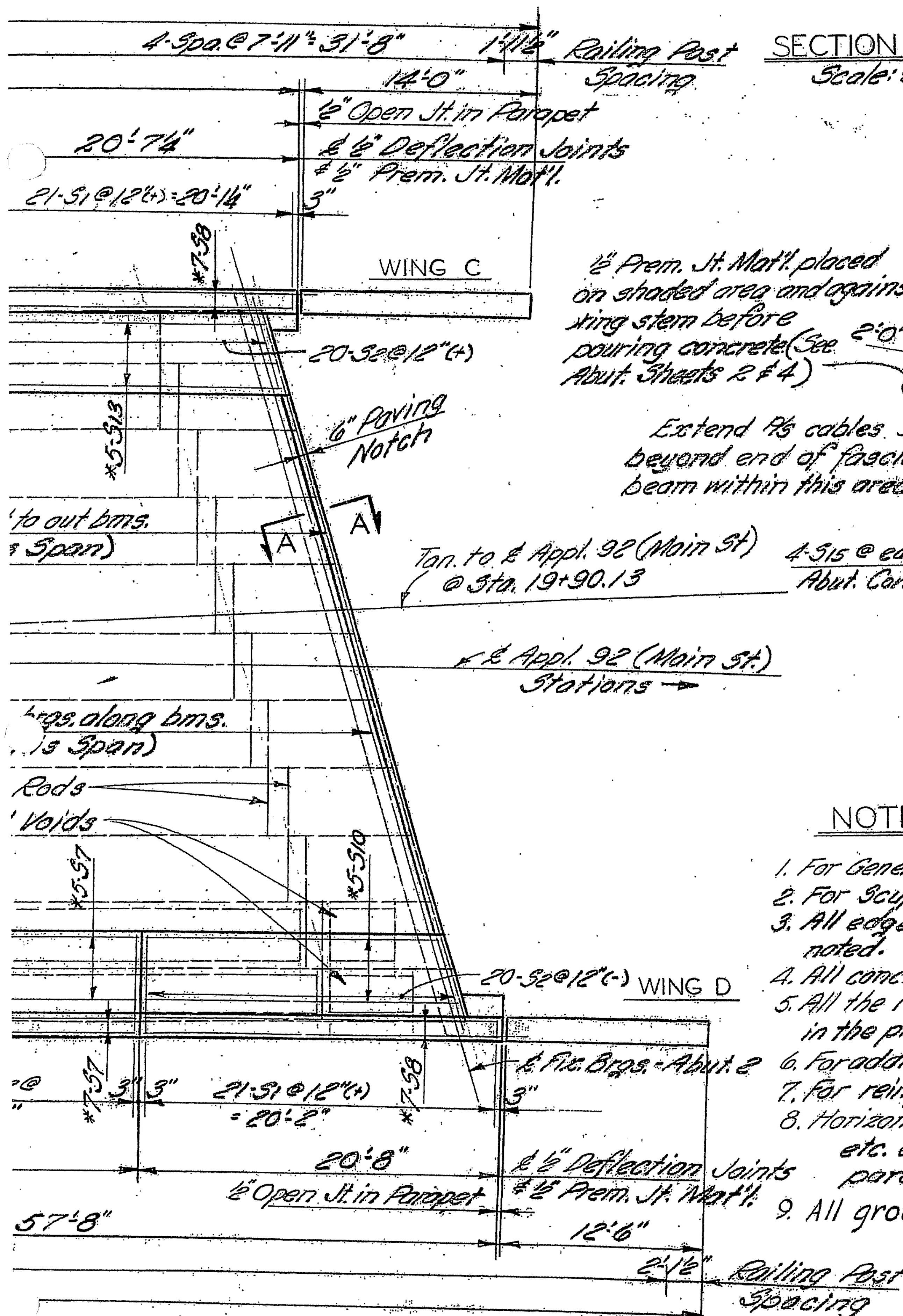
DRAWN BY T.R. CHECKED BY
DATE 6-24-50 DATE
ISSUE NO.

S-3661A
REF. DWG. S-1614-B

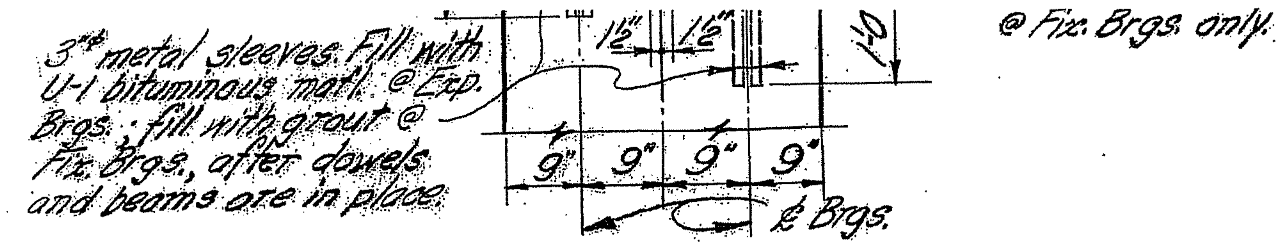
ORDER NO. XA-6804 DRAWING NO. 10 OF 13 D



PLAN - SPANS 3 & 4
Scale: ⅛"=1'-0"

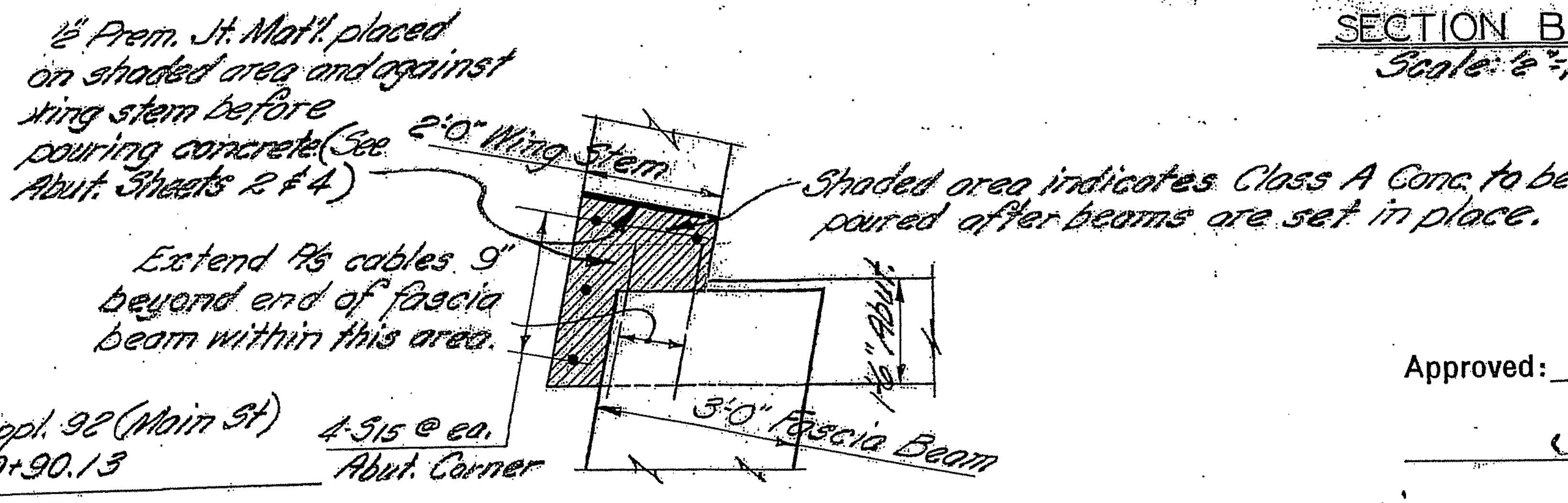


SECTION A-A
Scale: ½" = 1'-0"



Note: Dowels, dowel holes and sleeves occur in all beams at Fix. Brgs. and in fascia beams only at Exp. Brgs.

SECTION B-B
Scale: ½" = 1'-0"



TYPICAL CORNER DETAIL
Scale: ¾" = 1'-0"

Approved: JAN 24 1961

K. H. Jensen
Bridge Engineer

NOTES

1. For General Notes see Sheet 1.
2. For Scupper location and details see Sheet 7.
3. All edges shall have a 1"×1" chamfer unless otherwise noted.
4. All concrete shall be Class A, except Box Beams.
5. All the 1"⌀ Anchor dowels in brg. seats shall be included in the price paid for P/S, P/C Box Beams.
6. For additional superstructure details see sheet 6.
7. For reinf. bar schedule, see sheet 6.
8. Horizontal parapet dimensions, railing post spacing, etc. are measured along inside face of parapet.
9. All grout to be non-shrink grout.

Commonwealth of Pennsylvania

Department of Highways
BRIDGE UNIT

WASHINGTON COUNTY

L.R. 798 SECTION I-A

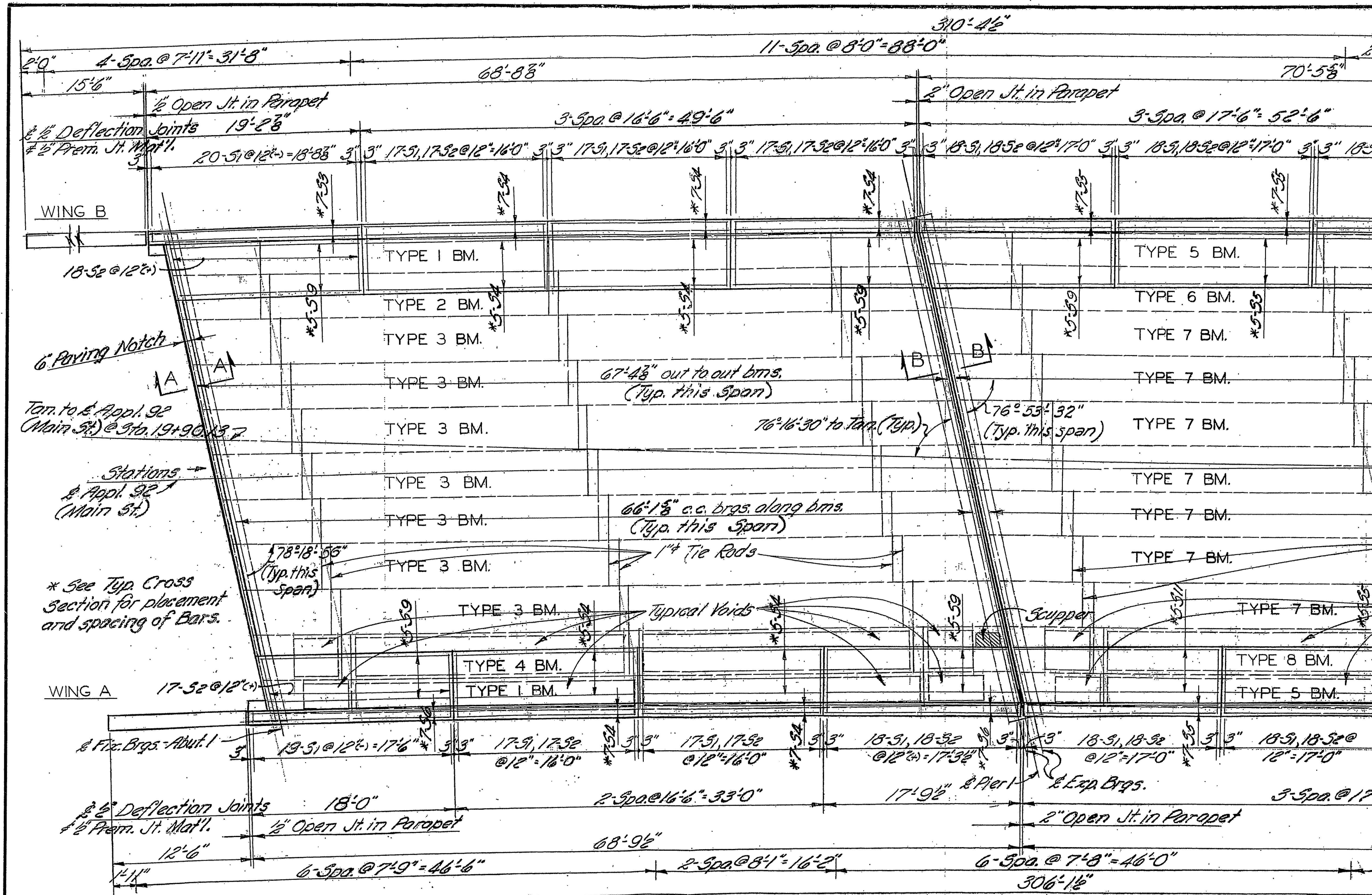
L.R. 798 UNDER APPL. 92 (MAIN ST.)

SLAB PLAN

STA. 993 + 42.73

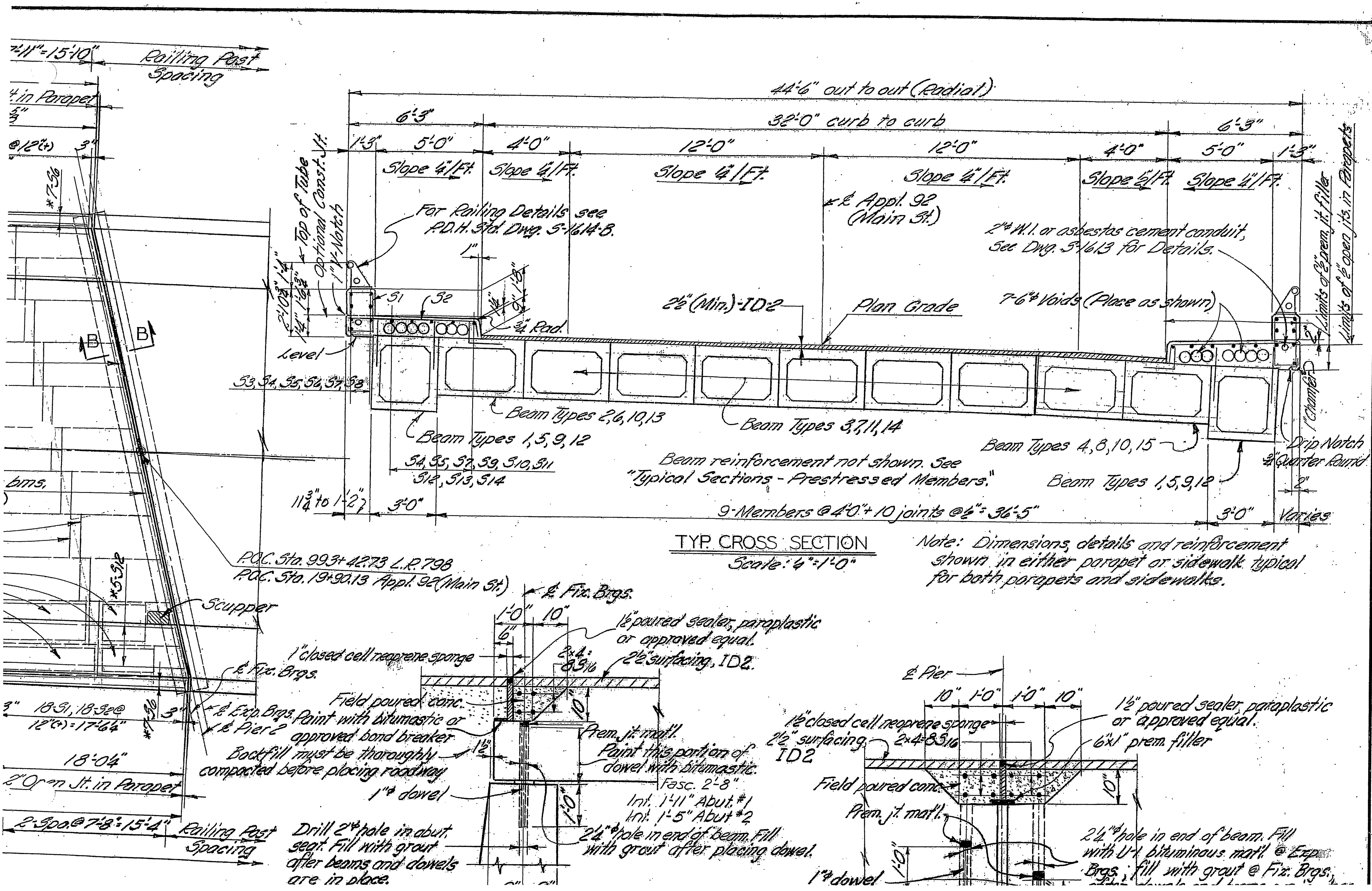
SCALE: AS NOTED
SHEET 5 OF 9

S-3530-A

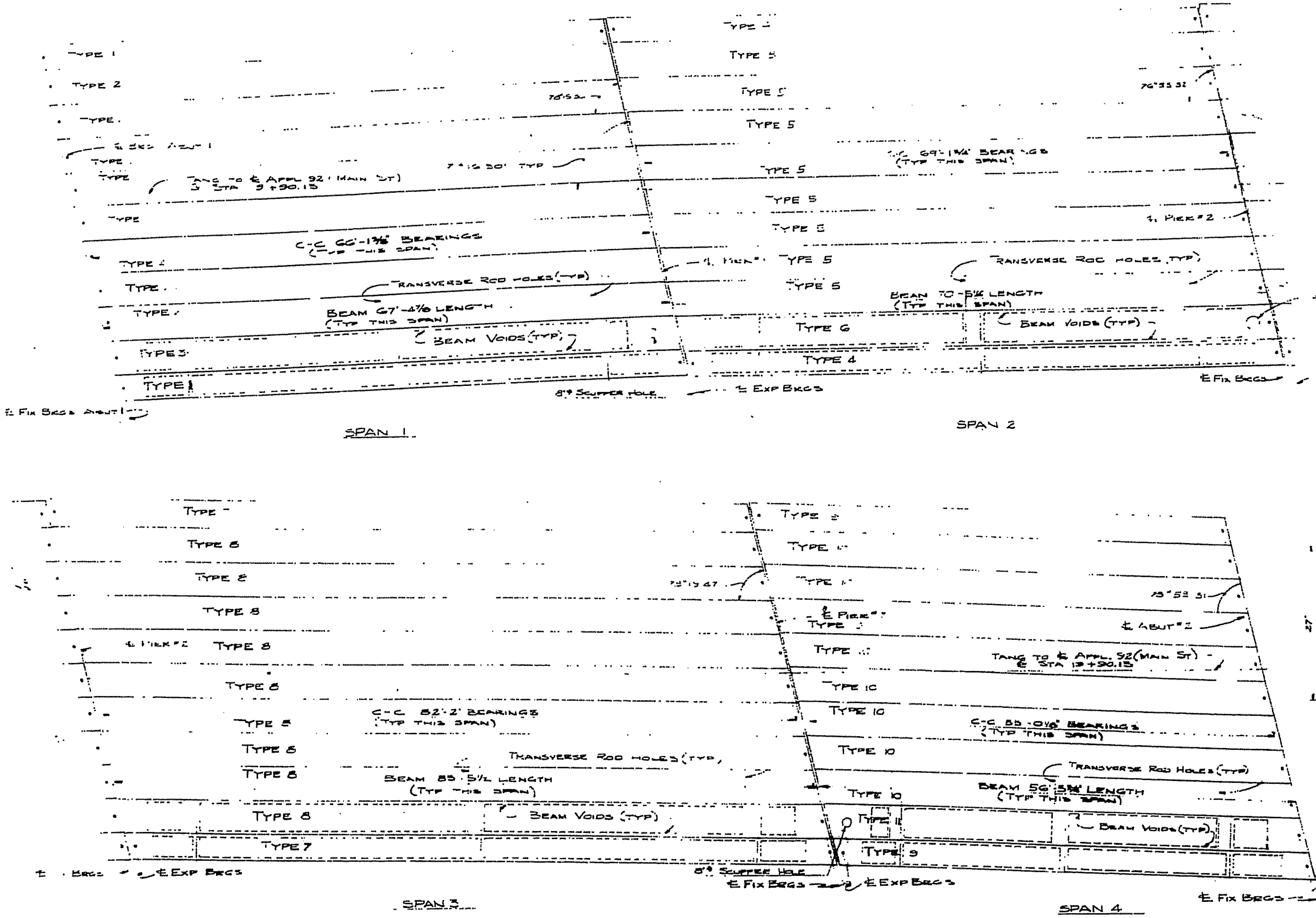


PLAN-SPANS 1 & 2

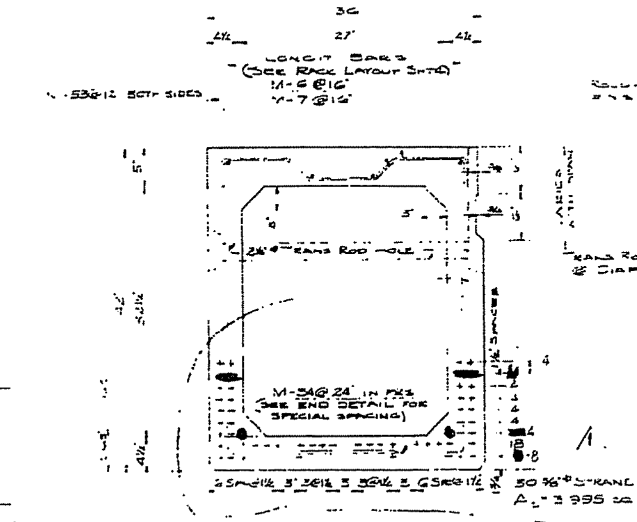
Scale: ⅛" = 1'-0"



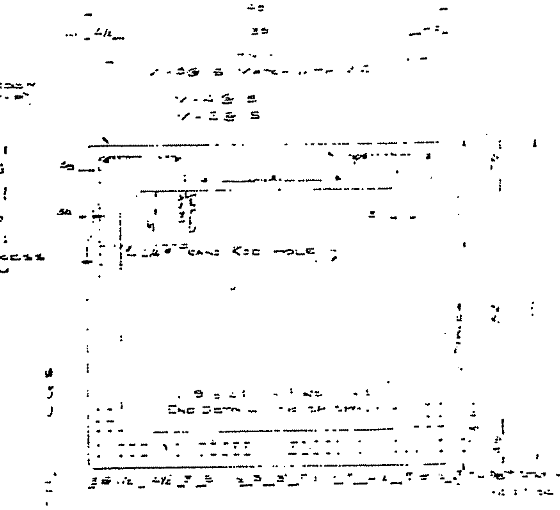
44'-6" out to out (Radial)
32'-0" curb to curb
6'-3"
1'-3"
5'-0"
4'-0"
12'-0"
Slope ¼"/Ft.
℄ Appl. 92 (Main St.)
For Railing Details see P.D.H. Std. Dwg. S-1614-8.
2"ϕ W.I. or asbestos cement conduit, See Dwg. S-1613 for Details.
Top of Tube
Optional Const. Jt.
1" V-Notch
2½" (Min.)-ID2
Plan Grade
7-6"ϕ Voids (Place as shown)
¾" Rad.
Level
S3, S4, S5, S6, S7, S8
Beam Types 2,6,10,13
Beam Types 3,7,11,14
Beam Types 1,5,9,12
Beam Types 4,8,10,15
S4, S5, S7, S9, S10, S11
S12, S13, S14
Beam reinforcement not shown. See "Typical Sections - Prestressed Members."
1" Chamfer
Drip Notch
¾" Quarter Round
Limits of 2" prem. jt. filler
Limits of ¾" open jts. in Parapets
11¾" to 1'-2"
3'-0"
9-Members @ 4'-0" + 10 joints @ ½" = 36'-5"
Varies
TYP. CROSS SECTION
Scale: ¼" = 1'-0"
Note: Dimensions, details and reinforcement shown in either parapet or sidewalk typical for both parapets and sidewalks.
Railing Post Spacing
in Parapet
*7-S6
B
P.O.C. Sta. 993+42.73 L.R. 798
P.O.C. Sta. 19+90.13 Appl. 92 (Main St.)
Scupper
*5-S12
℄ Fix. Brgs.
℄ Exp. Brgs.
℄ Pier 2
18-S1, 18-S2 @ 12"(+) = 17'-6¼"
18'-0¼"
2" Open Jt. in Parapet
2-Spa. @ 7'-8" = 15'-4"
Railing Post Spacing
1'-0"
10"
6"
1½" poured sealer, paraplastic or approved equal.
2½" surfacing, ID2.
2×4: 8.3 ½
1" closed cell neoprene sponge
Field poured conc.
Paint with bitumastic or approved bond breaker
Backfill must be thoroughly compacted before placing roadway
Prem. jt. mat'l.
Paint this portion of dowel with bitumastic.
Fasc. 2'-8"
Int. 1'-11" Abut. #1
Int. 1'-5" Abut. #2
1"ϕ dowel
Drill 2"ϕ hole in abut. seat. Fill with grout after beams and dowels are in place.
2¼"ϕ hole in end of beam. Fill with grout after placing dowel.
℄ Pier
1½" poured sealer, paraplastic or approved equal.
1½" closed cell neoprene sponge
2½" surfacing ID2
6"×1" prem. filler
2¼"ϕ hole in end of beam. Fill with U-1 bituminous mat'l. @ Exp. Brgs.; fill with grout @ Fix. Brgs.



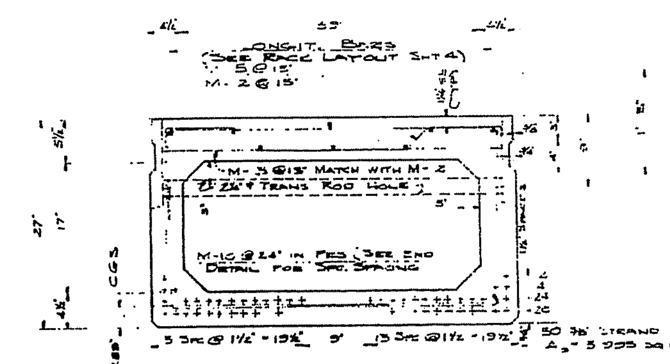
BEAM LAYOUT
1/8" = 1'-0"



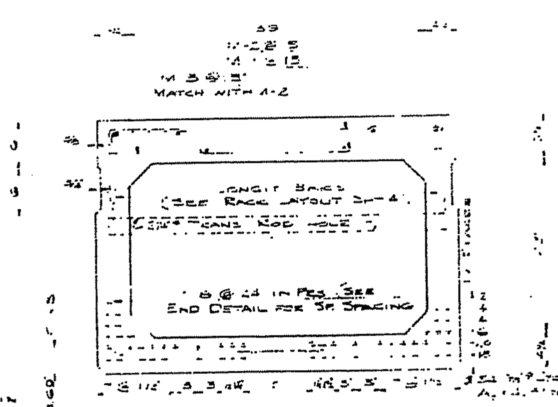
BEAM SECTION
BEAM TYPES 1,4,7 & 9
1" = 1'-0"



BEAM SECTION
BEAM TYPE 8



BEAM SECTION
BEAM TYPES 10 & 11
1" = 1'-0"

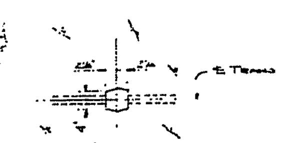


BEAM SECTION
BEAM TYPES

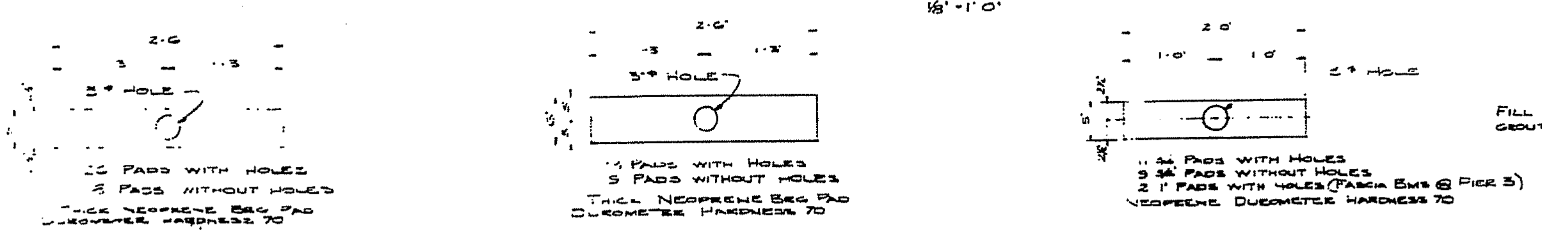
PRESTRESS NOTES

MINIMUM STRAND STRENGTH SHALL BE 250000 PSI
MINIMUM CONCRETE STRENGTH @ 28 DAYS:
BEAM TYPES 1,4,7,4 9 => 5440 PSI
BEAM TYPES 2,3,5,& 6 => 5440 PSI
BEAM TYPES 8 => 5440 PSI.
BEAM TYPES 10 & 11 => 5600 PSI.
MINIMUM CONCRETE STRENGTH @ RELEASE OF PRESTRESS
BEAM TYPES 1,4,7,4 9 => 4540 PSI.
BEAM TYPES 2,3,5,& 6 => 4540 PSI.
BEAM TYPES 8 => 4175 PSI
BEAM TYPES 10 & 11 => 4650 PSI
INITIAL PRESTRESS FORCE ON EACH ⅜"ϕ STRAND SHALL BE 4000 LBS.
ALL MATERIALS & WORKMANSHIP SHALL BE IN ACCORDANCE WITH PDH FORM 408/54 & SECT 6-24 JAN 10, 1958

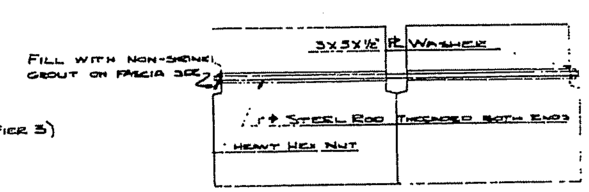
PENNA DEPT OF HWYS
APPROVED
ENGINEER IN CHARGE



TIE ROD RECESS DETAIL



BEARING PADS



INTERBEAM CONNECTION
½" = 1'-0"

No.	Description	By	Ck'd	Date
1	Revised strand pattern on beam types 1,4,7 & 9	F.C.F.	W.E.L.	3-24-61

REVISIONS

BMS ID: 62 4049 0030 1265

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

JOB: WASHINGTON COUNTY
LR 738 UNDER APPL 92 (MAIN ST.
STA. 993+42.73

FOR: YARSOLINO CONST CO.
PA. DEPT. OF HIGHWAYS

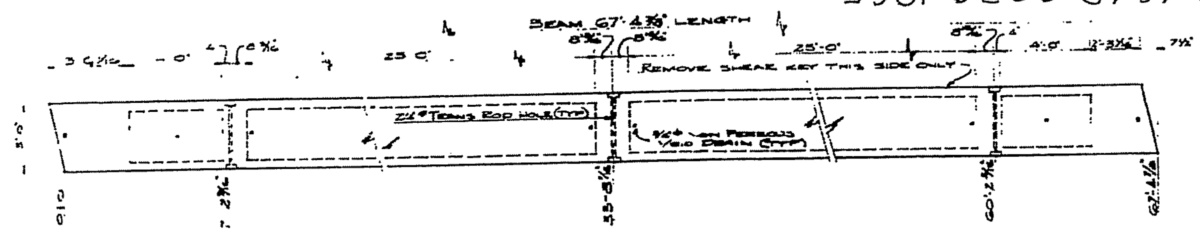
REVISIONS

DATE	MADE BY	CHECKED BY	REMARKS
2-27-61	CGB		PEEL APPR 2-20-61

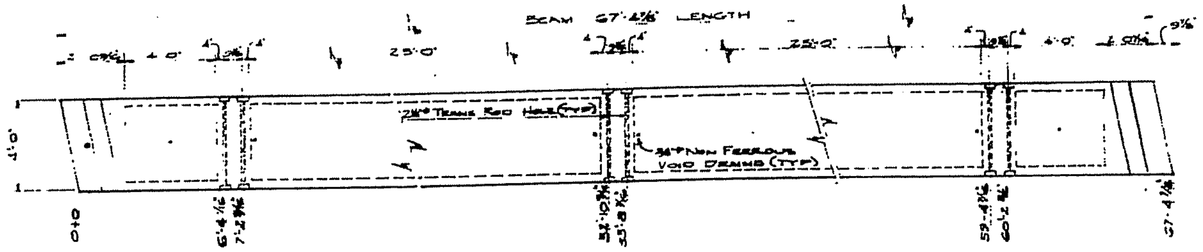
DWG. NO.	REF. DWG. NO.	JOB NO.
F-3530	S 3530-A	6017

DRAWN BY: C.G.B. | DATE: 1-9-61 | SHEET 1 OF 1
CHECKED BY: CGB | DATE: [illegible]

BMS ID: 62 4049 0030 1265



PLAN VIEW-BEAM TYPE 1
1/4" = 1'-0"



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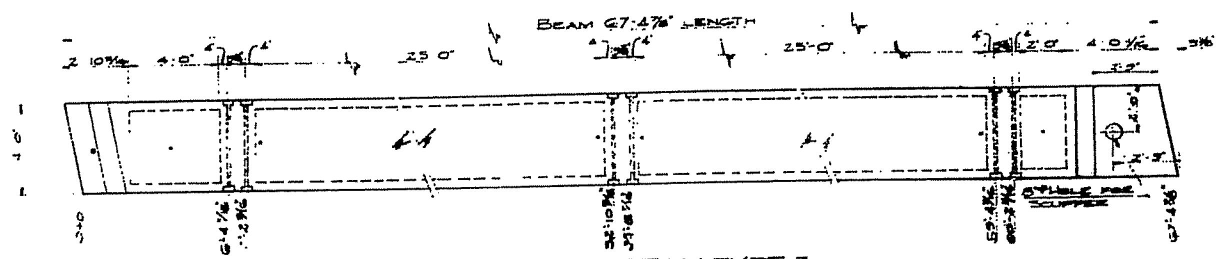
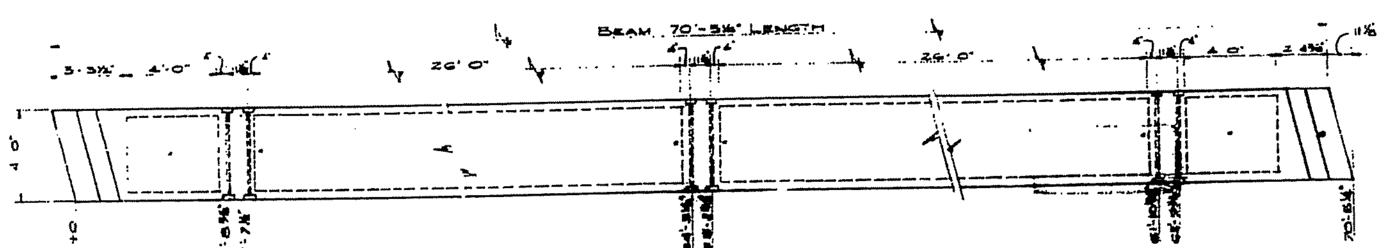


PLAN VIEW-BEAM TYPE 3
1/4" = 1'-0"



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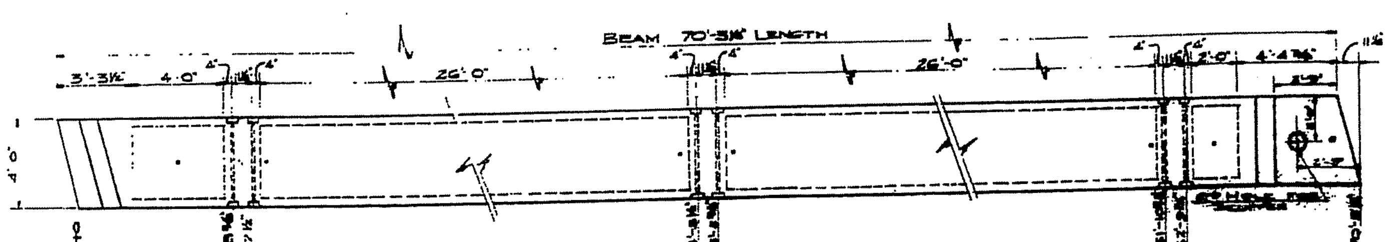
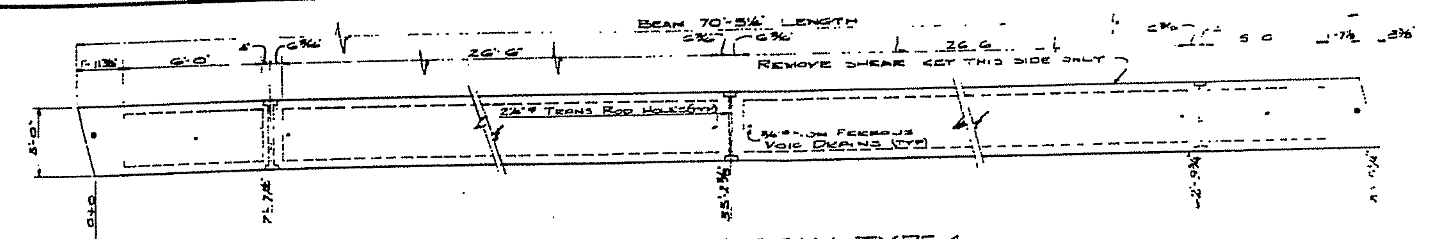
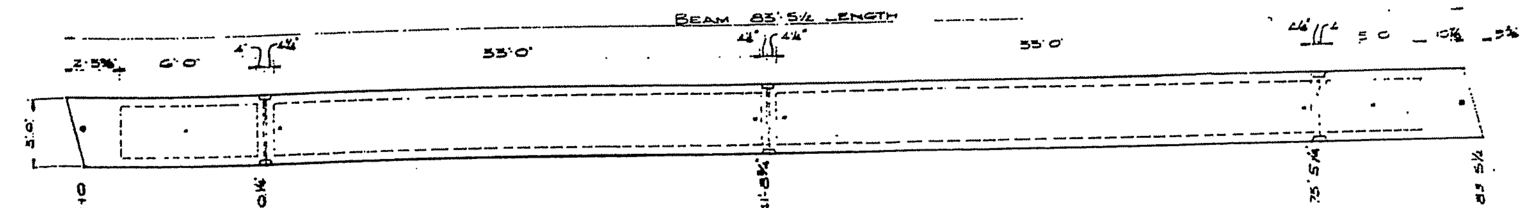


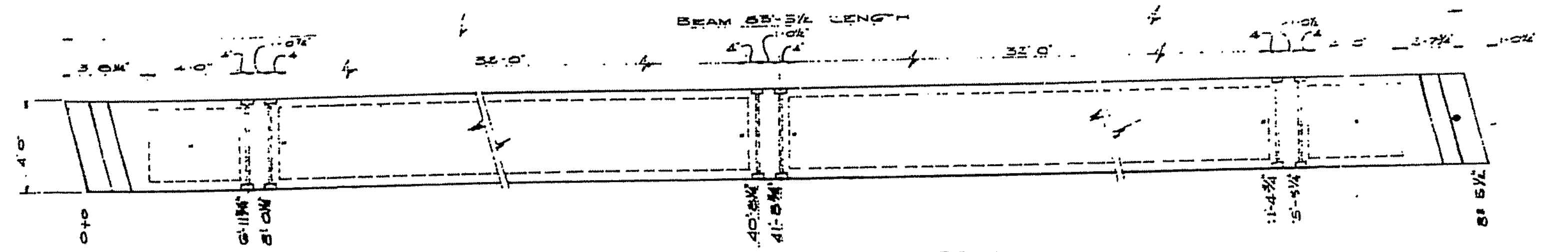
PLAN VIEW-BEAM TYPE 6
1/4" = 1'-0"



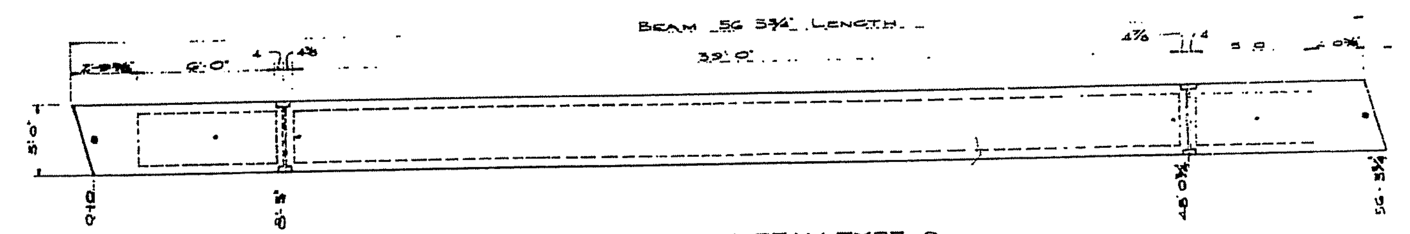
PLAN VIEW-BEAM TYPE 4
1/4" = 1'-0"



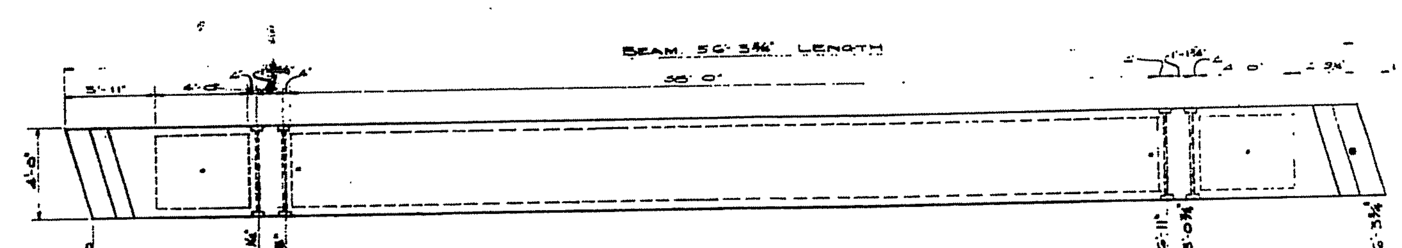
PLAN VIEW-BEAM TYPE 7



PLAN VIEW-BEAM TYPE 8
1/4" = 1'-0"



PLAN VIEW-BEAM TYPE 9
1/4" = 1'-0"



PLAN VIEW-BEAM TYPE 10
1/4" = 1'-0"

NOTE: FOR PLAN VIEW OF BEAM TYPE 11 SEE SHEET 3

PENNA. DEPT. OF HWYS.
APPROVED AS TO DESIGN
MAR 8 1961

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

JOB: WASHINGTON COUNTY I-A
LR 798 UNDER APPL. 92 (MAIN ST.)
STA. 993+42.73

FOR: MARSOLINO CONST. CO.
PA. DEPT. OF HIGHWAYS

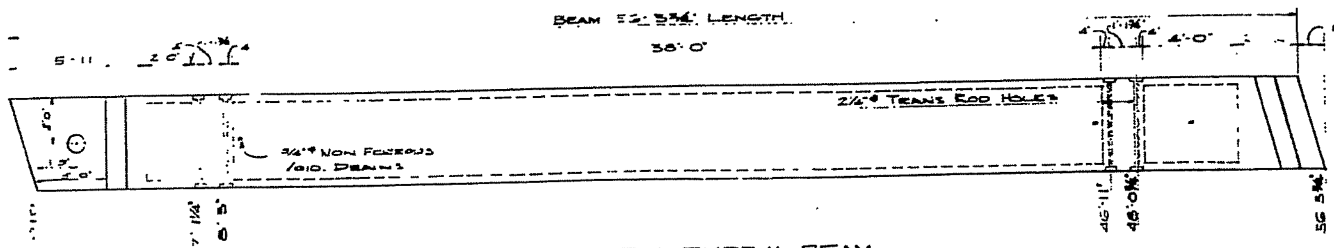
REVISIONS

DATE	MADE BY	CHECKED BY	REMARKS
2-27-61	CGB	CGB	PREL. APPR. 2-20-61

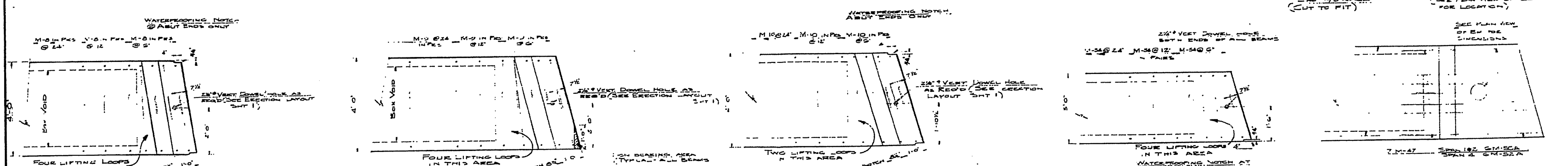
DWG. NO.	REF. DWG. NO.	JOB NO.
P-3530	S-3530A	6017

DRAWN BY	DATE	SHEET
C.G.B.	1-11-61	2 of 4
CHECKED BY C.G.B.	2-2-61	

BMS ID: 62 4049 0030 1265



PLAN VIEW - TYPE II BEAM
½" = 1'-0"



PLAN VIEW
ENDS OF 4' BEAMS
SPANS 1 & 2
½" = 1'-0"

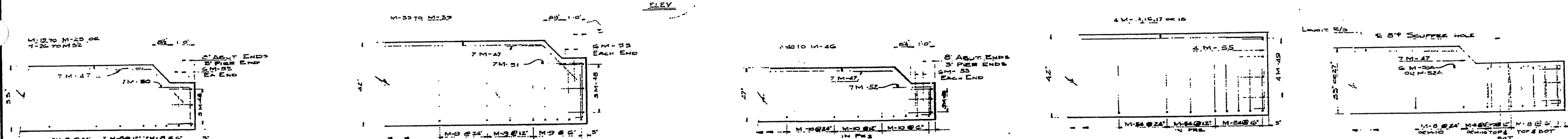
PLAN VIEW
ENDS OF 4' BEAMS
SPAN 3
½" = 1'-0"

PLAN VIEW
ENDS OF 4' BEAMS
SPAN 4
½" = 1'-0"

PLAN VIEW
ENDS OF 3' BEAMS
ALL SPANS
½" = 1'-0"

PLAN VIEW - BEAM ENDS
WITH SCUPPER HOLES
½" = 1'-0"

ELEV



ELEVATION - 4' BEAMS
SPANS 1 & 2
½" = 1'-0"

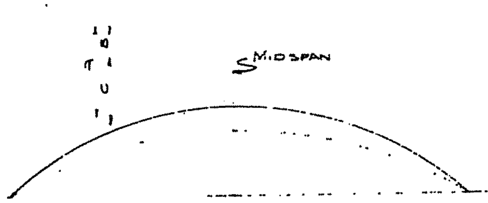
ELEVATION - 4' BEAMS
SPAN 3
½" = 1'-0"

ELEVATION - 4 BEAMS
SPAN 4
½" = 1'-0"

ELEVATION - 3' BEAMS
ALL SPANS
½" = 1'-0"

ELEVATION - BEAM ENDS
WITH SCUPPER HOLES
½" = 1'-0"

BEAM TYPE	CONCRETE (CU. YDS)	WEIGHT (TONS)	NUMBER OF BEAMS REQ'D
1	13.[illegible]	27.8	2
2	14.15	30.2	8
3	14.60	31.1	1
4	12.92	27.6	2
5	14.89	31.8	8
6	15.34	32.7	1
7	15.11	32.2	2
8	20.19	43.1	9
9	10.63	22.7	2
10	11.00	23.5	8
11	11.33	24.2	1



SPAN	A	B	C
1 & 2	1.49"	0.47"	1.02"
3	1.15	0.47	0.68"
4	1.37	0.51	1.06"

A: CAMBER DUE TO P/S FORCE (TO BE CHECKED BY CONTRACTOR IN FIELD.)
B: DEFLECTION DUE TO CURB, PARAPET, SIDEWALK & WEARING SURFACE
C: NET CAMBER
PRESTRESS CAMBER & DEAD LOAD DEFLECTION DATA SHOWN IS THEORETICAL & MAY VARY WITH CONCRETE STRENGTHS, VARIABLE PRESTRESSING CONDITIONS & PRESTRESS LOSSES

PENNA. DEPT. OF HWYS.
APPROVED AS TO DESIGN
MAR 8 1961

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

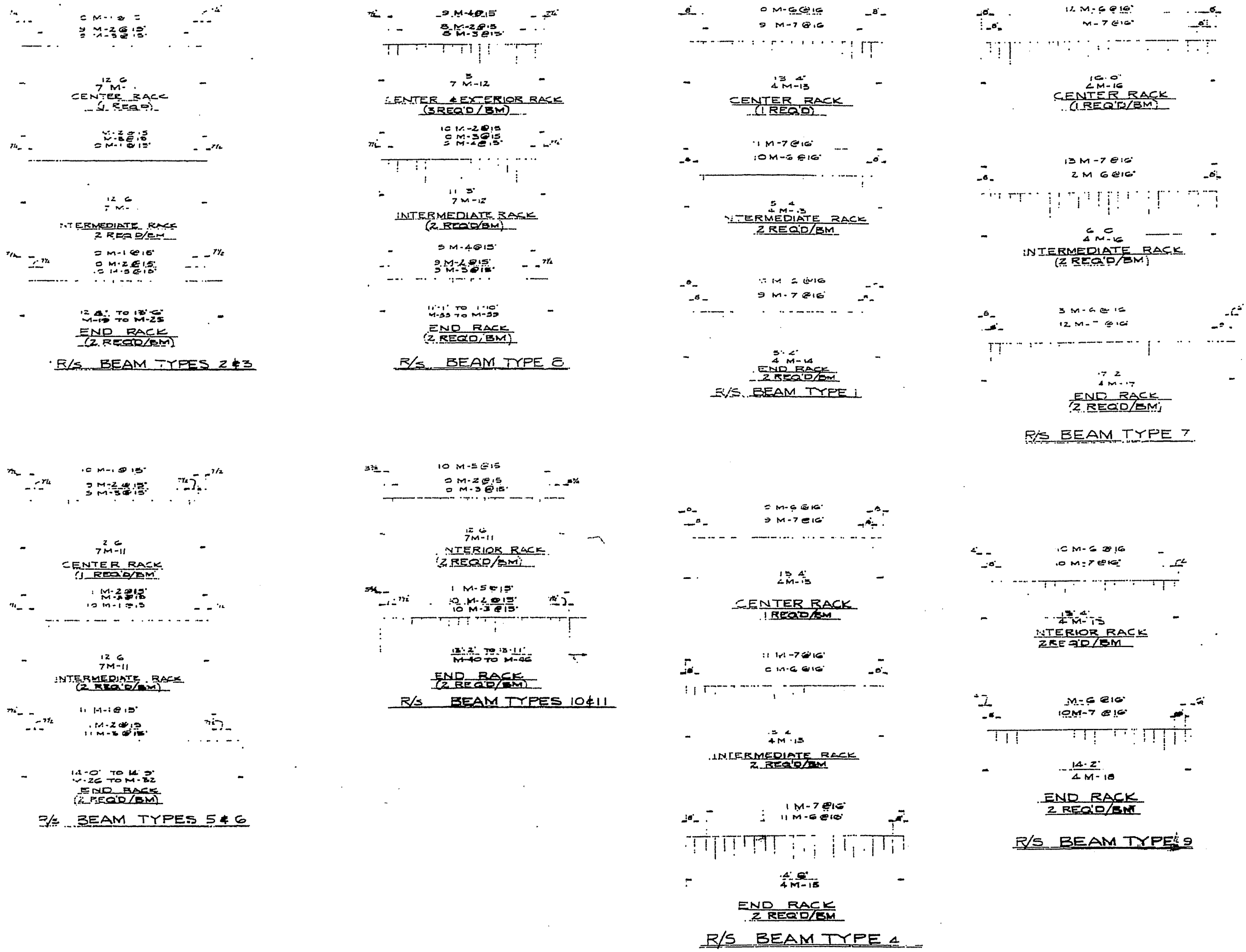
JOB: WASHINGTON COUNTY I-A
LR. 798 UNDER APPL.92 (MAIN ST
STA 993 + 4273

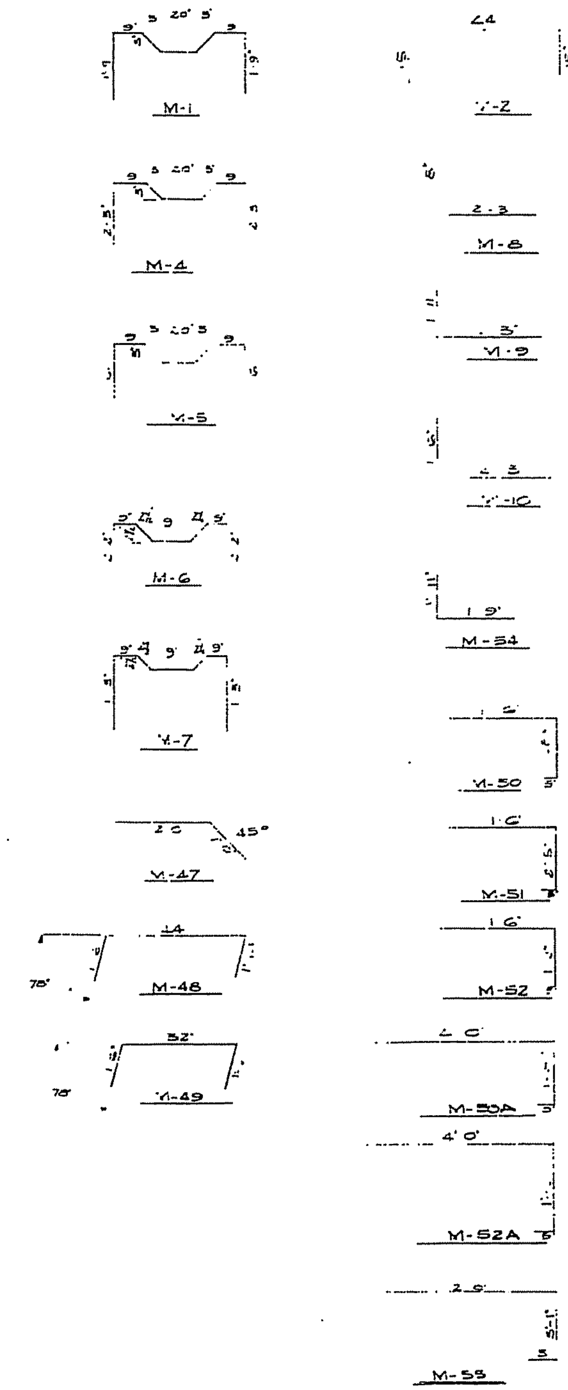
FOR: MARSOLINO CONST. CO.
PA. DEPT. OF HIGHWAYS

REVISIONS

DATE	MADE BY	CHECKED BY	REMARKS
2-27-61	C.G.B.	C.G.B.	PREL. APPROVAL 2-20-61

DWG. NO. P-3530 | REF. DWG. NO. S-3530A | JOB NO. 6017
DRAWN BY C.G.B. | DATE 1-12-61 | SHEET 3 OF 4
CHECKED BY C.G.B. | DATE 2-2-61





BAR LIST				
MARK	REQD	SIZE	LENGTH	DESCRIPTION
M-1	318	5	7 5'	BENT
M-2	1654	5	2 2	BENT
M-3	1654	5	3-9	STRAIGHT
M-4	567	5	6 5	BENT
M-5	578	5	2-11'	
M-6	412	4	7-2	
M-7	406	4	3 5'	
M-8	1296	4	3-..	
M-9	900	4	4 2	
M-10	260	4	3-9'	BENT
M-11	504	4	2 6'	STRAIGHT
M-12	315	4	3'	
M-13	64	4	13 4	
M-14	6	4	3 2	
M-15	16	4	4 6'	
M-16	24	4	6 0'	
M-17	16	4	7 2'	
M-18	16	4	14 2	
M-19	8	4	12 6	
M-20	18	4	2 10	
M-21	18	4	13' 0'	
M-22	18	4	3'	
M-23	18	4	3 5	
M-24	18	4	3 4'	
M-25	18	4	5 6'	
M-26	8	4	14 0'	
M-27	18	4	4 2'	
M-28	18	4	4-3	
M-29	18	4	4 5	
M-30	18	4	14 7'	
M-31	18	4	14 8	
M-32	18	4	14-9'	
M-33	8	4	11	
M-34	18	4	-2'	
M-35	18	4	11-4'	
M-36	18	4	11 6'	
M-37	18	4	-8	
M-38	18	4	9	
M-39	18	4	11 10	
M-40	18	4	13' 2	
M-41	18	4	13 3'	
M-42	18	4	13 4'	
M-43	18	4	13'-"	
M-44	8	4	13 9'	
M-45	18	4	3-10'	
M-46	18	4	5 11	STRAIGHT
M-47	568	5	4 0	BENT
M-48	216	5	8'	
M-49	54	5	5 8'	
M-50	238	5	2-7"	
M-51	126	5		
M-52	119	5		
M-50A	14	5		
M-52A	7	5	5	BENT
M-53	2305	4	2 0'	STRAIGHT
M-54	292	4	3 8'	BENT
M-55	64	5	5 6'	BENT

BMS ID: 62 4049 0030 1265

PENNA. DEPT. OF HWYS.
APPROVED AS TO DESIGN
MAR 6
ENGINEER OF BRIDGES

DICKERSON STRUCTURAL CONCRETE CORPORATION

SPANCRETE

REVISIONS

DATE	MADE BY	CHECKED BY	REMARKS
2-27-61	C.G.B.	CGB.	PREL APPR 2-20-61

JOB: WASHINGTON COUNTY LR 798 UNDER APPL 92 (MAIN ST STA 993 + 42 73

FOR: MARSCLINC CONST. CO. PA DEPT OF HIGHWAYS

DWG. NO.	REF. DWG. NO.	JOB NO.
P-3530	S 3530-A	6017

DRAWN BY	DATE	SHEET
C.G.B.	1-16-61	4 of 4
CHECKED BY	DATE	
CGB	2-2-61	

Appendix B

PennDOT Condition Assessment of Beam Sections

Inspection Methods & Techniques to Determine Non Visible Corrosion of Prestressing Strands in Concrete Bridge Components

Specialty Engineering, Inc. (SEI) inspected the seven (7) prestressed concrete box beam samples at Lehigh University's ATLSS Laboratory on November 19, 2008. The inspection was performed based on visual inspection following the PennDOT publication 100(a). Some possible defects cannot be identified because the sample beams were cut short and were placed individually and upside down. Those defects include: loss of camber, differential deflection between the adjacent beams, and the condition of the transverse tie rods. The following is the summary of our visual inspection findings. The condition ratings are purely based on the observed sample beam conditions. For all interior beam samples, the condition of the sides of the beams was not factored into the condition rating, since this examination is based on their in-situ inspection conditions.

"RT" and "LT" refer to the right and left sides of the beam, respectively.
The right or left sides of a beam are relative to how the beam would be viewed in the field, looking ahead.
Typically, the beam defects listed are for the underside of the beam, except as otherwise noted.

BMS2	DESCRIPTION	RATING	COMMENTS

BEAM ID: Clearfield, Beam 3, Span 1

(1A04) SUPERSTRUCTURE **5**

36x42 Prestressed concrete box beam –

- Longitudinal crack 18" from RT at Near End (4' long, up to 1/32" wide).
- Spall 30" wide, 4" long, 0.75" deep at Far End. (Possible damage from sectioning of beam)
- Longitudinal crack 15" from RT at Far End (6' long, up to 1/16" wide).

Right Side:

- (2) Horizontal cracks at Far End (1'to 2' long, hairline)

Left Side:

- Spall 7" wide, 7" long, 1.5" deep, 6" from beam bottom at Far End

BEAM ID: Clearfield, Beam 4, Span 2

(1A04) SUPERSTRUCTURE **4**

36x42 Prestressed concrete box beam –

- (2) Longitudinal hairline cracks 16" from RT at P01 with light waterstaining (7" and 31" long).
- Longitudinal wide crack 8" from LT (11' long, up to 1/32" wide).
- Spall 3"x2"x1" at P02 end.
- Edge spall 28"x2" at LT end of P02

Right Side:

- Horizontal crack 13" from beam bottom at P01 (42" long, up to 1/16" wide).
- Horizontal crack 4" from beam bottom at P02 (3' long, up to 1/8" wide)

Left Side:

- Diagonal crack 13" from beam bottom at P01 (30" long, up to 1/16" wide)

BEAM ID: Lakeview, Beam 7, Span 1

(1A04) SUPERSTRUCTURE **2** 48x27 Prestressed concrete box beam –

- Full-length delaminated concrete with exposed strands
- Longitudinal wide crack 20” from RT (full length, up to 1/8”wide)
- Spall (6 SF) RT of wide crack at P01 (3 broken and missing strands for 10’ long and 2nd layer strands at RT exposed)
- Spall (6 SF) LT of wide crack at P01 (13 broken strands and missing for 2’ long and 2nd layer strands at LT exposed)
- Exposed and corroded stirrup at P01

BEAM ID: Lakeview, Beam 16, Span 2

(1A04) SUPERSTRUCTURE **4** 48x42 Prestressed concrete box beam –

- Spall (1 SF, 1.25” deep) with one exposed strand at RT P01 (7.5” exposed)
- Longitudinal crack 5” from LT at P01 (3’ long, 1/64” wide)
- Spall (2 SF, 1.75” deep) with 3 exposed strands 3’ from P02 at RT (18” long exposed each)

Right Fascia side:

- No visible defects

Left Side:

- 1 ½” tie rod 4’ from P01

BEAM ID: Lakeview, Beam 19, Span 3

(1A04) SUPERSTRUCTURE **2** 48x42 Prestressed concrete box beam –

- Spall 3” long, 4.5” wide, 0.7” deep, 15” from RT at P02.
- Longitudinal crack 18” from LT at P02 (4’ long, hairline)
- Longitudinal crack 12” from LT at P03 (7’ long, hairline)

Right Side:

- Spall 2”x4.5”x1.5” deep, 6” from beam bottom at P02.

Left Side:

- Horizontal crack 7” from beam bottom at P03 (1’ long)
- Horizontal crack 7” from beam bottom at P02 (32” long)

BEAM ID: Main Street, Beam 2, Span 3

(1A04)	SUPERSTRUCTURE	**2**	48x42 Prestressed concrete box beam – ▪ Full-length spall RT half of Beam with 10 strands exposed almost entire length (7 broken or missing, remaining are exposed with heavy corrosion) ▪ Edge spall (5 SF) with 4 strands exposed (2 broken) 3.3' from P02 @LT **Right Side:** ▪ Horizontal crack 9" from beam bottom @ P02 (30" long) ▪ Horizontal crack 10" from beam bottom @ P03 (2' long)

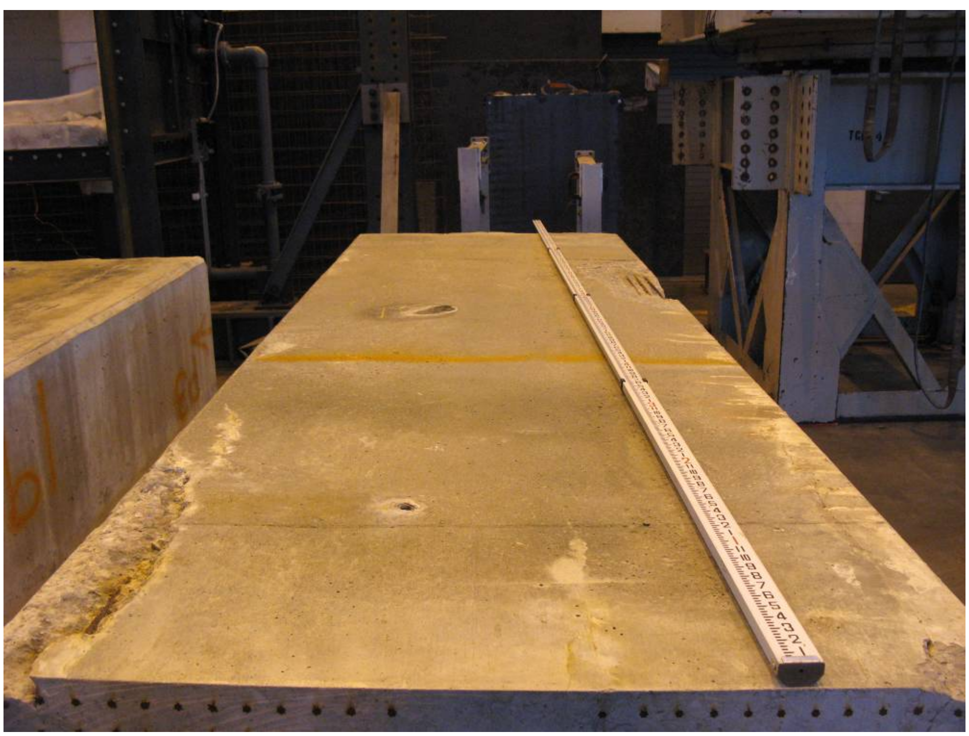
BEAM ID: Main Street, Beam 3, Span 3

(1A04)	SUPERSTRUCTURE	**4**	48x42 Prestressed concrete box beam – ▪ Longitudinal crack 16" from LT @ P02 (full length, up to 1/8" wide) ▪ Spall 29" long, 2" wide, 0.75" deep, and one exposed strand 4' from P02. **Right Side:** ▪ Diagonal crack @ P02. ▪ Horizontal crack 13" from beam bottom @ P03 (2' long) **Left Side:** ▪ Horizontal crack 14" from beam bottom @ P02 (10" long) ▪ Horizontal crack 7" from beam bottom @ P03 (9" long)

Clearfield, Beam 3, Span 1, Looking Ahead

Clearfield, Beam 4, Span 2 , Looking Back

Lakeview, Beam 7, Span 1, Looking Back



Lakeview, Beam 16, Span 2, Looking Ahead

Lakeview, Beam 19, Span 3, Looking Ahead

Main Street, Beam 2, Span 3, Looking Ahead

Main Street, Beam 3, Span 3, Looking Ahead

Appendix C

Recommended Pre-tensioned Concrete Box Beam Rating Procedure

Proposed Rating Recommendations for Prestressed Adjacent Box-Girder Bridges with Longitudinal Cracking

The following guidelines are recommended for the inspection of adjacent prestressed concrete non-composite box-girder bridges. The procedure requires that each beam member be evaluated for the presence of longitudinal cracking, spalled sections, exposed strands, and deteriorated concrete. The damage conditions shall be recorded to scale for each member.

For the purpose of load rating all damage within a region of two development lengths shall be considered to occur at the same section. The computed development length can be used; however, if design information is unavailable the following lengths can be used for typical seven wire strands:

Strand Nominal Diameter [in.]	3/8	7/16	1/2	½ Special
Inspection Window Length [in.]	128	150	170	180

The location of the reduced section strength shall be assumed to occur at the center of the inspection window. The strength reductions shall be based on the presence of longitudinal cracking and deteriorated concrete as noted in the following section.

For Specimens with Longitudinal Cracking

1. The following strand areas shall be reduced to 75% of the original cross-sectional area for capacity calculations:
   a. Strands on each level directly in line the crack.
   b. Strands closest to the exterior surface adjacent to and within 3 in. from the longitudinal crack.
2. For beams with longitudinal cracking or corrosion induced spalling, all other strands in the section shall be reduced to 95% of the original cross-sectional area for capacity calculations.

For Specimens with Deteriorated Concrete

(Adopted from "Guidelines for Estimating Strand Loss in Structural Analysis of PPC Deck Beam Bridges" by the Illinois Department of Transportation)

1. For exposed strands observed with sound concrete adjacent to and above the exposed strands, disregard the full strength of the exposed strands for capacity calculations.

2. For exposed strands observed with adjacent unsound concrete, disregard the full strength of the exposed strands and all strands in regions of unsound concrete for capacity calculations.

3. For exposed shear reinforcement bars, disregard the full strength of strands located in the lower row directly above the exposed section of stirrups for capacity calculations. If the concrete is found to be unsound adjacent to the exposed area, disregard the strength of all strands in all rows above the area of unsound concrete in capacity calculations.

4. For area of concrete where delaminations have been observed, remove all delaminated concrete to determine the depth of the concrete deterioration:

    a. If shear reinforcement bars or strands are exposed, treat as in cases “1” through “3” as shown above.

    b. If no shear reinforcement bars or strands are exposed but there are indications that the exposed concrete is unsound within the affected area, disregard the strength of all strands located in the rows of strands above the area for capacity calculations.

    c. If no steel reinforcement is exposed in the affected area and the concrete is deemed as sound, do not disregard the strength of strands in the strength analysis.

5. For wet or stained areas of concrete observed on the bottom or side of beams, closely inspect those areas to determine the soundness of the concrete:

    a. If close inspection indicates that the concrete is unsound or delaminated, treat as in case “4” above.

    b. If close inspection confirms that the concrete is sound, do not disregard the strength of strands in the strength analysis.

Example Case Study

A prestressed concrete box beam section is illustrated in Figure 1. The damage within a region of one development length is included in the section image. Field inspection of the beam identified three longitudinal cracks, spalling and an area of unsound concrete. The construction documentation indicates that the beam is reinforced with 36 - 3/8 in. diameter seven-wire grade 270 prestressing strands. The spacing and arrangement of the strands is shown in Figure 1.

Using the recommended rating procedure the area reductions and reduced flexural strength is computed. This is conducted in the following stages: 1) the location of cracking, spalling and deteriorated concrete is used to determine a reduced area of prestressing steel, 2) a new center of gravity of steel and corresponding eccentricity is computed, 3) a reduced nominal moment capacity is computed in accordance with ACI 318 recommendations.

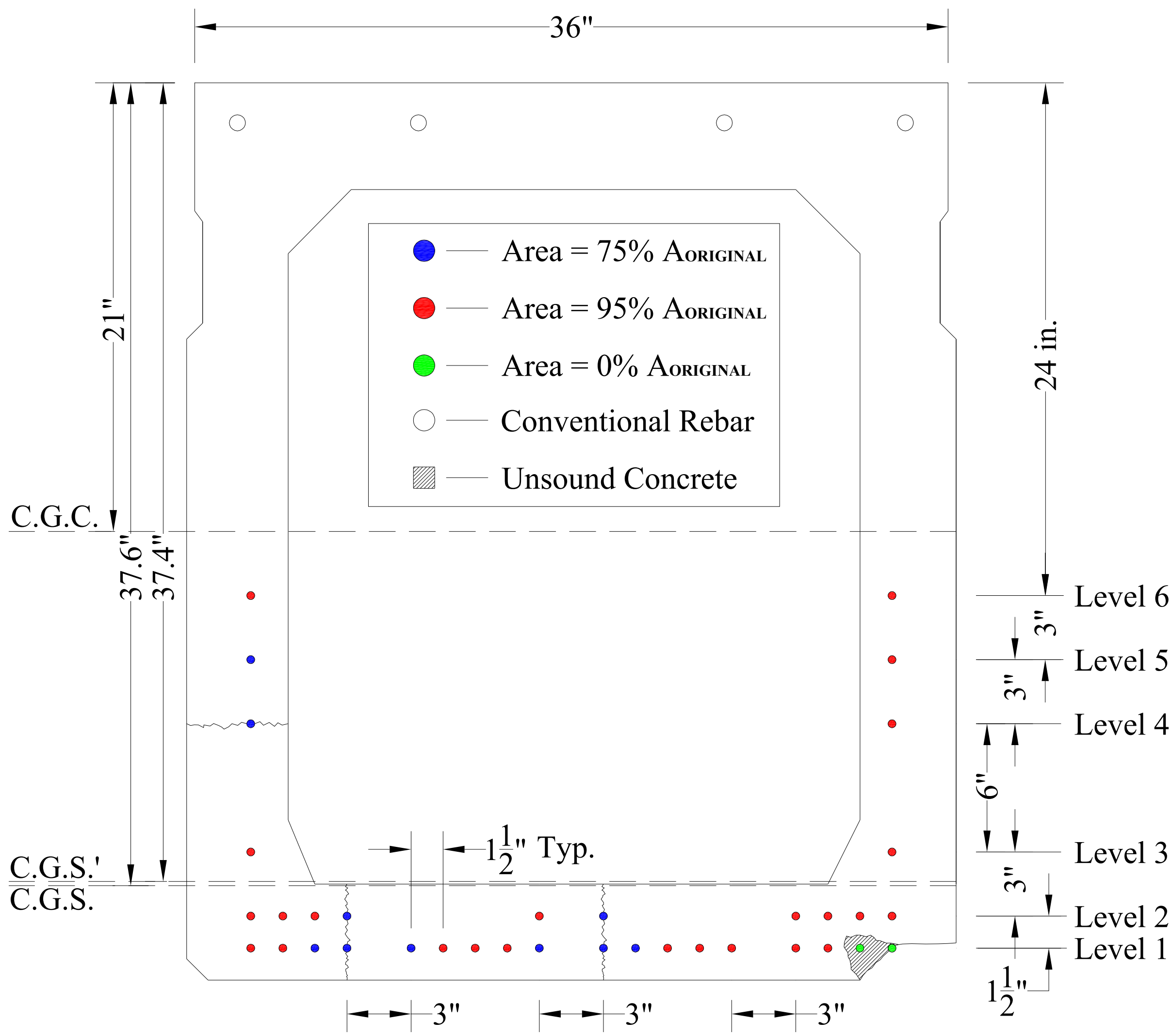


Figure 1: Sample Damaged Section Geometry

Level:	Original Area:	Depth from Top:	Reduced Area for Capacity Calculations:
1	$A_1 = 18 \times 0.085\ in^2 = 1.53\ in^2$	$d_1 = 40.5$ in	$A'_1 = [(10 \times 95\%) + (6 \times 75\%) + (2 \times 0\%)] \times 0.085\ in^2 = 1.19\ in^2$
2	$A_2 = 10 \times 0.085\ in^2 = 0.85\ in^2$	$d_2 = 39$ in	$A'_2 = [(8 \times 95\%) + (2 \times 75\%)] \times 0.085\ in^2 = 0.77\ in^2$

Level:	Original Area:	Depth from Top:	Reduced Area for Capacity Calculations:
3	$A_3 = 2 \times 0.085\ in^2 = 0.17\ in^2$	$d_3 = 36$ in	$A'_3 = (2 \times 95\%) \times 0.085\ in^2 = 0.16\ in^2$
4	$A_4 = 2 \times 0.085\ in^2 = 0.17\ in^2$	$d_4 = 30$ in	$A'_4 = [(1 \times 95\%) + (1 \times 75\%)] \times 0.085\ in^2 = 0.15\ in^2$
5	$A_5 = 2 \times 0.085\ in^2 = 0.17\ in^2$	$d_5 = 27$ in	$A'_5 = [(1 \times 95\%) + (1 \times 75\%)] \times 0.085\ in^2 = 0.15\ in^2$
6	$A_6 = 2 \times 0.085\ in^2 = 0.17\ in^2$	$d_6 = 24$ in	$A'_6 = (2 \times 95\%) \times 0.085\ in^2 = 0.16\ in^2$

The distance from the extreme compression fiber to the center of gravity of steel is computed for the original section, d_p, and the damaged section d'_p as follows.

$$d_p = \frac{\sum A_i \cdot d_i}{\sum A_i} \qquad \text{and} \qquad d'_p = \frac{\sum A'_i \cdot d_i}{\sum A'_i}$$

<u>Calculations:</u>

$$\Sigma A_i = [1.53\ in^2 + 0.85\ in^2 + 0.17\ in^2 + 0.17\ in^2 + 0.17\ in^2 + 0.17\ in^2] = 3.06\ in^2$$

$$\Sigma A'_i = [1.19\ in^2 + 0.77\ in^2 + 0.16\ in^2 + 0.15\ in^2 + 0.15\ in^2 + 0.16\ in^2] = 2.58\ in^2$$

$$d_p = [1.53*40.5 + 0.85*39 + 0.17*36 + 0.17*30 + 0.17*27 + 0.17*24]/(3.06) = 37.6 \text{ in}$$

$$d'_p = [1.19*40.5 + 0.77*39 + 0.16*36 + 0.15*30 + 0.15*27 + 0.16*24]/(2.58) = 37.4 \text{ in}$$

$$e_p = d_p - \text{C.G.C.} = 37.6 \text{ in} - 21 \text{ in} = 16.6 \text{ in}$$

$$e'_p = d'_p - \text{C.G.C.} = 37.4 \text{ in} - 21 \text{ in} = 16.4 \text{ in}$$

$$\text{Ø}M_n = 1995 \text{ ft*k}$$

$$\text{Ø}M'_n = 1715 \text{ ft*k}$$

Appendix D

Ash Street Bridge Rating Evaluation Example

This example is developed for using the PennDOT PS3 computer program and is based on the rating recommendations proposed by Lehigh University. This procedure only applies to flexure ratings. Shear rating is not covered in this example. The information obtained from the Ash Street Bridge over Roaring Brook in Lackawanna County is used in this example. Figure 1 shows the elevation view of the bridge. The bridge consists of twelve (12) prestressed concrete adjacent box beams with no composite deck slab. The bridge has a roadway width of 24'-5", plus a 6'-2" wide sidewalk on either side of the roadway. The bridge span is 66.6' long. The bridge railings consist of iron railings along both fascias. A water main line is carried by the bridge on the left side. A cross section of the bridge is shown in Figure 2.

Figure 1 Left elevation

Appendix-A: PennDOT Standard Drawings-ST207 (selected sheets)

Appendix-B: Current rating method (PennDOT SOL-431-07-08)

Appendix-C: Proposed rating method (Lehigh University)

Appendix-D: PS3 program output

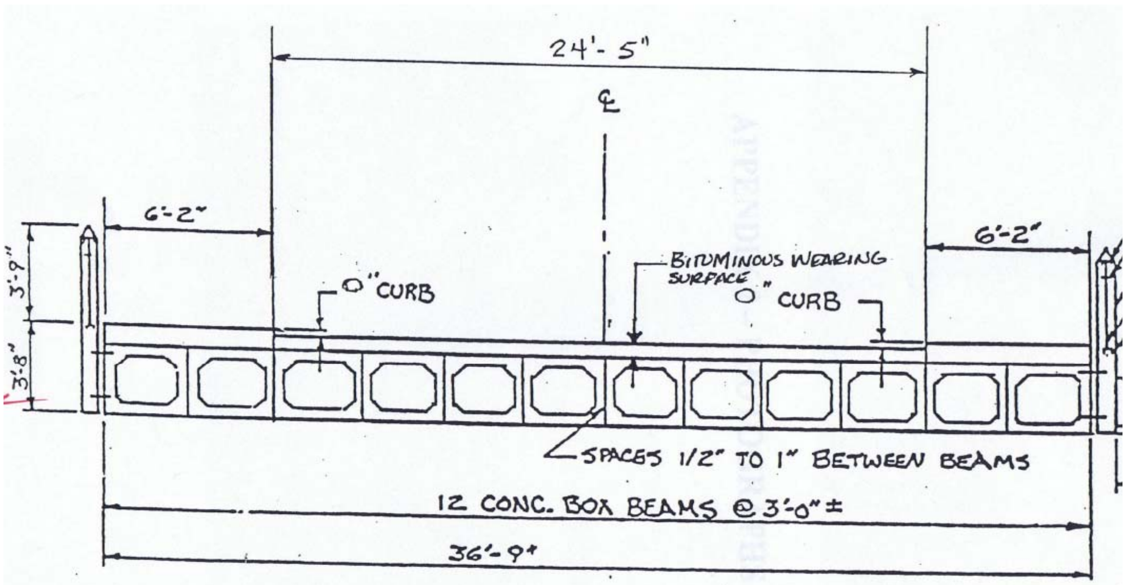
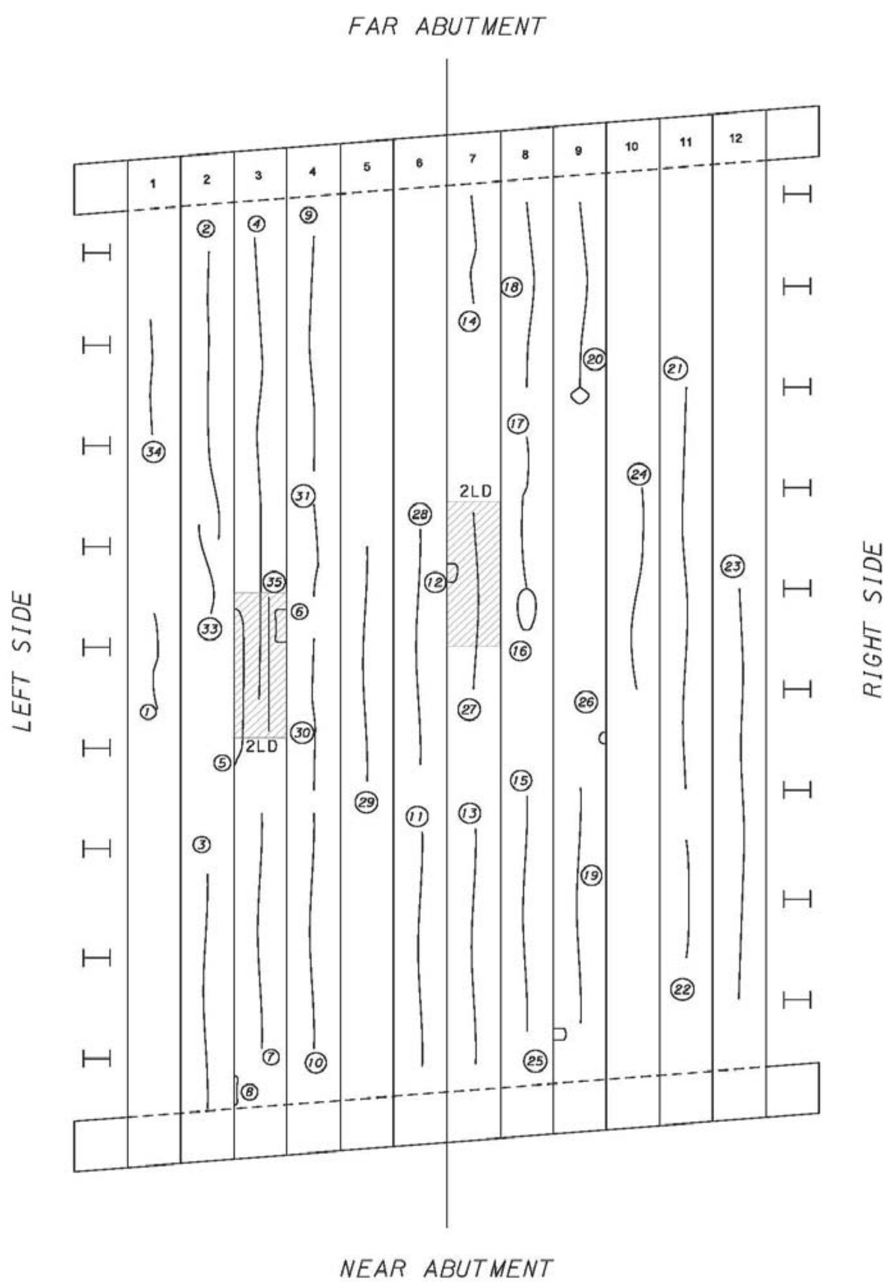


Figure 2 Cross Section

A routine inspection has identified many longitudinal cracks and areas of spalls and delamination with exposed strands, as shown in Figure 3 and Figure 4a.

Figure 3 Underside of Beam 3 at mid-span, Looking back. Exposed strand on Beam 3 and longitudinal cracks.



The shaded areas represent the inspection windows for load ratings. The window length is 150 in for this bridge.

Figure 4a. Field sketch of cracks, spalls, and delamination on bridge.

Beam 1	(34) 10' FINE CRACK @ FAR ABUTMENT
	(1) 4' FINE CRACK @ 0.5 SPAN
Beam 2	(33) 6' FINE CRACK @ 0.5 SPAN
	(3) 10' FINE CRACK @ NEAR ABUTMENT
	(2) 20' FINE CRACK @ FAR ABUTMENT
Beam 3	(8) 3' X 0.8' SPALL @ NEAR ABUTMENT - CORRODED SHEAR REINFORCEMENT
	(7) 10' FINE CRACK @ NEAR ABUTMENT
	(6) 3' X 0.5' AREA OF DELAMINATION @ 0.5 SPAN
	(5) 15' X 0.5' SPALL @ 0.5 SPAN - [1] STRAND EXPOSED & BROKEN
	(4) 13' FINE CRACK @ FAR ABUTMENT
	(35) 3’ FINE CRACK @ 0.5 SPAN
Beam 4	(31) 6' FINE CRACK @ 0.5 SPAN
	(30) 10' FINE CRACK @ 0.5 SPAN
	(10) 8' FINE CRACK @ NEAR ABUTMENT
	(9) 10' FINE CRACK @ FAR ABUTMENT
Beam 5	(29) 15' FINE CRACK @ 0.5 SPAN
Beam 6	(28) 15' FINE CRACK @ 0.5 SPAN
	(11) 8' FINE CRACK @ NEAR ABUTMENT
Beam 7	(27) 12' FINE CRACK @ 0.5 SPAN
	(14) 6' FINE CRACK @ FAR ABUTMENT
	(13) 8' FINE CRACK @ NEAR ABUTMENT
	(12) 2' X 1' AREA OF DELAMINATION @ 0.5 SPAN
Beam 8	(18) 10' FINE CRACK @ FAR ABUTMENT
	(17) 8' FINE CRACK @ 0.5 SPAN PROPOGATING FROM AREA OF DELAMINATION
	(16) 4' X 1' AREA OF DELAMINATION @ 0.5 SPAN
	(15) 8' FINE CRACK @ NEAR ABUTMENT
Beam 9	(26) 1’ X 0.5’ DELAMINATION @ 1/3 SPAN
	(25) 1’ X 1’ DELAMINATION @ 0.1 SPAN
	(20) 15' FINE CRACK @ FAR ABUTMENT
	(19) 7' FINE CRACK @ NEAR ABUTMENT
Beam 10	(24) 17' FINE CRACK @ 0.5 SPAN
Beam 11	(22) 5' FINE CRACK @ NEAR ABUTMENT
	(21) 25' FINE CRACK @ 0.5 SPAN
Beam 12	(23) 30' FINE CRACK @ 0.5 SPAN

Figure 4b cont. Notes on the deteriorations.

MATERIAL PROPERTIES AND BEAM DIMENSIONS FOR PS3 INPUT

PennDOT BMS2 indicates that the standard drawing ST-207 was used to design this bridge. No shop drawings were available for this bridge and therefore the strand pattern in the beams are unknown. With field measurements, it was determined that the box beam is a standard 36"x36" size. The beam properties are shown as follows:

P_i =		kips (Initial Prestressing Force)
f'_{cb} =		ksi (28 day Concrete Compressive Strength - Beams)
f'_{cs} =	N/A	ksi (28 day Concrete Compressive Strength - Slab)
f'_{ci} =		ksi (Initial Concrete Compressive Strength)
f_{si} =	175.0	ksi (Initial Tensile Stress of Prestressing Steel)
f'_s =	250.0	ksi (Ultimate Tensile Strength of Prestressing Steel)
f_{ci} =	0	ksi (Allowable Concrete Compressive Strength - Before Losses)
f_{ti} =	0.000	ksi (Allowable Tension in Top Fiber Concrete - Before Losses)
f_c =	0.0	ksi (Allowable Concrete Compressive Strength - After Losses)
f_t =	0.000	ksi (Allowable Tension in Concrete in Precompressed Tensile Zone - After Losses)

Strand Area = 0.109 in^2

PennDOT ST-207 defines the area of 7/16" strands as 0.109 in^2. PCI Design Handbook, however, defines the area as 0.108 in^2.

D =	36	B3 =	3.00
W1 =	36.00	B4 =	3.00
W2 =	36.00	D1 =	12.00
W3 =	5.00	D2 =	5.75
T1 =	5.00	X1 =	1.50
T2 =	5.00	X2 =	3.00
B1 =	3.00	Slab Thick =	0.00
B2 =	3.00	Haunch =	0.00

STRAND DETAILS FOR PS3 INPUT

From the PennDOT standard drawing ST-207, the beams have a strand CG of 2.36 in from the bottom of the beams. Based on this value a strand pattern is assumed which will result in a C.G. location close to that in the standard drawing. If the shop drawings are available, the exact strand pattern should be used in the following rating procedure.

ROW #	d_i (in)	A_i (in^2)	$A_i * d_i$
Row 2	3.5	1.308	4.578
Row 1	1.5	1.744	2.616
Total Strand Area		3.052	7.194

	Design Standard	Equivalent
CG	2.360	2.357

d_i : C.G of each row of strands from bottom of the beam.

A_i: Total area of strands in each row (A=0.109 in^2 for each strands).

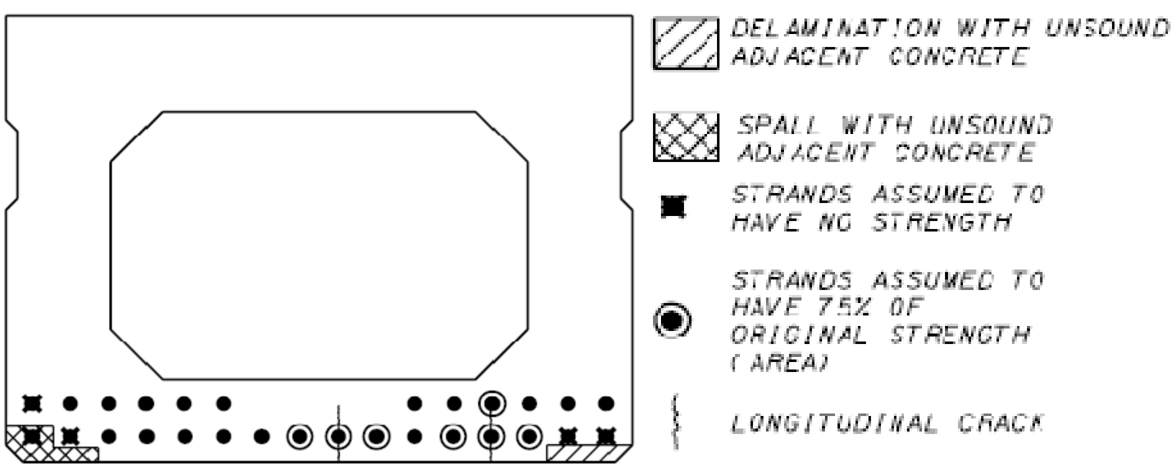
THE PROPOSED METHOD

DEVELOPMENT LENGTH

Knowing the diameter of the strands (7/16") and the initial and ultimate tensile stress in the strands (175 ksi and 250 ksi respectively), the inspection window length is determined to be 150 in. The windows are shown as shaded areas in Figure 4a for Beams 3 and 7. The deterioration within this length is evaluated for all the beams at all locations to determine the location and reduction of strands for the rating.

REMAINING STRAND AREA

Through the inspection of the field notes, Beam 3 appears to have the most deterioration within the inspection window lenth at mid-span, where the beam has the greatest moment and will likely govern the rating. The bottom layer of strands that are at crack locations or within 3" from any crack is to be reduced to 75% of their original area. The strands at unsound concrete or are exposed are to be entirely discounted. All other strands will be reduced to 95% of their original area. The effective remaining strand area is then computed.

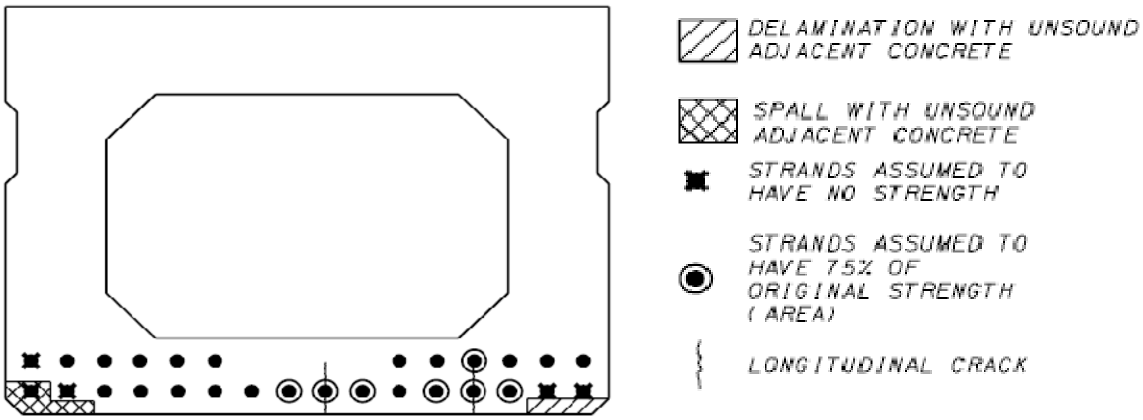


BEAM 3 CROSS SECTION

Beam 3 Total Area = (0.95*16+7*.75)*.109 = **2.229** in^2

EQUIVALENT C.G. AND AREA OF STRAND PATTERN WITH SECTION REDUCTION (BEAM 3)

(for PS3 input)
Since the PS3 program does not accept a strand pattern with various individual strand area, an equivalent C.G. and area for the strands with section reduction need to be determined for PS3 input.



BEAM 3 CROSS SECTION

Row 1 remaining area = (0.95*10 + 0.75*1)*0.109 = 1.117 (in^2)

Row 2 remaining area = (0.95*6 + 0.75*6)*0.109 = 1.112 (in^2)

ROW #	d*i* (in)	A*i* (in^2)	# of Remaining strands	A*i* * d*i*
Row 2	3.5	1.117	10	3.91
Row 1	1.5	1.112	10	1.668
Total		2.229	20	5.578

Reduced CG 2.502

STRAND DETAILS FOR PS3 INPUT (PROPOSED METHOD)

Area = 0.109 in^2

G1 = 1.50 in (distance from bottom of beam to centroid of bottom row of strands)

G2 = 2.502 in (CG)

Number of Strands in each Row:

R1	20	R6	
R2		R7	
R3		R8	
R4		R9	
R5		R10	

STIRRUP DETAILS

Area = 0.20 in^2

fsy = 40 ksi

PS3 Input:

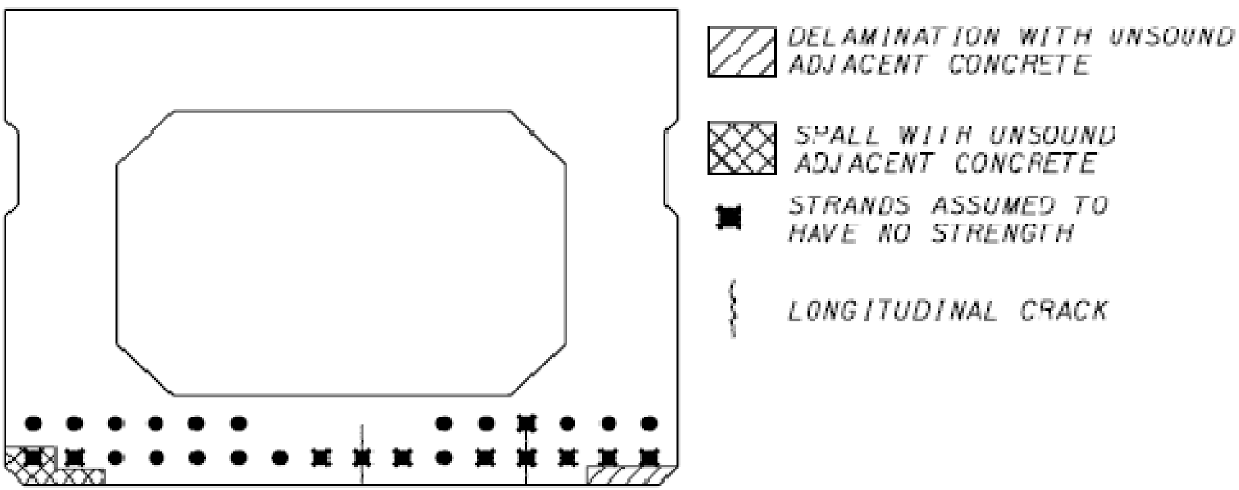
# Stirrups	@	Spacing		Location	Spacing
4	@	6	in	0.00	6.000
2	@	12	in	2.00	12.000
4	@	24	in	4.00	24.000

Note: When CG is input into the G2 command, the total number of strands shall be placed in the R1 command.

THE CURRENT METHOD

The “current method” is based on the current PennDOT rating procedures outlined in Publication 238, Part IP, 6.6.3.3.1I. This method is based on deterioration at each cross section, in contrast to the deterioration within a window length used in the proposed method.

Strands located adjacent to cracks are assumed to have a 100% reduced area. The current method does not have any clear guidelines for spalls or delaminations. The strands located above a spall or delamination are assumed ineffective and are entirely discounted. For an exposed strand it is assumed that the strand's cross sectional area is reduced by 125%. For longitudinal cracks, the two strands directly above the crack and the two adjacent strands on the first row are assumed to be ineffective.



BEAM 3 CROSS SECTION

Beam 3 Total Area = (17)*0.109 -0.25*.109*4= **1.744** in^2

SPECIALTY ENGINEERING, INC.
Consulting and Research Engineers

PROJECT	PRESTRESSED EXAMPLE		
SHEET NO.	12	OF	16
CALCULATED BY	TRK	DATE	5/5/2010
CHECKED BY	TJ	DATE	5/21/2010

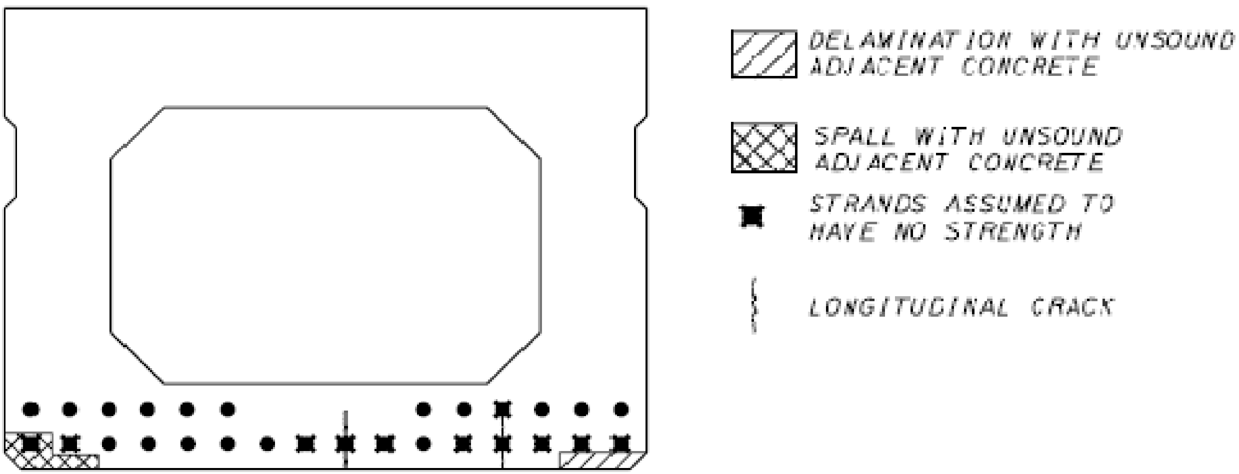
EQUIVALENT STRAND PATTERN AND AREA WITH SECTION REDUCTION (current method)

Find an equivalent C.G. and area of the strands with section reduction to be input into PS3.

ROW #	d_i (in)	A_i (in^2)	# of Remaining strands	A_i * d_i
Row 2	3.5	1.199	11	4.1965
Row 1	1.5	0.545	5	0.8175
Total		1.744	16	5.014

Equivalent Strand Area = 0.109 in^2

C.G. = 2.875 in



BEAM 3 CROSS SECTION

STRAND DETAILS FOR PS3 INPUT (CURRENT METHOD)

Area = 0.109 in^2

G1 = 1.50 in (distance from bottom of beam to centroid of bottom row of strands)

G2 = 2.875 in (vertical distance between each row of strands)

Number of Strands in each Row:

R1	16	R6	
R2		R7	
R3		R8	
R4		R9	
R5		R10	

STIRRUP DETAILS

Area = 0.20 in^2

fsy = 40 ksi

PS3 Input:

# Stirrups	@	Spacing		Location	Spacing
4	@	6	in	0.00	6.000
2	@	12	in	2.00	12.000
4	@	24	in	4.00	24.000

DISTRIBUTION FACTORS: BASED ON AASHTO (A3.23)

Width of Beam(b) = 3.0 ft

No. Lanes (N_L) = 3

No. Beams (N_g) = 12

For Concrete Box Beams used in multi-beam decks:

		Range of Applicability:	
θ =	20	0° < θ ≤ 60°	OK
Width of Beam (b) =	3 feet	3.5' ≤ b ≤ 6'	S.A.F. Not Applicable
Span Length (L) =	66.60 feet	20' ≤ L ≤ 120'	OK
No. Beams (Nb) =	12	5 ≤ Nb ≤ 20	OK
Depth of Beam (d) =	36 in	17" ≤ d ≤ 60"	OK

S.A.F = 1.0+ (12xL/90d)x√(tanθ) = N/A

Shear: (A3.23.1)

$$D.F. = 0.5 \times \left(1 + \frac{0}{3}\right) = \underline{0.500}\ \text{AXLES}$$

Moment: (A3.23.4.3 - 13th Edition, per D3.23.4.3)

$S = (12N_L+9)/N_g$ = **3.75** equation (3-12)

W = 36.75 ft (overall width of bridge)

L = 66.60 ft (span Length)

K = 1.0 for Box Beam

C = K(W/L) = **0.552** **C <= 3 use equation (3-13)** D = **6.73**

D.F. = 0.5 x (S/D) = 0.279 AXLES

Deflection:

$$\frac{\text{No. Lanes}}{\text{No. Beams}} = \left(\frac{3}{12}\right) = \underline{0.250}\ \text{AXLES}$$

DEAD LOADS

No. Lanes = 3

No. Beams = 12

UDLF

For an interior non composite adjacent box beam, all loads are included into the UDLF command. The only loads to be included are wearing surfaces and utilities. In this case there is a wearing surface but no utilities.

Bituminous: Bituminous weight = 0.150 kcf

Thick. = 8 "

total weight of Bituminous = 0.1000 ksf (of Beam)

Total = **0.1000 ksf** **UDLF**

RATING COMPARISON:

(Between the proposed and current methods)

The tabulated comparisons are for flexure ratings only. Both methods do not consider the effects of prestress losses in the strands that may be affected by the section loss assumed in the strands. The resulting inventory ratings are computed based on the service state and the operating ratings are based on the ultimate strength of the beam.

VEHICLES	INVENTORY RATINGS (TONS)		Percent Difference
	Current Method Beam 3	Proposed Method Beam 3	
H20	2.95	14.38	387.46
HS20	3.78	18.43	387.57
ML80	3.28	15.98	387.20
TK527	3.70	18.05	387.84

VEHICLES	OPERATING RATINGS (TONS)		Percent Difference
	Current Method Beam 3	proposed Method Beam 3	
H20	24.54	43.17	75.92
HS20	31.47	55.36	75.91
ML80	27.28	48.00	75.95
TK527	30.82	54.22	75.92

Note: A positive value means the proposed method is less conservative than the current method.

The percent difference is the percentage that the new method differs from the current method.

Appendix A- PennDOT Standard Drawings ST-207

GENERAL NOTES

1. ALL MATERIALS AND WORKMANSHIP SHALL BE IN ACCORDANCE WITH P.D.H. FORM 408/60, INCLUDING THE CURRENT SUPPLEMENT FOR SECTION 6.14, AND 408/49.
2. DESIGN SPECIFICATION: "STANDARD SPEC'S FOR HIGHWAY BRIDGES", 1961, OF THE AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS WITH EXCEPTIONS AS NOTED IN DESIGN NOTES OF THESE STANDARDS.
3. LOADINGS: LOADING (LIVE LOAD AND FUTURE DEAD LOADS) AS USED IN DESIGN SHALL BE SPECIFIED ON DESIGN DRAWINGS.
4. CLASS "AA" CONCRETE SHALL BE USED IN ROADWAY SLAB, CURBS, SIDEWALKS, CHEEKWALLS DIAPH., PARAPETS, AND WHERE NOTED ON PLANS. ALL OTHER CONCRETE SHALL BE CLASS "A" & "B" UNLESS OTHERWISE NOTED.
5. STEEL REINFORCEMENT BARS DESIGNED FOR 20,000 P.S.I. AND DETAILED AS PER A.C.I. CODE. BARS TO BE LAPPED 30 DIAMETERS. REINFORCING BARS SHALL BE INTERMEDIATE OR HARD GRADE, OR RAIL-STEEL.
6. CHAMFER ALL EXPOSED EDGES 3/4" FOR PRESTRESSED CONCRETE AND 1" FOR ALL OTHER CONCRETE, UNLESS SHOWN OTHERWISE.
7. FOR PARAPET, RAILING AND GUARD CONNECTIONS, SEE P.D.H. STANDARD DRAWING ST 140 OR ST 141.
8. PRETENSIONING: MINIMUM 28-DAY CYLINDER STRENGTH, MINIMUM CYLINDER STRENGTH AT RELEASE OF PRESTRESS SHALL BE SPECIFIED ON DESIGN PLANS AND SHOP DRAWINGS.

DESIGN CRITERIA

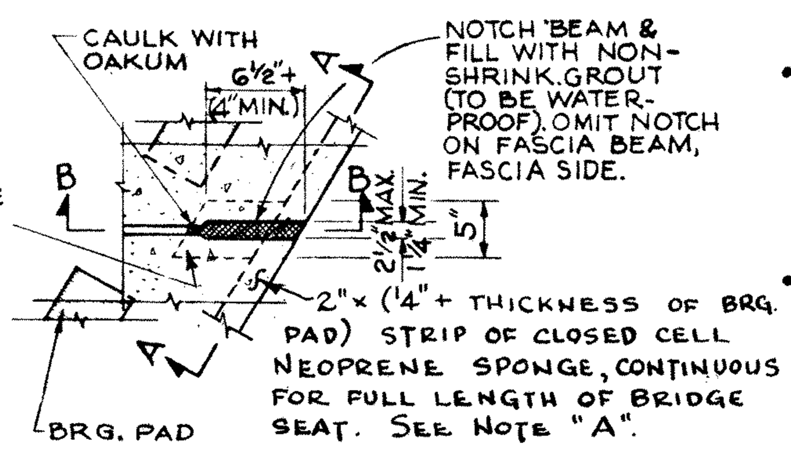
1. PRESTRESSING STRANDS

ASTM GRADE				SPECIAL GRADE			
NOM. DIA. (IN.)	NOM. STEEL AREA OF STRAND (SQ.IN.)	ULTIMATE STRENGTH (LBS)	INITIAL FORCE PER STRAND (LBS)	NOM. DIA. (IN.)	NOM. STEEL AREA OF STRAND (SQ.IN.)	ULTIMATE STRENGTH (LBS)	INITIAL FORCE PER STRAND (LBS)
1/4	0.036	9,000	6,300	1/4	VALUES CURRENTLY IN USE MAY INCREASE OR DECREASE ACCORDING TO MANUFACTURERS' APPROVED GUARANTEED ULTIMATE STRENGTH.		
3/8	0.080	20,000	14,000	3/8			
7/16	0.109	27,000	18,900	7/16			
1/2	0.144	36,000	[illegible]	1/2			

- ALL DESIGNS SHALL BE BASED ON THE USE OF ASTM GRADE STRANDS.
- SPECIAL GRADE STRANDS MAY BE SHOWN ON THE SHOP DRAWINGS AND IF APPROVED BY THE BRIDGE ENGINEER, MAY BE USED INSTEAD OF ASTM GRADE STRANDS.
- 1/2" DIA. STRANDS OF EITHER GRADE MAY BE USED ONLY WITH SPECIAL APPROVAL OF THE BRIDGE ENGINEER.

2. TEMPORARY ALLOWABLE STRESSES BEFORE CREEP AND SHRINKAGE:
   COMPRESSIVE STRESSES IN CONCRETE, PRETENSIONED: $0.60\ f'_{ci}$ (MAX.)
   TENSION IN CONCRETE (UNREINFORCED) : $0.00\ f'_{ci}$
   WHEN THE COMPUTATIONS SHOW TENSION IN THE COMPRESSION FLANGE (I.E. TOP FLANGE OF SIMPLE SPANS), UNPRESTRESSED REINFORCEMENT SHALL BE USED AND DESIGNED TO TAKE THE TOT. TENSILE STRESS. COMPUTED TENSION STRESSES IN THE CONCRETE SHALL NOT EXCEED $0.12\ f'_{ci}$ FOR TEMPORARY STRESSES, $0.096\ f'_{ci}$ FOR DESIGN STRESSES.
   PRESTRESSING STEEL WIRE OR STRAND : $0.70\ f'_s$ (MAX.)
3. ALLOWABLE STRESSES UNDER DESIGN LOADS:
   COMPRESSION IN CONCRETE : $0.48\ f'_{ci}$ (MAX)
   TENSION IN THE PRECOMPRESSED TENSILE ZONE: 0.00
   PRESTRESSING STEEL WIRE OR STRAND : $0.60\ f'_s$ OR $0.80\ f_y$ (WHICHEVER IS LESS)
4. MINIMUM STRENGTH OF THE CONCRETE AT PRESTRESSING SHALL BE 4500 P.S.I. HIGHER STRENGTH THAN 5000 P.S.I. AT PRESTRESSING MAY BE USED IN DESIGN ONLY WITH SPECIAL APPROVAL OF THE BRIDGE ENGINEER.
5. NEOPRENE BEARING PADS
   "DESIGN OF NEOPRENE BRIDGE BEARING PAD" BY E.I. DU PONT DE NEMOURS & CO. (INC.) EXCEPT AS NOTED
   1- (a) 25% MAX. SHEAR STRAIN.
      (b) MAX CHANGE IN TEMP. FOR CONCRETE = 50°F.
      (c) THERMAL COEFFICIENT OF CONCRETE = 0.000006
      THICKNESS OF PAD; IN. = 0.0144 × (BEAM LENGTH IN FT.)
   2- D.L.+L.L. BEARING PRESSURE 500 PSI MAX.
   INSTRUCTIONS:
   1- CHECK FOR BEARING HARDNESS:
      COMP. STRESS; PSI = (D.L.+L.L.; LBS) ÷ (LENGTH × WIDTH; IN.)
      SHAPE FACTOR = $\frac{\text{LENGTH} \times \text{WIDTH; IN.}}{2(\text{LENGTH}+\text{WIDTH; IN.})(\text{THICKNESS; IN.})}$
      COMP. STRAIN SHALL NOT EXCEED 15%
   2- CHECK SLIPPAGE:
      BEAM TRAVEL PAD CAN ABSORB WITHOUT SLIP. IN.
      $= \frac{(\text{D.L.; LBS}) \times \text{THICKNESS; IN.}}{5 \times \text{LENGTH} \times \text{WIDTH; IN.}} \times 1.8 \div \begin{bmatrix} 160 \text{ IF HARDNESS IS } 60 \\ 215 \text{ IF HARDNESS IS } 70 \end{bmatrix}$
      BEAM TRAVEL THAT WILL OCCUR; IN. = 0.00504 × (BEAM LENGTH; FT.)
      (a) TEMP. RANGE FOR CONCRETE = 70°F.
      (b) LOWEST TEMP. FOR SHEAR MODULUS OF NEOPRENE = 0°F.
      BEAM TRAVEL THAT WILL OCCUR MUST BE LESS THAN THE AMOUNT PAD CAN ABSORB.

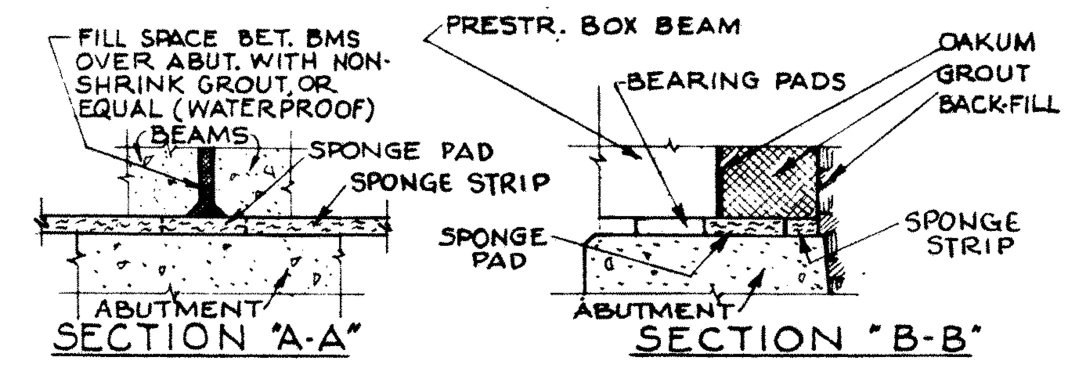
WATERPROOFING AT ABUTMENT



SECTIONAL PLAN

NOTES

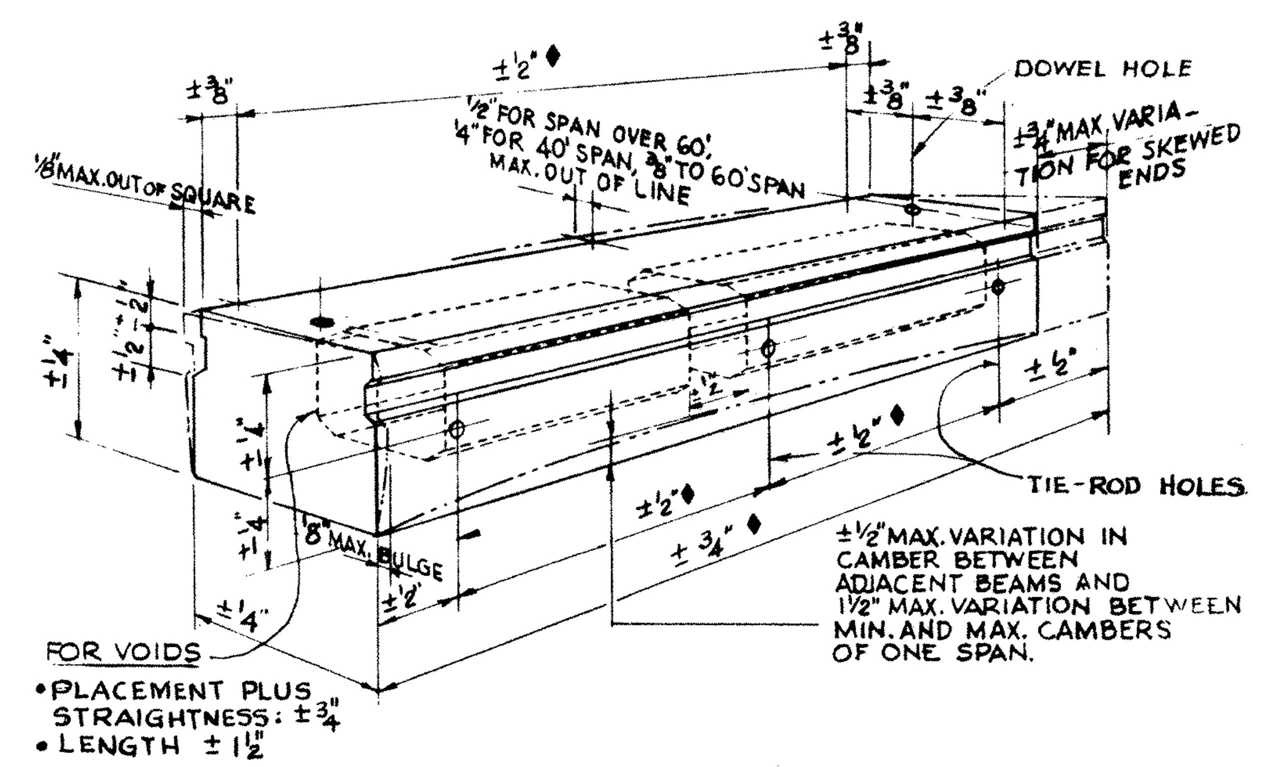
- A DIFFERENT TYPE OF WATERPROOFING MAY BE USED AT NO INCREASE IN COST TO THE DEPARTMENT SUBJECT TO APPROVAL OF BRIDGE ENGINEER.
- THESE DETAILS, WITH INFORMATION SHOWN, ARE TO BE INCLUDED IN THE PRESTR. BOX BEAM DESIGN DRAWINGS.



SECTION "A-A" SECTION "B-B"

BEAM FABRICATION TOLERANCES

(DO NOT SHOW ON DESIGN PLANS)



- PLACEMENT PLUS STRAIGHTNESS: ±3/4"
- LENGTH ±1 1/2"

TOLERANCES SHOWN ARE THE MAXIMUM PERMISSIBLE VARIATIONS FROM THE DIMENSIONS SHOWN ON THE DESIGN OR SHOP DWG'S

♦ LONGITUDINAL TOLERANCES BASED ON DESIGN LENGTH AS SHOWN ON THE CONTRACT PLANS; FOR CASTING LENGTH SEE SHEET 5 OF THESE STANDARDS

INSTRUCTIONS:

1. BEAMS EXCEEDING THE TOLERANCES AS SPECIFIED ON THESE PLANS WILL BE REJECTED FOR SHIPMENT AND ERECTION.
2. SPECIAL WRITTEN APPROVAL FOR ACCEPTANCE MAY BE REQUESTED FROM THE BRIDGE ENG. FOR BMS. EXCEEDING THE TOLERANCES. WITH THIS REQUEST, SUBMIT A SEPARATE SKETCH (SHOWING ALL NECESSARY DIMENSIONS) FOR EACH REJECTED BEAM.
3. DIRECT A "COPY" OF THE REQUEST, WITH THE SKETCH(S) TO THE PENNA. DEPARTMENT OF HIGHWAYS TESTING LABORATORY.
4. SUCH BEAMS MAY BE SHIPPED AND ERECTED ONLY AFTER THE BRIDGE ENGINEER HAS GIVEN THE WRITTEN APPROVAL.

BEARING PADS

INSTRUCTIONS:

BEARING PAD REQUIREMENT	SINGLE AND DOUBLE SPANS	MULTIPLE SPAN (IN EXCESS OF TWO SPANS)
NEOPRENE BEARING PAD	SPANS 60' OR LONGER	SPANS 40' OR LONGER
2-1/4" THICK GRAPHITE IMPREGNATED ASBESTOS BRG. PAD OR NEOPRENE BRG. PAD	SPANS UP TO 60'	SPANS UP TO 40'

GRAPHITE IMPREGNATED ASBESTOS BEARING PADS SHALL BE DESIGNED FOR BEARING PRESSURE OF 800 PSI MAXIMUM AND A WIDTH OF 8" MAXIMUM.

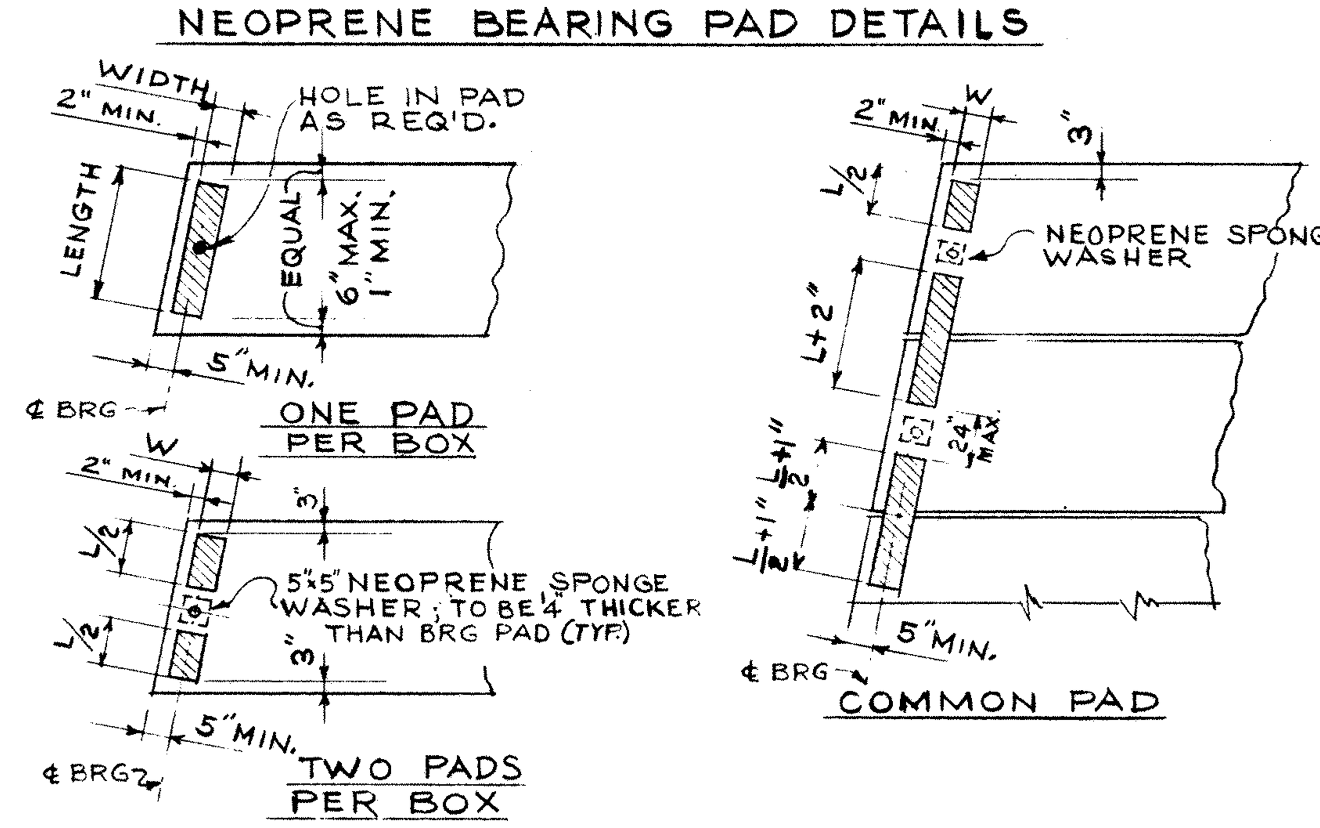
DIMENSIONS SHOWN IN TABLE TO BE USED AS GUIDE ONLY.

DIMENSIONS (INCHES) OF 60 DUROMETER NEOPRENE BEARING PADS – SEE NOTE "B"

SPAN RANGE FEET	17" BOX			21" BOX			27" BOX			33" BOX			36" BOX			42" BOX		
	W	T	L	W	T	L	W	T	L	W	T	L	W	T	L	W	T	L
30-40	5	3/4	24	5	3/4	24	5	5/8	24	5	5/8	24	5	5/8	24	5	1/2	24
41-50	5	3/4	24	5	3/4	24	5	5/8	24	5	5/8	24	5	5/8	24	5	5/8	24
51-60				5	3/4	24	5	3/4	24	5	3/4	24	5	3/4	24	5	3/4	24
61-70							5	3/4	24	5	3/4	24	5	3/4	24	5	3/4	24
71-80							5	1	24	5	1	24	5	1	24	5	1	24
81-90										5	1	25	5	1	25	5	1	26
91-100																6	1 1/4	26

W = WIDTH T = THK. L = LENGTH
MIN. W = 5" OR 5T (WHICHEVER IS LARGER)

NEOPRENE BEARING PAD DETAILS



DOWEL AND SHEAR BLOCK REQUIREMENTS

FIXED END

CONDITION	1-1 1/4" ϕ DOWEL PER BEAM AND SHEAR BLOCKS OR CHEEKWALLS	1-1 1/4" DOWEL PER BEAM	1-1" ϕ DOWEL PER BEAM	ALTERNATE DOWELS SEE SH. #3
SKEW WITH GRADE UP TO 4%	35° TO 44°	45° TO 59°	60° TO 89°	90°
SKEW WITH GRADE ABOVE 4%	35° TO 59°	60° TO 89°	90°	
SKEW ANY GRADE & SUPERELEV.	35° TO 89°	90°		

ALL DOWEL HOLES TO BE FILLED WITH NON-SHRINK GROUT (SEE SHEET 3)

EXPANSION END

PROVIDE ALTERNATE DOWELS (SEE SH. #3) AND SHEAR BLOCKS FOR SKEWS LESS THAN 89°. NO DOWELS FOR 90° SKEW. PROVIDE SHEAR BLOCKS. ALL DOWEL HOLES SHALL BE FILLED WITH RUBBERISED JOINT MATERIAL.

NOTE "A" – STRIP IS TO BE SHIPPED AND INSTALLED IN ONE CONTINUOUS PIECE. SPLICING SHALL BE MADE IN THE SPONGE FABRICATOR'S SHOP USING AN APPROVED SPONGE ADHESIVE CONTAINING 26 ± 2 % NEOPRENE SOLIDS OR EQUAL.

NOTE "B" – DESIGN COMPUTATIONS FOR BOTH FIXED AND EXPANSION PADS SHALL BE SUBMITTED WITH PLANS FOR FINAL REVIEW.

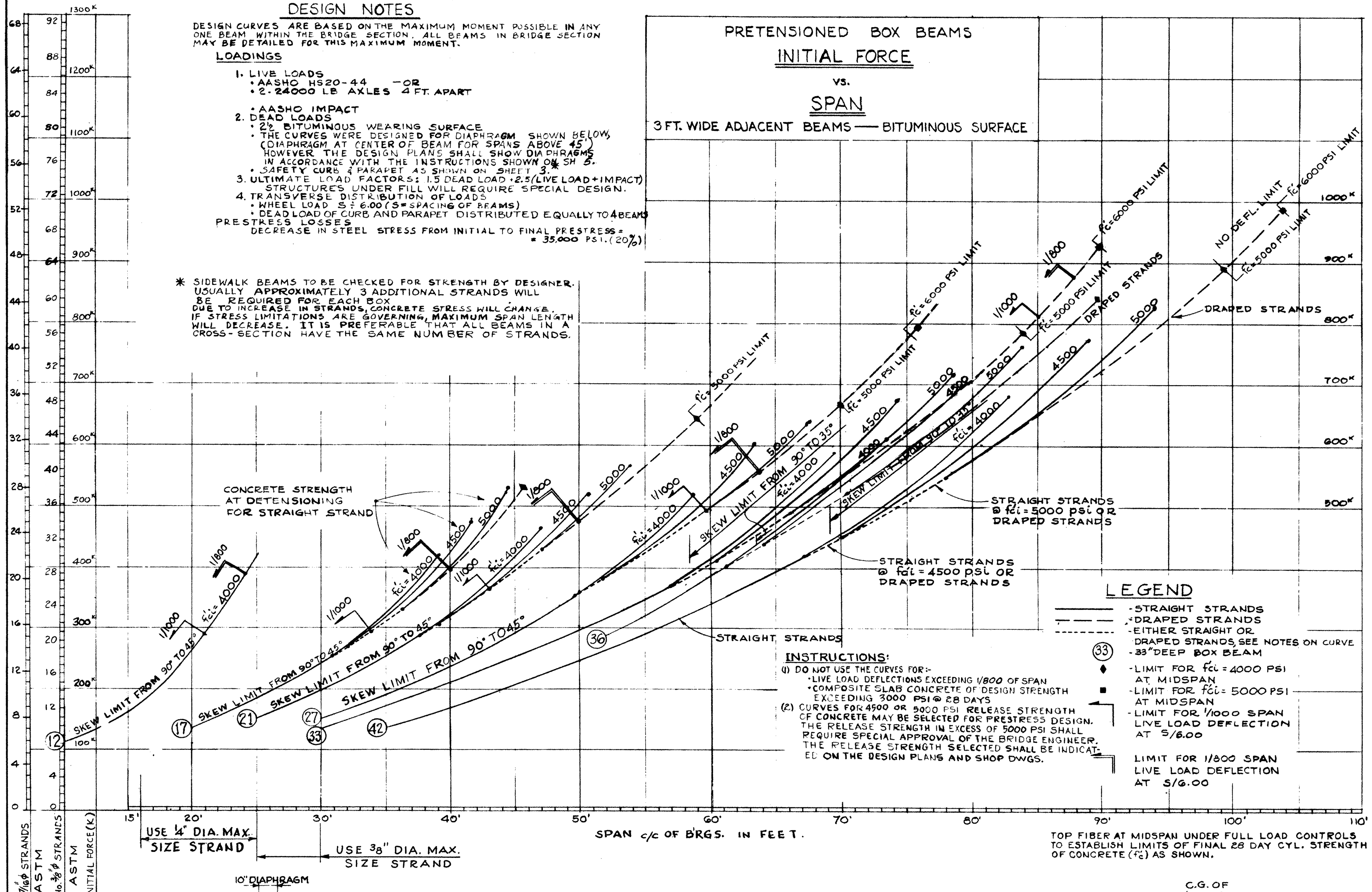
Approved: AUG 17 1964
K.H. Jensen
Engineer

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT
PRESTRESSED CONCRETE BRIDGE STANDARDS
3 FT. ADJACENT BOX BEAMS
BITUMINOUS SURFACE COURSE
GENERAL NOTES AND DETAILS

SCALE: NO SCALE
SHEET 1 OF 7
ST-207

PREPARED BY SCHUPACK AND ZOLLMAN, NEWTOWN SQ., PA., AND THE PENNSYLVANIA DEPARTMENT OF HIGHWAYS – CENTRAL OFFICE – BRIDGE UNIT.

P 45



DESIGN NOTES

DESIGN CURVES ARE BASED ON THE MAXIMUM MOMENT POSSIBLE IN ANY ONE BEAM WITHIN THE BRIDGE SECTION. ALL BEAMS IN BRIDGE SECTION MAY BE DETAILED FOR THIS MAXIMUM MOMENT.

LOADINGS

1. LIVE LOADS
   - AASHO HS20-44 —OR
   - 2-24000 LB AXLES 4 FT. APART
   - AASHO IMPACT
2. DEAD LOADS
   - 2½ BITUMINOUS WEARING SURFACE
   - THE CURVES WERE DESIGNED FOR DIAPHRAGM SHOWN BELOW, (DIAPHRAGM AT CENTER OF BEAM FOR SPANS ABOVE 45') HOWEVER THE DESIGN PLANS SHALL SHOW DIAPHRAGMS IN ACCORDANCE WITH THE INSTRUCTIONS SHOWN ON SH 5.
   - SAFETY CURB & PARAPET AS SHOWN ON SHEET 3.*
3. ULTIMATE LOAD FACTORS: 1.5 DEAD LOAD + 2.5 (LIVE LOAD + IMPACT) STRUCTURES UNDER FILL WILL REQUIRE SPECIAL DESIGN.
4. TRANSVERSE DISTRIBUTION OF LOADS
   - WHEEL LOAD S ÷ 6.00 (S = SPACING OF BEAMS)
   - DEAD LOAD OF CURB AND PARAPET DISTRIBUTED EQUALLY TO 4 BEAMS

PRESTRESS LOSSES
DECREASE IN STEEL STRESS FROM INITIAL TO FINAL PRESTRESS = 35,000 PSI. (20%)

* SIDEWALK BEAMS TO BE CHECKED FOR STRENGTH BY DESIGNER. USUALLY APPROXIMATELY 3 ADDITIONAL STRANDS WILL BE REQUIRED FOR EACH BOX. DUE TO INCREASE IN STRANDS, CONCRETE STRESS WILL CHANGE. IF STRESS LIMITATIONS ARE GOVERNING, MAXIMUM SPAN LENGTH WILL DECREASE. IT IS PREFERABLE THAT ALL BEAMS IN A CROSS-SECTION HAVE THE SAME NUMBER OF STRANDS.

INSTRUCTIONS:

(1) DO NOT USE THE CURVES FOR:-
- LIVE LOAD DEFLECTIONS EXCEEDING 1/800 OF SPAN
- COMPOSITE SLAB CONCRETE OF DESIGN STRENGTH EXCEEDING 3000 PSI @ 28 DAYS

(2) CURVES FOR 4500 OR 5000 PSI RELEASE STRENGTH OF CONCRETE MAY BE SELECTED FOR PRESTRESS DESIGN. THE RELEASE STRENGTH IN EXCESS OF 5000 PSI SHALL REQUIRE SPECIAL APPROVAL OF THE BRIDGE ENGINEER. THE RELEASE STRENGTH SELECTED SHALL BE INDICATED ON THE DESIGN PLANS AND SHOP DWGS.

LEGEND

- STRAIGHT STRANDS
- DRAPED STRANDS
- EITHER STRAIGHT OR DRAPED STRANDS, SEE NOTES ON CURVE
- (33) 33" DEEP BOX BEAM
- ◆ LIMIT FOR f'_{ci} = 4000 PSI AT MIDSPAN
- ■ LIMIT FOR f'_{ci} = 5000 PSI AT MIDSPAN
- LIMIT FOR 1/1000 SPAN LIVE LOAD DEFLECTION AT S/6.00
- LIMIT FOR 1/800 SPAN LIVE LOAD DEFLECTION AT S/6.00

TOP FIBER AT MIDSPAN UNDER FULL LOAD CONTROLS TO ESTABLISH LIMITS OF FINAL 28 DAY CYL. STRENGTH OF CONCRETE (f'_c) AS SHOWN.

C.G. OF STRANDS TO BOTTOM OF BEAM-INCHES "g"

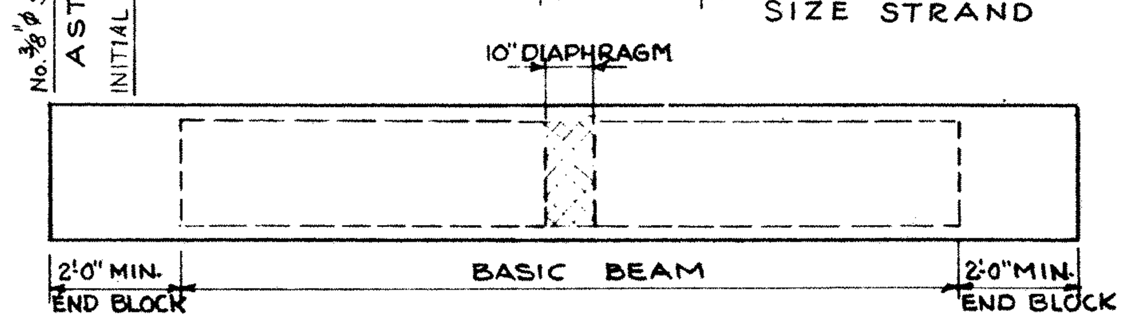
FOR 7/16" Ø STRANDS

INITIAL FORCE (KIPS)

BOX	f'_{ci}	151	189	227	265	302	340	378	416	454	491	529	567	605	643	680	718	756	794	832	870	907	945	983	1021
12 x 36 STRAIGHT STR'DS	4000	1.50	1.88	2.26	2.64	2.77	2.89	3.02	3.14																
	4500																								
	5000																								
	DRAPED																								
17 x 36 STRAIGHT STR'DS	4000	1.50	1.50	1.50	1.70	2.05	2.34	2.57	2.77																
	4500				1.58	1.85	2.26	2.49	2.68	2.84															
	5000				1.50	1.50	1.77	2.06	2.30	2.48	2.65	2.96													
	DRAPED				1.50	1.50	1.70	1.90	2.03	2.16	2.26	2.36													
21 x 36 STRAIGHT STR'DS	4000	1.50	1.50	1.50	1.50	1.71	2.13	2.45	2.74	2.94															
	4500					1.56	1.99	2.34	2.61	2.84	3.04														
	5000					1.50	1.70	1.90	2.06	2.33	2.58	2.80	2.91												
	DRAPED					1.50	1.70	1.90	2.03	2.16	2.26	2.36	2.43	2.50	2.66	2.83	3.06								
27 x 36 STRAIGHT STR'DS	4000	1.50	1.50	1.50	1.50	1.50	1.70	2.17	2.56	2.91	3.28														
	4500						1.70	2.07	2.41	2.73	3.03	3.29	3.55												
	5000						1.70	1.90	2.05	2.16	2.33	2.63	2.90	3.18											
	DRAPED						1.70	1.90	2.03	2.16	2.26	2.36	2.43	2.50	2.66	2.83	3.06	3.30	3.58						
33 x 36 STRAIGHT STR'DS	4000	1.50	1.50	1.50	1.50	1.50	1.70	2.09	2.65	3.13	3.51	3.86	4.19												
	4500							1.98	2.55	3.01	3.43	3.79	4.07	4.33	4.59										
	5000							1.90	2.03	2.16	2.34	2.81	3.09	3.46	3.78	3.99	4.21								
	DRAPED							1.90	2.03	2.16	2.26	2.36	2.43	2.50	2.66	2.83	3.06	3.30	3.58	3.86	4.18	4.50			
36 x 36 STRAIGHT STR'DS	4000						1.72	1.90	2.24	2.80	3.28	3.64	3.98	4.27											
	4500								2.04	2.17	2.56	3.01	3.38	3.73	4.01	4.29									
	5000								2.04	2.17	2.27	2.38	2.78	3.16	3.50	3.78	4.04	4.27							
	DRAPED											2.36	2.43	2.50	2.68	2.83	3.08	3.30	3.59	3.86					
42 x 36 STRAIGHT STR'DS	4000	1.50	1.50	1.50	1.50	1.50	1.70	1.90	2.03	2.71	3.31	3.80	4.21	4.60	4.97	5.28									
	4500								2.08	2.41	3.01	3.53	3.95	4.36	4.72	5.01	5.30	5.56							
	5000								2.03	2.16	2.26	2.36	2.68	3.21	3.62	3.98	4.32	4.60	4.88	5.11					
	DRAPED								2.03	2.16	2.26	2.36	2.43	2.50	2.66	2.83	3.06	3.30	3.58	3.86	4.18	4.50	4.85	5.19	

INSTRUCTIONS:

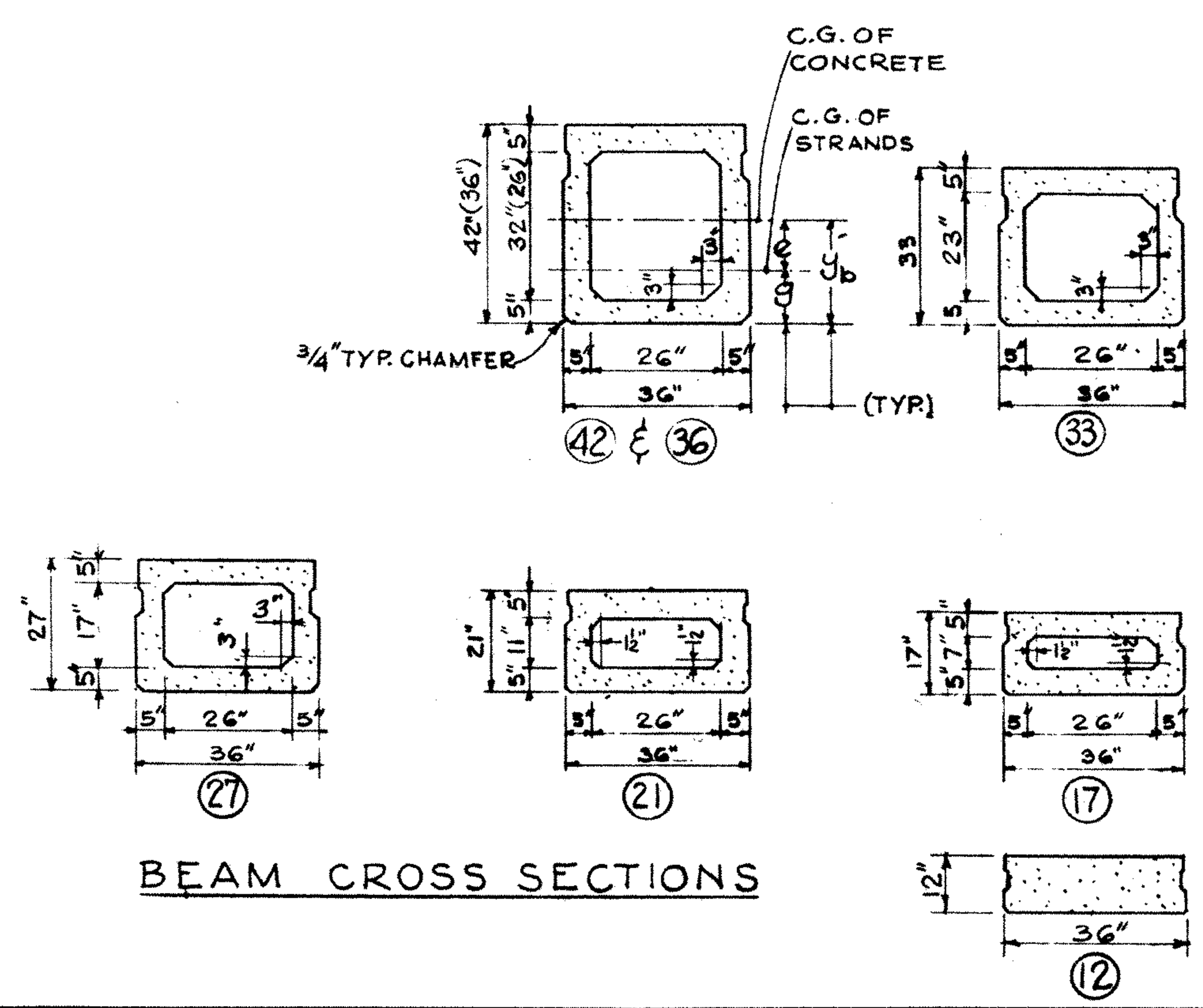
1. INITIAL PRESTRESS FORCE AND C.G. OF STEEL FROM BOTTOM OF BEAM SHALL BE SHOWN ON DESIGN DRAWING. CORRESPONDING NO. OF STRANDS AND STRAND PATTERN SHALL BE SHOWN ON SHOP DRAWING.
2. THE VALUE OF "g" GIVEN IS BASED ON A TYPICAL STRAND PATTERN USING 7/16 Ø STRANDS AS SHOWN ON SHEET 5. FOR STRAND SIZES OTHER THAN 7/16 Ø OR PRE-POST-TENSIONING, "g" AND THE INITIAL FORCE MAY VARY SLIGHTLY TO SATISFY THE PRACTICAL STRAND PATTERN AND THE ALLOWABLE STRESSES. IN ANY CASE, STRESSES SHALL BE CHECKED AT ALL CRITICAL SECTIONS AT VARIOUS STAGES IF A DIFFERENT INITIAL FORCE OR DIFFERENT "g" IS USED.
3. DRAPED STRANDS:-
   - THE GRAPHS FOR DRAPED STRANDS SHOW THE PRESTRESSING FORCE AND ECCENTRICITY AT MIDSPAN. TO BE USED AS GUIDE ONLY.
   - STRESSES AT POINTS OTHER THAN MIDSPAN MUST BE CHECKED FOR TRAJECTORY AND CONCRETE STRENGTH CHOSEN.
   - THE SYMBOLS ◆ AND ■ ON THE CURVES SHOW THE LIMITS OF SPAN FOR VARIOUS CONCRETE STRENGTHS BASED ON STRESSES AT MIDSPAN ONLY.
   - DRAPING OR SIMILAR EFFECTS OF DRAPING MAY BE ACHIEVED BY DEFLECTING STRANDS, BY CHANGING CONCRETE DIMENSIONS AT ENDS, OR BY USING SOME DRAPED, POST-TENSIONED STEEL.
4. CURVES ARE PLOTTED USING MODIFIED INTERSTATE LOADING FOR MOMENTS UP TO 35 FT. SPAN AND FOR SHEAR UP TO 30 FT. SPAN.
5. CURVES ARE DESIGNED CONSIDERING THE ALLOWABLE TENSION LIMITATIONS ON THE BASIC SECTION.
   - TENSION STEEL SHOWN ON SHEET 5 IS NOMINAL STEEL. ADDITIONAL STEEL SHALL BE FURNISHED, IF REQUIRED, AND SHALL BE COMPUTED.



TYPICAL PLAN

QUANTITIES		QUANTITIES AND SECTION PROPERTIES						
ITEM	UNIT	12" BEAM	17" BEAM	21" BEAM	27" BEAM	33" BEAM	36" BEAM	42" BEAM
BASIC BEAM	CY/FT.	0.109	0.110	0.120	0.134	0.153	0.164	0.176
END BLOCKS ▲	CY/BM	–	0.62	0.78	0.99	1.21	1.34	1.54
DIAPHRAGM ▲	CY/BM	–	0.04	0.06	0.09	0.12	0.15	0.18
SECT. PROPERTIES		BASIC BEAMS						
A	IN²	423.75	426.25	466.25	521	594.50	624.50	684.50
y_B	IN	5.97	8.41	10.36	13.33	16.28	17.76	20.73
Z_t	IN³	850	1611	2338	3511	4907	5629	7160
Z_B	IN³	858	1646	2401	3600	5040	5781	7370
I	IN⁴	5,128	13,839	24,874	47,989	82,049	102669	152,542

▲ QUANITIES FOR 90° SKEW BRIDGES ONLY (SEE SKETCH ABOVE). MODIFY FOR BRIDGES WITH OTHER SKEWS.
Z_t SECT. MODULUS AT TOP
Z_B SECT. MODULUS AT BOT.



BEAM CROSS SECTIONS

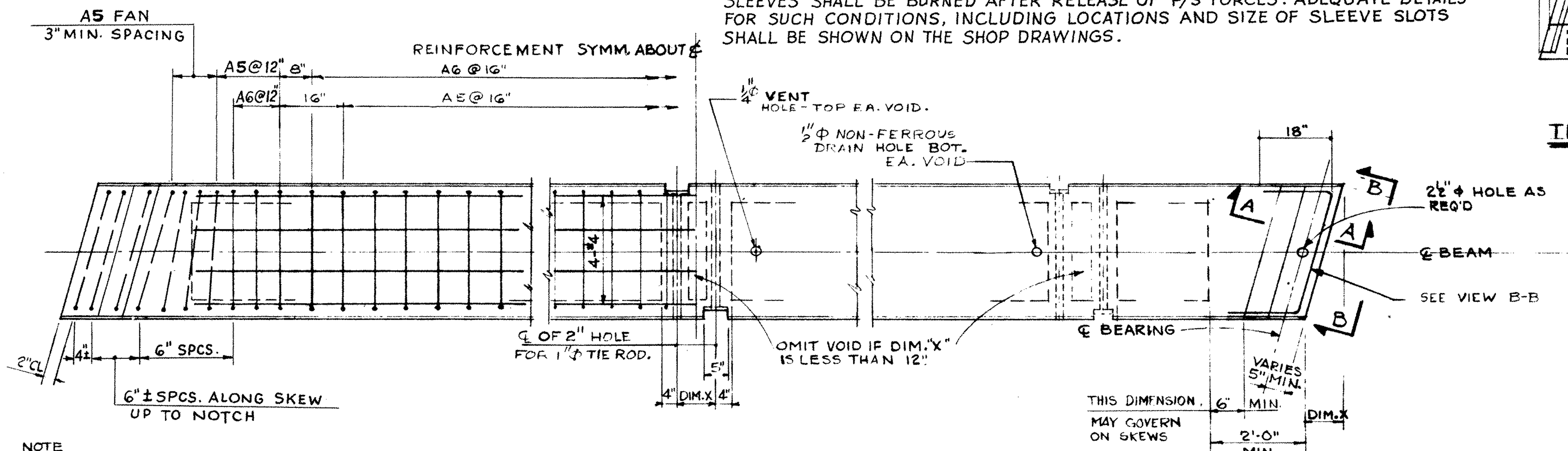
APPROVED AUG 17 1964
K. H. Jensen
BRIDGE ENGINEER

Commonwealth of Pennsylvania
Department of Highways
BRIDGE UNIT
PRESTRESSED CONCRETE BRIDGE STANDARDS
3 FT. ADJACENT BOX BEAMS
BITUMINOUS SURFACE COURSE
DESIGN CURVES AND DATA

SHEET 4 OF 7
ST-207

PREPARED BY SCH... AND ZOLLMAN, ... IN COOPERATION WITH THE PRESTRESSED CONCRETE ASSOCIATION OF PENNSYLVANIA FOR THE ... DEPARTMENT OF HIGHWAYS

DISTRIBUTION OF STRANDS:
STRANDS SHALL BE DISTRIBUTED AS EVENLY AS PRACTICAL. STRAND PATTERNS WHICH WOULD RESULT IN SEVERE FORCE CONCENTRATION SHALL BE AVOIDED. WHEN DRAINAGE SLEEVES ARE REQUIRED IN BEAMS HAVING A DENSE STRAND PATTERN, THE DRAINAGE SLEEVES IF LOCATED NEAR THE END OF THE BEAM MAY BE SLOTTED AS REQUIRED TO PERMIT FREE PASSAGE OF STRANDS. STRANDS EXTENDING THROUGH SLEEVES SHALL BE BURNED AFTER RELEASE OF P/S FORCES. ADEQUATE DETAILS FOR SUCH CONDITIONS, INCLUDING LOCATIONS AND SIZE OF SLEEVE SLOTS SHALL BE SHOWN ON THE SHOP DRAWINGS.

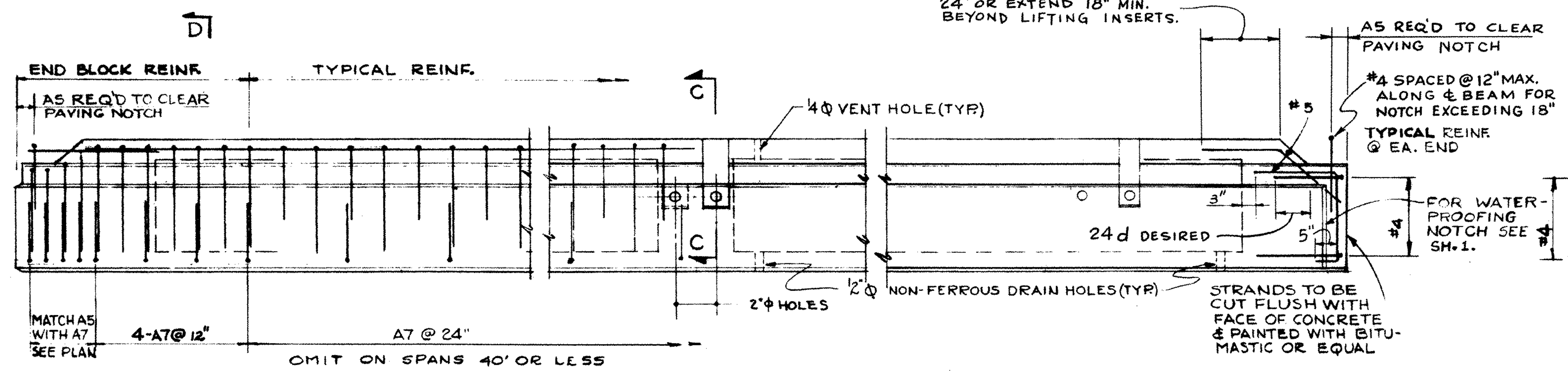


NOTE
AT FABRICATORS OPTION SPLAYED "Γ" BARS MAY BE SUBSTITUTED FOR BARS A5 & A7 @ ENDS

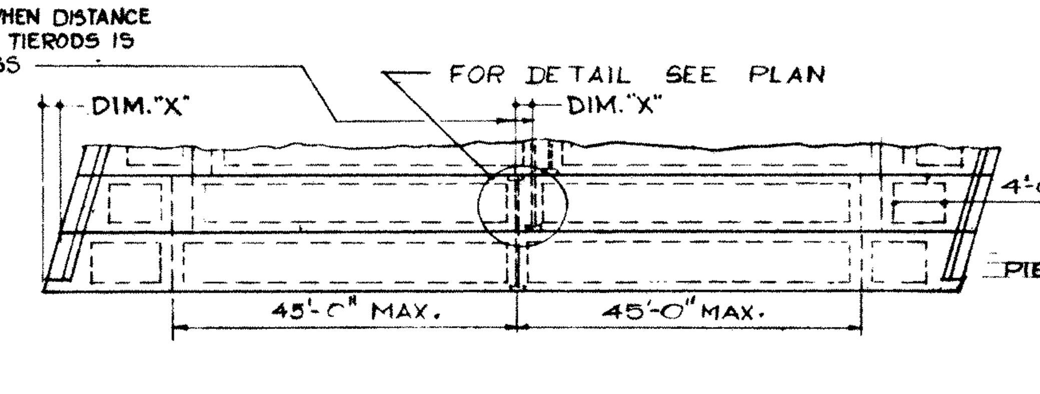
STIRRUPS SPACING TO BE USED AS A GUIDE. FOR SPANS EXCEEDING THE DEPTH TO SPAN RATIO OF 1/29 COMMPUTATIONS FOR STIRRUPS SHALL BE SUBMITTED.

PLAN ♦

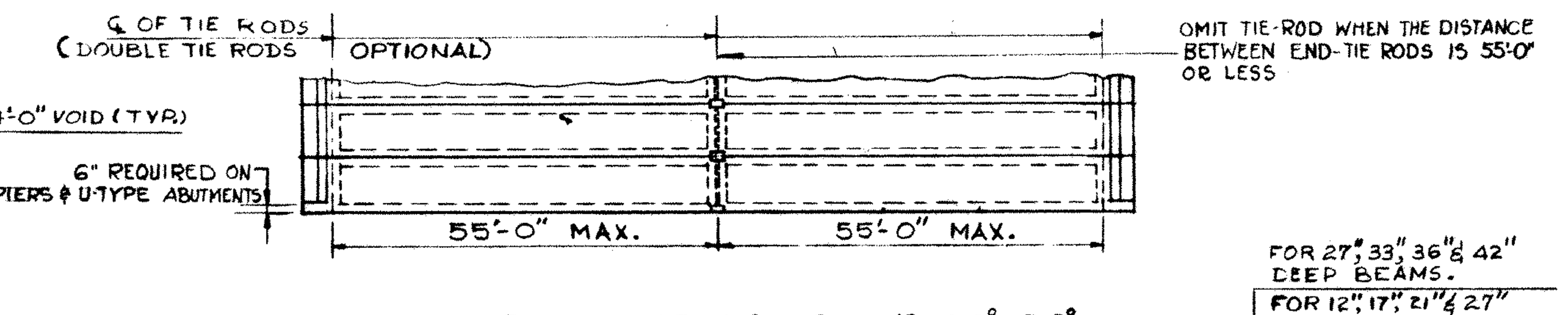
♦ TYPICAL PLAN FOR SKEWS LESS THAN 80°. FOR SKEWS 90°-80°, MODIFY AS NECESSARY.



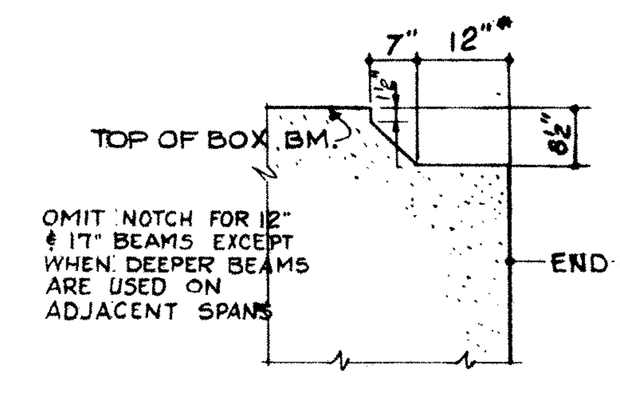
ELEVATION



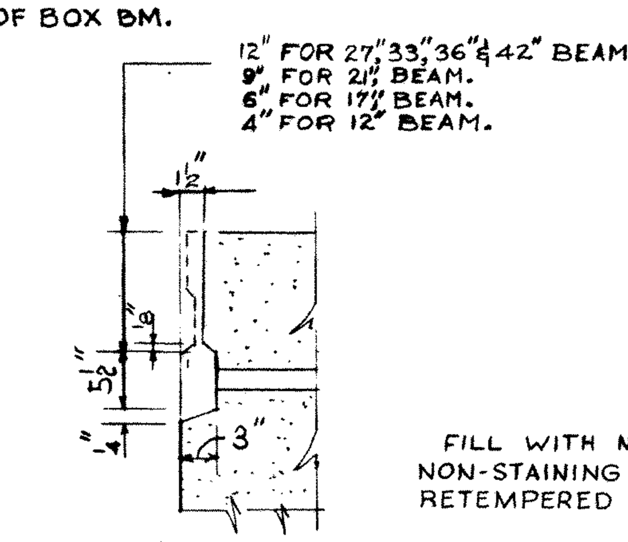
TIE ROD LOCATIONS - SKEWS LESS THAN 80°



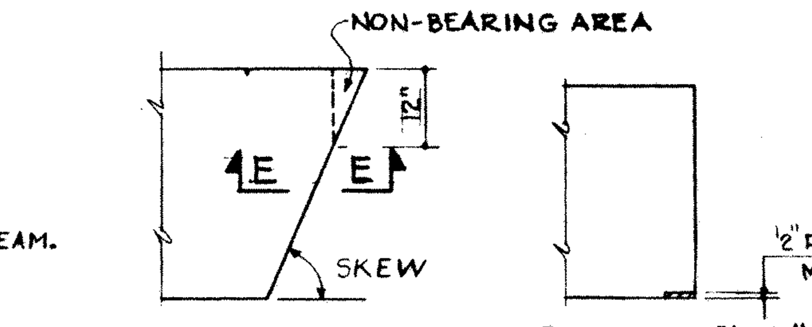
TIE ROD LOCATIONS - SKEWS 90°-80°



SECTION "A-A"



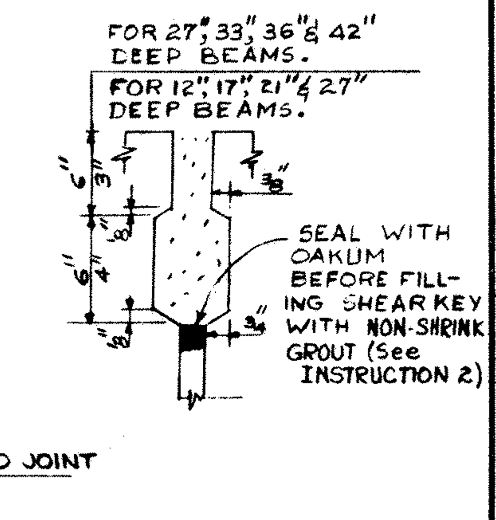
SECTION "C-C"



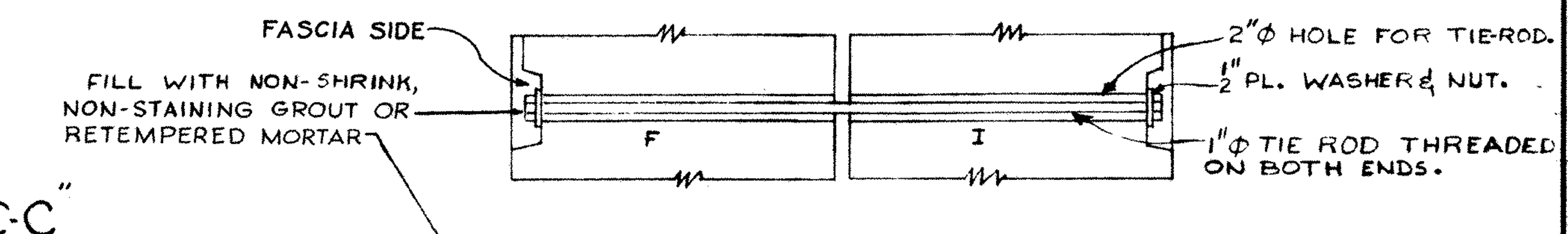
PLAN-END OF BEAM

SECTION "E-E"

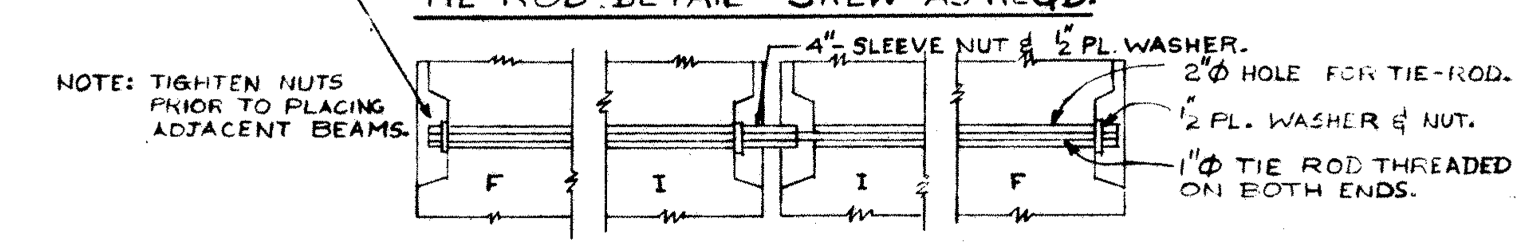
FOR SKEWS 85° OR LESS



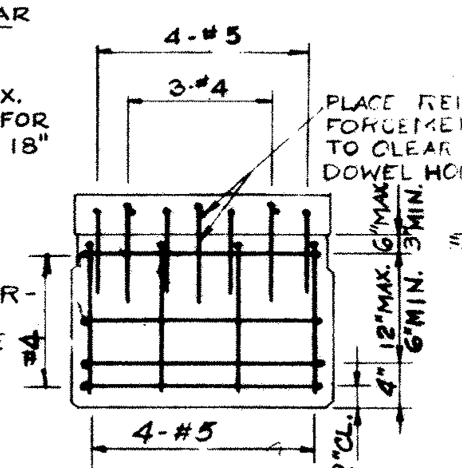
SHEAR KEY DETAIL



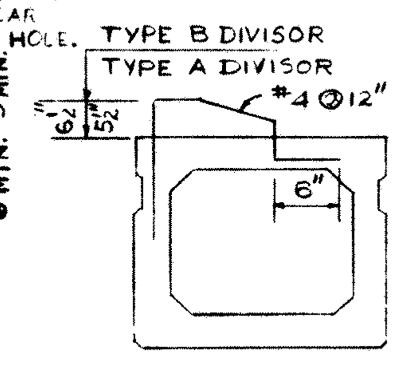
TIE ROD DETAIL - SKEW AS REQ'D.



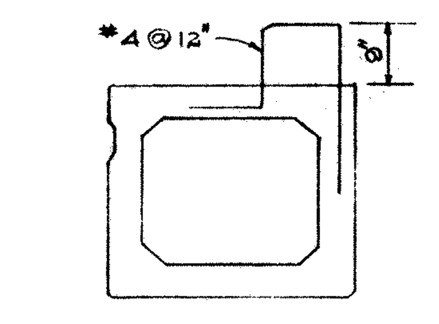
TIE ROD DETAIL - 90° SKEW



VIEW "BB"



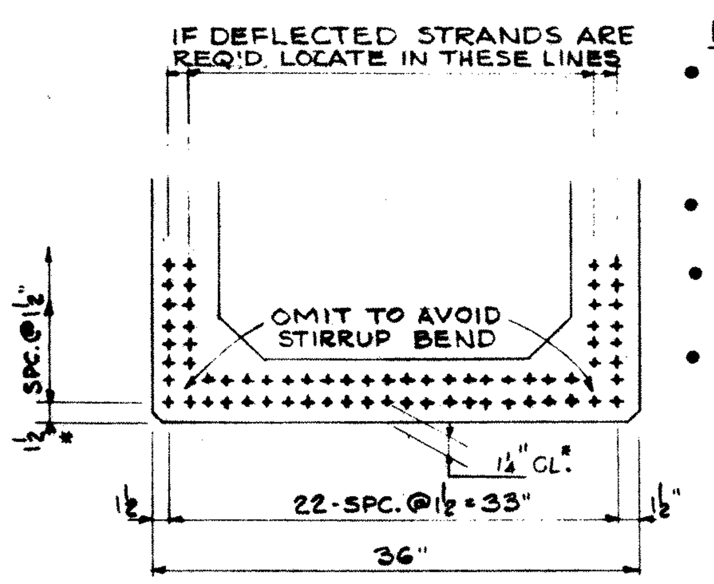
REINFORCING FOR MEDIAN



REINFORCING FOR PARAPET

INSTRUCTIONS

- APPLICABLE DETAILS SHOWN ON THIS SHEET ARE TO BE INCLUDED IN FABRICATORS SHOP DWGS.
- NON-SHRINK GROUT SHALL CONTAIN APPROVED ADMIXTURE FOR NON-SHRINKING GROUT.
- TOPS OF ALL BEAMS SHALL BE STRUCK OF AND SCORED AS SPECIFIED IN SECTION 6·14·3(3)(i).
- THIS SHEET TO BE WORKED WITH SH. 3 & 4.
- DESIGN LENGTH SHALL BE SHOWN ON THE SHOP DWGS. CASTING LENGTH MAY BE GIVEN IN ( ).
- PRESTRESS CAMBER AND DEAD LOAD DEFLECTIONS SHALL BE SHOWN ON THE SHOP DWGS. AND DESIGN PLANS.
- SHOW TYPICAL CROSS SECTION AND FRAMING PLAN OF BRIDGE ON SHOP DRAWING.
- SHOW TIE ROD DETAIL AND BEARING PAL DETAIL ON SHOP DWGS.
- FOR BEAM FABRICATION TOLERANCE SEE SH. 1.
- ERECTION SEQUENCE SHALL BE SHOWN ON THE DESIGN DWGS.



TYPICAL STRAND LOCATION FOR 3/8" φ STRAND

* 1 1/4" MIN. CLEARANCE SHALL BE MAINTAINED AT ALL TIMES.

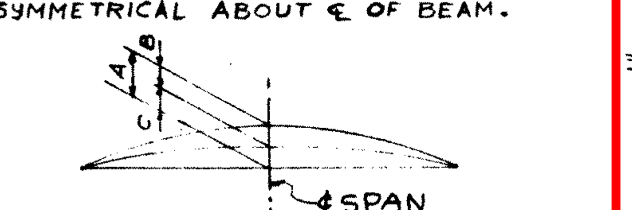
1 3/4" FROM ℄ OF STRANDS TO BOTTOM OF BEAM IS DESIRED. CHAIRS WILL BE REQUIRED FOR BEAMS WITH SPANS OVER 104'-0".

1 1/2" FROM ℄ OF STRANDS TO BOTTOM OF BEAM WILL REQUIRE CHAIRS FOR SPAN OVER 50' (TO ELIMINATE EXESS STRAND DRAPE).

CHAIRS SHALL BE DETAILED AND SHOWN ON SHOP DRAWING.

NOTE

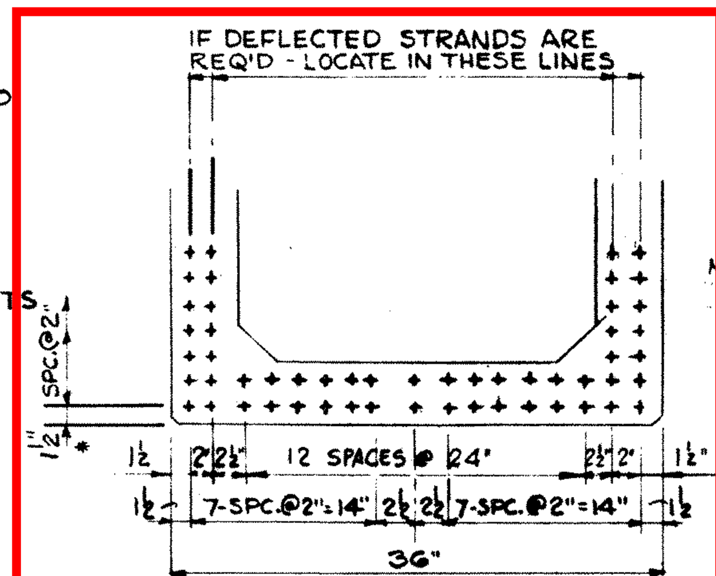
- TO OBTAIN REQUIRED ECCENTRICITY OF PRESTRESSING FORCE @ ℄ SPAN, A STRAND PATTERN SHALL BE ESTABLISHED. THE TEMPLATE PATTERNS SHOWN SHALL BE USED FOR THIS PURPOSE.
- SEE TABLE OF ECCENTRICITIES ON SHEET 4.
- OMIT STRAND @ ℄ OF BEAM AS REQ'D BY DOWEL OR DRAINAGE ARRANGEMENTS.
- STRAND PATTERN TO BE ALWAYS SYMMETRICAL ABOUT ℄ OF BEAM.



A = ESTIMATED PRESTRESS CAMBER LESS DEFLECTION DUE TO DEAD LOAD OF BEAM (TO BE CHECKED IN FIELD).
B = DEFLECTION DUE TO DEAD LOAD OF W.S. CURB OR SIDEWALK AND PARAPET.
C = A-B = NET FINAL CAMBER.

- BEARING SEAT ELEVATIONS & WEARING SURFACE THICKNESS SHALL BE CALCULATED CONSIDERING THE VALUE "C".
- THE THICKNESS OF THE WEARING SURFACE SHALL BE INCREASED TO COMPENSATE FOR ANY INACCURACIES IN THE CAMBER OF THE BEAMS.
- PRESTRESSED CAMBER AND DEAD LOAD DEFLECTION DATA SHOWN IS THEORETICAL AND MAY VARY WITH CONCRETE STRENGTH, VARIABLE PRESTRESSING CONDITIONS AND PRESTRESS LOSSES.

TYPICAL CAMBER DIAGRAM & NOTES
(TO BE SHOWN ON SHOP DWGS. & DESIGN PLANS)



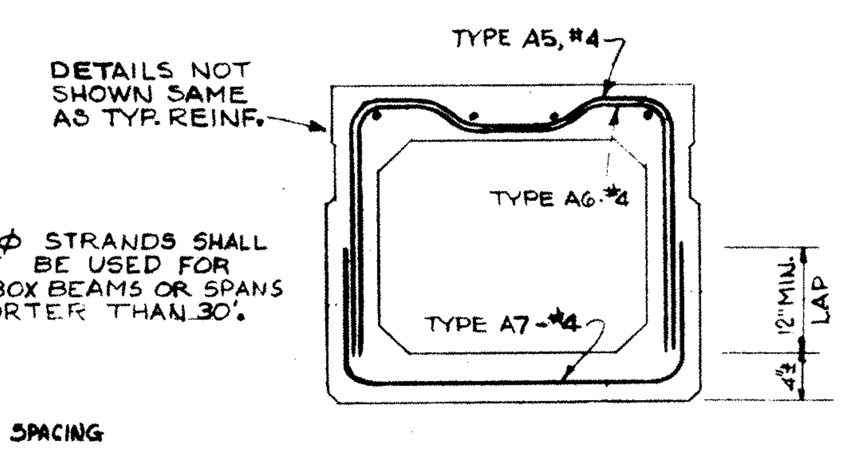
TYPICAL STRAND LOCATION FOR 7/16" φ STRAND

NOTE: 7/16 φ STRANDS SHALL NOT BE USED FOR 12" BOX BEAMS OR SPANS SHORTER THAN 30'.

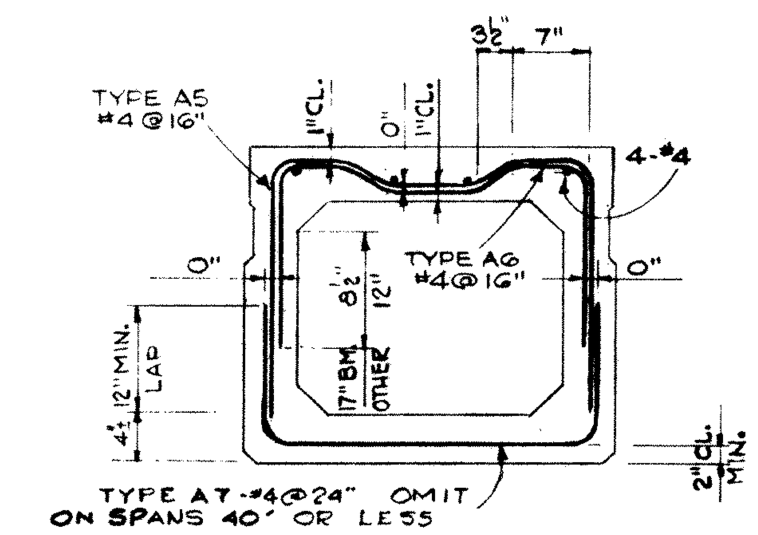
INSTRUCTIONS FOR CAMBER

- CAMBER DUE TO PRESTRESS AND DEFLECTION DUE TO BEAM DEAD LOAD MAY BE CALCULATED USING; A 5% TO 15% LOSS IN PRESTRESS FORCE, $f'ci$ IN CALCULATING E_{ci} AND A CREEP FACTOR OF 1.5 TO 2.C (IN CASE EXACT VALUES ARE NOT AVAILABLE 5% & A FACTOR OF 2.0 IS ACCEPTABLE).
- THE DEFLECTION DUE TO SUPERIMPOSED DEAD LOADS SHALL BE CALCULATED USING; $f'c$ IN CALCULATING E_c AND NO CREEP FACTOR.

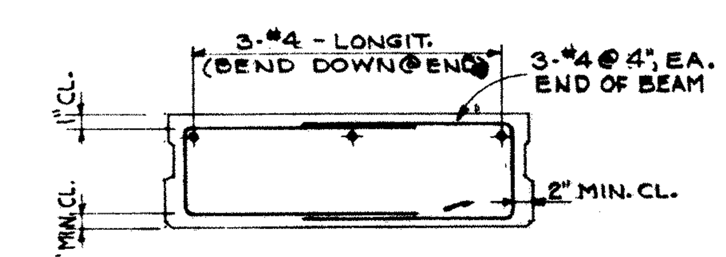
THE END BLOCK REINFORCING SHOWN SHALL BE INCREASED FOR SHARP SKEWS AND FOR HIGHLY CONCENTRATED PRESTRESSING FORCE. REARRANGEMENT OF STRANDS SHALL BE GIVEN CONSIDERATION.



SECTION "DD"
END BLOCK REINF.

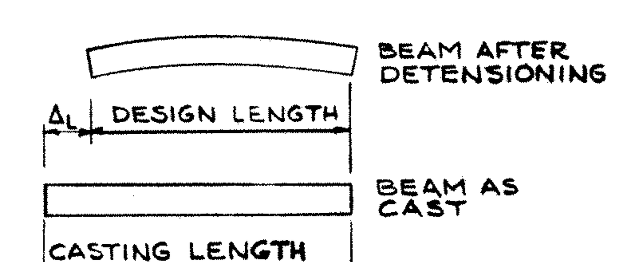


TYPICAL REINFORCEMENT



REINFORCEMENT - 12" BEAM

ANTICIPATED Δ_L INCHES				
BOX	SPAN			
	30	50	70	90
17×36	1/8	5/8		
21×36	1/8	1/2		
27×36	0	3/8	3/4	
33×36	0	1/4	5/8	
36×36	0	1/4	5/8	7/8
42×36	0	1/4	1/2	7/8



BEAM LENGTH

NOTE: THE STRAND PATTERNS shown are for ASTM GRADE STRANDS THE SPACING OF SPECIAL GRADE STRANDS SHALL BE SUCH THAT THE FORCE PER ROW IN THE FLANGES OF BEAMS SHALL BE NEARLY THE SAME MAGNITUDE AS IF ASTM GRADE STRANDS WERE USED.

APPROVED AUG 17 1964
K. H. Jensen
BRIDGE ENGINEER

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Department of Highways
BRIDGE UNIT
PRESTRESSED CONCRETE BRIDGE STANDARDS
3 FT. ADJACENT BOX BEAMS
BITUMINOUS SURFACE COURSE
BEAM DETAILS

SCALE: No Scale.
SHEET 5 OF 7

ST-207

46
PREPARED BY SCHUPACK AND ZOLLMAN, NEWTOWN SQUARE, IN COOPERATION WITH THE PRESTRESSED CONCRETE ASSOCIATION OF PENNSYLVANIA FOR THE PENNSYLVANIA DEPARTMENT OF HIGHWAYS.

Appendix B- Current Rating Method

(PennDOT SOL-431-07-08)

DATE: September 24, 2007 431-07-08

SUBJECT: Bridge Safety Inspection Program,
Revision to Publication 238 Bridge Safety Inspection Manual,
Inspection and Rating of Adjacent Non-Composite Prestressed
Concrete Box Beams

TO: **District Executives**

FROM: Brian G. Thompson, P.E. /s/
Acting Director for
Bureau of Design

This Strike-Off Letter, a time neutral policy, contains the Department's policy on the safety inspection and load rating of adjacent non-composite prestressed concrete box beam bridges.

In accordance with Publication 238, IP 1.6 and 1.7, local bridge owners shall also comply with these provisions.

The following pages are to be inserted into the October 2002 Edition of Publication 238:

- IP 2.12 — Adjacent Non-composite Prestressed Concrete
- IE 3.8.3.3.1I — Adjacent Non-composite Prestressed Concrete Box Beams
- IE 6.6.3.3.1I — Adjacent Non-composite Prestressed Concrete Box Beams
- IE 6.7.1 — Dead Load
- IE 6.7.3 — Distribution of Loads
- Appendix IP 03-B — Guidelines for Live Load Rating of Selected Concrete Bridges Without Plans Using Engineering Judgement

If you have any questions, please contact Thomas P. Macioce, P.E., Chief Bridge Engineer at (717) 787-2881 or Harold Rogers, P.E., at (717) 787-3767.

Attachments

4310/HCR/pvd

cc: R. H. Hogg, P.E., 8th Floor, CKB
B. G. Thompson, P.E., 7th Floor, CKB
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100 Airside Drive/Moon Township, PA 15108
Attn: Mark Mlynarski, P.E. and
Ray Hartle, P.E.

Mackin Engineering Company
Attn: Mr. Elmer Jarvis
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117 Industry Drive
Pittsburgh, PA 15275

2.11 SIGN STRUCTURE SAFETY INSPECTIONS

THIS SECTION IS CURRENTLY UNDER REVIEW AND WILL BE RELEASED AT A LATER DATE

2.12 ADJACENT NON-COMPOSITE PRESTRESSED CONCRETE BOX BEAMS

The December 2005 collapse of a fascia beam on the bridge carrying State Route 1014 over Interstate 70 in Washington County resulted in a review of the procedures and practices used in the safety inspection and load rating analysis for adjacent non-composite prestressed concrete box beam bridges.

The bridge had four simple spans with a bituminous wearing surface (without a waterproofing membrane). There had been considerable damage to the failed fascia beam and interior beams due to overheight vehicle collisions. Through the years, the loss of additional prestressing strands occurred due to continuing corrosion. Some of this strand loss was not detectable using routine visual inspection methods. The result was that the fascia beam collapsed under its own weight.

As a result of this review and studies of the failed beam conducted by the University of Pittsburgh and Lehigh University, the following sections of Publication 238 have been modified or added:

- Field Inspection Guidelines: For guidelines on inspection procedures and documentation of findings, see IE 3.8.3.3.1I.
- Load Rating: For guidelines on general requirements, see IE 6.6.3.3.1I. For distribution of barrier dead load, see IE 6.7.2. For distribution of live loads, see IE 6.7.3. Load ratings by Engineering Judgement per Appendix IP 03-B is no longer permitted for adjacent non-composite prestressed concrete box beams, see IP 03-B, Applicability of Guidelines.

The Navigational Controls are to be inventoried and noted in BMS Items D12 and D12A. The conditions of these controls should be noted in the inspection report with the substructure unit(s) they protect.

3.8.3 Superstructure

"The following shall replace the first sentence of M 3.8.3".

This article includes discussions covering inspection of all commonly-encountered types of superstructures composed of prestressed concrete, reinforced concrete, structural steel, iron or timber, including bearings, connection devices, and protective coatings.

3.8.3.1 STEEL BEAMS, GIRDERS AND BOX SECTIONS

"The following shall supplement M 3.8.3.1".

Guidance and requirements for the inspection of steel bridges considering fatigue and fracture is presented in IP 2.4.

3.8.3.2 REINFORCED CONCRETE BEAMS AND GIRDERS

"The following shall supplement the first paragraph of M 3.8.3.2".

To aid in locating hairline cracks, wet the concrete surface with small amounts of water and allow to dry. Cracks will be visible due to capillary action of the water in the cracks.

3.8.3.3 PRESTRESSED CONCRETE, BEAMS, GIRDERS AND BOX SECTIONS

"The following shall supplement the first paragraph of M 3.8.3.3".

For Prestressed beams made continuous for live load, examine the beams carefully for cracks in the region within two to three beam depths from interior supports. Diagonal web cracks may be evidence of shear-related problems. Transverse cracks across the bottom flange may be caused by poor bonding or development of the positive moment hook bars and/or the pre-stressing strands. Longitudinal cracking of the bottom flange, especially in box beams, may be an indication of corrosion of prestress strands. The level of inspection intensity and the presence or lack of cracking should be noted in the field reports so that long-term performance of beams can be tracked. Because the details and methods of construction for pre-stressed beam bridges made continuous for live load are varied, the design, shop drawings, and construction records should be carefully reviewed for the inspection.

IC3.8.3.3 Prestressed concrete beams made continuous for live load may be subject to positive moment stresses at interior supports due to forces created by restraint of creep and shrinkage of the beam concrete.

To aid in locating hairline cracks, wet the concrete surface with small amounts of water and allow to dry. Cracks will be visible due to capillary action of the water in the cracks.

3.8.3.3.1I ADJACENT NON-COMPOSITE PRESTRESSED CONCRETE BOX BEAMS

The inspection of adjacent non-composite prestressed concrete box beams is to include a review of the items listed below with the findings documented in the inspection report:

IC3.8.3.3.1I Without an effective Non-Destructive Evaluation (NDE) tool to detect the extent of strand corrosion and the remaining effective prestressing force,

REQUIREMENTS

Beam Spalls and/or Delaminations:

- Location on beam
- Dimensions of spall (length, width, depth)
- Type and size of steel exposed, if any, (mild or prestressing steel)
- Probable cause of spall
- Date spalls were first discovered

Note: Loose concrete should be removed during inspection to determine extent of spall and to prevent debris from falling on any underpassing route.

Exposed and/or Damaged Strands:

- Location within span.
- Number and size of strands exposed/damaged
- Date strand exposure/damage first noted
- Probable Cause, if different from spall

Other General Information:

- Web cracks - number, width , orientation, and location. **Note: Cracks directly under or beginning at an open deflection joint parapet in the middle ½ of the span should be suspected as a potential indicator of sudden beam failure. Notify BQAD immediately to assist in the evaluation.**
- Flange cracks - number, width, orientation, and location
- Beam camber or sag - Flat or negative beam camber seen in the field may be indicative of internal distress. Measurements can be made to compare to as-built conditions or shop drawings.
- Shear key condition, if visible. Leakage through the shear keys or longitudinal cracks in the pavement shall be noted.

Plan and Cross-Section Sketches of Beams

The bridge inspection and rating file shall contain a plan and cross-section of any beam rated. All beams with exposed strands shall have a cross-section showing the size and locations of exposed and/or damaged strands. For consistency, use the following symbols on the beam cross-section for documentation during inspection and analysis:

- • Strands still effective
- o Strands presumed (not known) to be not effective
- x Lost strand (Broken or corroded). Exposed strands shall be considered as "lost" unless corrosion is minimal (mostly shiny surface).

Adjacent non-composite prestressed concrete box beam bridges with damaged strands or concrete shall be considered high priority for inspection and ratings.

COMMENTS

the best information of current beam conditions must be made available to the rating engineer to predict the safe load capacity. Some items, above and beyond the strand loss and concrete deterioration/damage, that may be contributing factors to failures include:

- No concrete deck – when only a bituminous wearing surface and no waterproofing membrane is provided, roadway drainage can be held in the overlay, creating a continually wet environment for corrosion.
- Without a composite concrete deck, redundancy of beams is reduced.
- Shear keys – poor quality grout does not provide an effective load transfer mechanism between beams. The effectiveness of the shear key can deteriorate with age.
- Transverse tie rods – without significant post-tensioning and/or effective shear keys, tie-rods cannot be fully depended upon for load sharing, especially for fascia beams.
- Severe skew (< 60º)
- Asymmetrical loss of prestressing force and/or concrete quality due to damage or corrosion.
- Open joints between parapet sections can direct roadway drainage onto the outside face of the fascia beam and provide a point of reduced beam stiffness or stress concentration.

3.8.3.4 TIMBER SYSTEMS

"The following shall supplement M 3.8.3.4".

Stressed timber superstructures should receive special attention during inspections. Stressed timber superstructures consist of longitudinal timber planks (set on edge) that are squeezed together by transverse prestressing (post-tensioning) high strength steel bars. This prestressing

IC3.8.3.4 The Transportation Research Record 1740 Paper No. 00-1191 entitled "Field Performance of Stress-very low ratings for beams especially at beam ends where ΦM_n will most always be less than $1.2M_{cr}$.

6.6.3.3.1I ADJACENT NON-COMPOSITE PRESTRESSED CONCRETE BOX BEAMS

Load ratings of beams with deteriorated and/or damaged prestressing strands are to be based on the following procedures:

- Visually observed strands + 25% - Deduct 100% of all exposed strands plus an additional 25% (125% of the total area of the exposed strands) from capacity calculations.
- Strands adjacent to or intersecting a crack shall be considered ineffective in the region immediately adjacent to the crack.
- If significant strand loss is noted (>20%), especially for fascia beams, contact BQAD for further instructions.
- For beams with no exposed strands but which appear to have internal damage (as evidenced by bottom flange cracking with rust and/or delamination), contact BQAD for further instructions.
- For fascia beams with Capacity/Dead Load < 1.5 or an Operating Rating < 1.5 based on a conventional analysis, an analysis that considers biaxial stresses will be performed by BQAD.
- These analysis methods may also be applicable to other pre-stressed box beam bridges

IC 6.6.3.3.1I Based on limited research of beams with longitudinal cracks in the bottom flange, the strand above the crack as well as the two adjacent lower layer stands may be deteriorating. For this condition, a parametric study of strand loss should be performed to determine the sensitivity of beam capacity to strand loss.

Because the live load portion of the total load carried by fascia beams is small, the load rating may be > 1.0 and not reflect the marginal capacity above dead load. Thus when Capacity/Dead Load is < 1.5, a more detailed analysis is required.

6.7 LOADINGS

Additional requirements for PA bridges are contained in IP 3

6.7.1 Dead Load

"The following shall supplement the second paragraph of M 6.7.1".

For encased I-beam (EIB) bridge analyses, the following criteria will determine whether the composite or non-composite section carries the superimposed dead load and live load:

- If the structure was built using <u>Shored</u> construction, the composite section may be used to carry the superimposed dead load and the live load.
- If the structure was built using Unshored construction, the non-composite section is to be used to carry the superimposed dead load and the live load.
- If the Deck or Superstructure (BMS Item E17 or E18) is in poor condition, the non-composite section is to be used to carry the superimposed dead load and the live load regardless of the construction method used to build the structure.

IC6.7.1 See BAR7 computer program documentation for discussion regarding EIB beams and their analysis.

"Add the following paragraph at the end of M 6.7.1".

For adjacent non-composite prestressed concrete box beams, the following criteria shall be used to determine the distribution of barrier dead loads:

REQUIREMENTS COMMENTS

- Assume fascia beams support 100% of the barrier dead load.
- Assume the first interior beams support 50% of the barrier dead load.

6.7.2 Rating Live Load

Additional requirements for PA bridges are contained in IP 3.2.2

6.7.3 Distribution of Loads

"Add the following paragraph at the end of M 6.7.3".

For adjacent non-composite prestressed concrete box beams, the following criteria shall be used to determine the distribution of live loads for moment and shear:

- Fascia girder shall use the larger of the LFD Distribution Factor (IP 3.3.2.2) or Lever Rule (AD 4.6.2.2).
- Interior girder shall use a wheel load distribution factor = 1.0 where there is a loss of grout in the shear key and/or tie rod.

6.8 DOCUMENTATION OF RATING

Additional requirements for PA bridges are contained in IP 8.

Guidelines for Live Load Rating of
Selected Concrete Bridges Without Plans
Using Engineering Judgement

Description:

The following is a guideline for using engineering judgement to determine the live load rating capacity of selected concrete bridges where the structural components of the main load carrying members are not known with sufficient confidence to use an analytical approach for the rating. These bridges are frequently known as “concrete bridges without plans”. These guidelines follow the approach outlined in IP 3.2.2.1.

Disclaimer:

This guideline does not relieve the rating engineer of his responsibility of determining the applicability of the bridge to this methodology, of properly accessing the condition of the bridge and its behavior under live load, and/or of verifying the accuracy of the resulting ratings.

Applicability of Guidelines:

- The structural components of the main load carrying members are not known sufficiently to use an analytical approach to determine the live load ratings
- The condition of the main load carrying members is known and rated using the Condition Rating as set forth in *BMS Coding Manual Pub 100A*
- The behavior of the bridge under vehicular live load is known by visual observation
- This method is limited to the following types of non-Fracture Critical superstructures
  - Reinforced Concrete Slab
  - Reinforced Concrete T-Beam
  - Prestressed/Pretensioned Concrete Beams (Not permitted for Adjacent Non-Composite Prestressed Concrete Box Beams)
- The method is limited to simple span structures with lengths from 8’ to 50’ and for all skews
- The inspection frequency specified in Table A of these guidelines will not be exceeded

Assumptions:

1. The critical legal load for the range of applicability is the ML80 vehicle
2. Moment controls the live load rating
3. Safe Load Capacity = 100% of Operating Rating, except for members in critical or serious condition
4. Inventory Rating = 60% of Operating Rating

Procedure:

1. Determine the condition rating for the critical main load-carrying member of the bridge from a bridge safety inspection performed in accordance with Pub 238.

2. Determine the distress level of the bridge superstructure under vehicular live load using Table B of these guidelines.

   NOTE: Member condition ratings of 5 through 9 should not see distress under live load. If distress is observed, the member condition rating should be no higher than a 4.

3. Determine the ML80 truck live load ratings (IR-Inventory Rating, OR-Operating Rating, SLC-Safe Load Capacity) using Table A of these guidelines, using the following:
   A. Condition rating of main load carrying member
   B. Distress level of bridge superstructure
   C. ADTT (Average Daily Truck Traffic) on the bridge

4. Determine the live loadings for the other Bridge Posting Vehicles (H, HS, TK527) based on a comparison of their live load bending moments to the ML80 bending moment for the bridge’s span length.
   A. Determine the Rating Factor for ML80 Safe Load Capacity

$$\text{Rating Factor for } SLC_{ML80} = (SLC_{ML80} / W_{ML80})$$

where: SLC_{ML80} *is from Table A*
W_{ML80} *= ML80 Gross Vehicle Weight = 36.64 T*

Appendix D – PS3 Program Output

```
********************************************************************************
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Proposed Method (Lehigh)

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PROGRAM P4353030                                               06/04/2010 15:16
VERSION 3.5.0.1(TEST)     LAST UPDATED 12/29/2003        DOCUMENTATION 06/2002

INPUT: C:\Users\Thomas\Desktop\NEW_ME~1.DAT
```

Proposed method

```
NEW_METHOD BEAM 3 WITH STRAND LOSS

STRUCTURE ID - 35730215080021 - ASH ST OVER ROARING CR
```

SLC LEVEL	LIVE LOAD	OUT-PUT	IMPACT FACTOR	GAGE DISTANCE	PASSING DISTANCE	ROADWAY WIDTH	LOAD FACTORS DLF	LLF	I OR F
	H	0	0.000	0.0	0.0	36.75	0.00	0.00	I

PRINCIPAL STRESSES	DESIGN	SKEW CORRECTION FACTOR	IR STRESS LEVEL	AASHTO FC
	R	0.000	0.000	

BRIDGE CROSS SECTION AND LOADING

BEAM SPACING	DISTRIBUTION FACTORS SHEAR	MOMENT	DEFLECTION	UNIT WEIGHT OF DECK CONCRETE	UDLF	DEAD LOADS DL1	FWS	DL2	INITIAL P/S FORCE
36.5	0.500	0.279	0.250	0.0000	0.1000	0.000	0.000	0.000	0.000

ECCENTRICITY MIDSPAN	END	P/S LOSS %	XDRAPE	LEHIGH LOSS METHOD T0	TS	TD	IC	MFG	IST	STRAND L or S	RATINGS w/ & w/o FWS
0.000	0.000	0.08	0.0000	0	0	0	0	0	0	S	N

SPAN LENGTHS (SIMPLE)

SPAN #	1	2	3	4	5	6	7	8	BEAM PROJ
LENGTH	66.60								9.000

PRESTRESS CRITERIA

BEAM CONC F'CB	SLAB CONC F'CS	CONC INIT F'CI	STEEL INIT FSI	STEEL YIELD Fy	STEEL ULT F'S	INITIAL ALLOWABLE COMP FCI	TENS FTI	DRP/DBND FTFD
5.000	0.000	0.000	175.0	0.0	250.0	0.000	0.000	0.000

FINAL ALLOWABLE COMP FC	TENS FT	SLAB FCS	ALLOW SHEAR VHA	OR STRESS LEVEL	STEEL E	MODULAR RATIOS DES	ULT	CREEP FACTOR	EST. % LOSS	STRAND DIAMETER
0.000	0.000	0.000	0.000	0.000	0	0.000	0.000	0.0	0.0	0.4380

NUMBER OF ROWS	NUMBER OF Lx	STIRRUP DETAILS
0	0	Y

PRESTRESSED CONCRETE BEAM DIMENSIONS

TYPE	COMP	DESIGNATION	D	W1	W2	W3	T1	T2
B	N	36/36	36.000	36.000	35.250	5.000	5.500	5.000

B1	B2	B3	B4	D1	D2	X1	X2	SLAB THICK	HAUNCH
3.00	3.00	3.00	3.00	6.00	6.00	0.375	0.750	0.00	0.00

STRAND DETAILS

AREA	G1	G2	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10
0.109	1.50	2.502	20									

STIRRUP DETAILS

SPEC. FOR ANAL/RATE	STIRRUP AREA	FSY	LOCATION	SPACING	LOCATION	SPACING	LOCATION	SPACING
	0.200	40	0.00	6.000	2.00	12.000	4.00	24.000
			0.00	0.000	0.00	0.000	0.00	0.000

DEFAULT VALUES

GAGE DIST	PASS DIST	DLF	LLF	SKEW C.F.	UNIT WT DK CONC	INT DIA THICK	INT DIA WEIGHT
6.0	4.0	1.30	2.17	1.000	0.150	10.0	0.560

CN INI F'CI	ST YLD Fy	AASHTO FC	COMP FC	TENS FT	ALLOW SHR-VHA	OR STR LEVEL	IR STR LEVEL
4.250	212.5	N	2.000	0.212	0.300	0.900	0.800

STEEL E	CREEP FACTOR	SPEC A/R
28000	1.6	1979

ONE INTERIOR DIAPHRAGM IS ASSUMED AT MIDSPAN

```
*******************************************************************************
*                        RATING OF AN INTERIOR BEAM                           *
*******************************************************************************
```

BASIC BEAM SECTION PROPERTIES

DEPTH IN	AREA IN.2	WEIGHT LBS/FT	M OF I IN.4	N.A. TO TOP YT IN.	N.A. TO BOT YB IN	Z TOP IN.3	Z BOT IN.3
36.00	637.5	689.05	104563.0	18.49	17.51	5656.4	5970.2

UNIFORM DEAD LOADS ACTING ON GIRDER (KIPS/FT)

GIRDER WEIGHT	FORMWORK WEIGHT	INPUT DL1	FUTURE WEARING SURFACE	INPUT DL2	TOTAL DL1	TOTAL DL2
0.6891	0.3042	0.0000	0.0000	0.0000	0.9932	0.0000

DEAD LOAD AND LIVE LOAD REACTIONS

DL1 REACTION	DL2 REACTION	IMPACT FACTOR	LL+I H20 REACTION	LL+I HS20 REACTION	LL+I ML80 REACTION	LL+I TK527 REACTION
33.4	0.0	1.261	23.9 L	30.7 T	29.7 T	30.4 T

LL+I P-82 REACTION
54.4 T

PRESTRESSING FORCE (STRAND PATTERN UNKNOWN)

INITIAL	LOSS %	EFFECTIVE	NO. OF STRANDS	ECCENTRICITY	C.G.S.
381.500	20.00	305.200	20	15.012	2.502

****
*   RATING SUMMARY   *
****

FLEXURAL RATINGS (BASED ON MOMENT) — SHEAR RATINGS (1979 I)

LOAD		FACTOR	TONS	LOCATION FROM CL BRG		FACTOR	TONS	LOCATION FROM CL BRG
H20	IR	0.719	14.38	33.300	IR	2.855	57.10	16.650
	OR	2.159	43.17	33.300	OR	4.765	95.31	16.650
HS20	IR	0.512	18.43	33.300	IR	2.047	73.70	16.650
	OR	1.538	55.36	33.300	OR	3.417	123.02	16.650
ML80	IR	0.436	15.98	33.300	IR	1.827	66.94	16.650
	OR	1.310	48.00	33.300	OR	3.050	111.75	16.650
TK527	IR	0.451	18.05	33.300	IR	1.819	72.75	16.650
	OR	1.355	54.22	33.300	OR	3.036	121.43	16.650
P-82	IR	0.279	28.42	33.300	IR	1.078	109.98	16.650
	OR	0.823	83.97	29.970	OR	1.800	183.58	16.650

****
*   CONTROLLING RATINGS   *
****

VEHICLE TYPE	IR	OR
H20 LOADING (TONS)	14.38 F	43.17 F
HS20 LOADING (TONS)	18.43 F	55.36 F
ML80 LOADING (TONS)	15.98 F	48.00 F
TK527 LOADING (TONS)	18.05 F	54.22 F
P-82 LOADING (TONS)	28.42 F	83.97 F

F = FLEXURAL RATING   S = SHEAR RATING

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♀              PRESTRESSED CONCRETE GIRDER DESIGN AND RATING           333768

PROGRAM P4353030                                              06/04/2010 15:08
VERSION 3.5.0.1(TEST)     LAST UPDATED 12/29/2003       DOCUMENTATION 06/2002

INPUT: C:\Users\Thomas\Desktop\CURREN~1.DAT
```

current method

```
NEW_METHOD BEAM 3 WITH STRAND LOSS

STRUCTURE ID - 35730215080021 - ASH ST OVER ROARING CR
```

SLC LEVEL	LIVE LOAD	OUT-PUT	IMPACT FACTOR	GAGE DISTANCE	PASSING DISTANCE	ROADWAY WIDTH	LOAD FACTORS DLF	LLF	I OR F
	H	0	0.000	0.0	0.0	36.75	0.00	0.00	I

PRINCIPAL STRESSES	DESIGN	SKEW CORRECTION FACTOR	IR STRESS LEVEL	AASHTO FC
	R	0.000	0.000	

BRIDGE CROSS SECTION AND LOADING

BEAM SPACING	DISTRIBUTION FACTORS SHEAR	MOMENT	DEFLECTION	UNIT WEIGHT OF DECK CONCRETE	UDLF	DEAD LOADS DL1	FWS	DL2	INITIAL P/S FORCE
36.5	0.500	0.279	0.250	0.0000	0.1000	0.000	0.000	0.000	0.000

ECCENTRICITY MIDSPAN	END	P/S LOSS %	XDRAPE	LEHIGH LOSS METHOD T0	TS	TD	IC	MFG	IST	STRAND L or S	RATINGS w/ & w/o FWS
0.000	0.000	0.08	0.0000	0	0	0	0	0	0	S	N

SPAN LENGTHS (SIMPLE)

SPAN #	1	2	3	4	5	6	7	8	BEAM PROJ
LENGTH	66.60								9.000

PRESTRESS CRITERIA

BEAM CONC F'CB	SLAB CONC F'CS	CONC INIT F'CI	STEEL INIT FSI	STEEL YIELD Fy	STEEL ULT F'S	INITIAL ALLOWABLE COMP FCI	TENS FTI	DRP/DBND FTFD
5.000	0.000	0.000	0.0	0.0	250.0	0.000	0.000	0.000

FINAL ALLOWABLE COMP FC	TENS FT	SLAB FCS	ALLOW SHEAR VHA	OR STRESS LEVEL	STEEL E	MODULAR RATIOS DES	ULT	CREEP FACTOR	EST. % LOSS	STRAND DIAMETER
0.000	0.000	0.000	0.000	0.000	0	0.000	0.000	0.0	0.0	0.4380

NUMBER OF ROWS	NUMBER OF Lx	STIRRUP DETAILS
0	0	Y

PRESTRESSED CONCRETE BEAM DIMENSIONS

TYPE	COMP	DESIGNATION	D	W1	W2	W3	T1	T2
B	N	36/36	36.000	36.000	35.250	5.000	5.500	5.000

B1	B2	B3	B4	D1	D2	X1	X2	SLAB THICK	HAUNCH
3.00	3.00	3.00	3.00	6.00	6.00	0.375	0.750	0.00	0.00

STRAND DETAILS

AREA	G1	G2	R1	R2	R3	R4	R5	R6	R7	R8	R9	R10
0.109	1.50	2.875	16									

STIRRUP DETAILS

SPEC. FOR ANAL/RATE	STIRRUP AREA	FSY	LOCATION	SPACING	LOCATION	SPACING	LOCATION	SPACING
	0.200	40	0.00	6.000	2.00	12.000	4.00	24.000
			0.00	0.000	0.00	0.000	0.00	0.000

DEFAULT VALUES

GAGE DIST	PASS DIST	DLF	LLF	SKEW C.F.	UNIT WT DK CONC	INT DIA THICK	INT DIA WEIGHT
6.0	4.0	1.30	2.17	1.000	0.150	10.0	0.560

CN INI F'CI	ST INI FSI	ST YLD Fy	AASHTO FC	COMP FC	TENS FT	ALLOW SHR-VHA	OR STR LEVEL
4.250	175.0	212.5	N	2.000	0.212	0.300	0.900

IR STR LEVEL	STEEL E	CREEP FACTOR	SPEC A/R
0.800	28000	1.6	1979

ONE INTERIOR DIAPHRAGM IS ASSUMED AT MIDSPAN

****
*   RATING OF AN INTERIOR BEAM   *
****

BASIC BEAM SECTION PROPERTIES

DEPTH IN	AREA IN.2	WEIGHT LBS/FT	M OF I IN.4	N.A. TO TOP YT IN.	N.A. TO BOT YB IN	Z TOP IN.3	Z BOT IN.3
36.00	637.5	689.05	104563.0	18.49	17.51	5656.4	5970.2

UNIFORM DEAD LOADS ACTING ON GIRDER (KIPS/FT)

GIRDER WEIGHT	FORMWORK WEIGHT	INPUT DL1	FUTURE WEARING SURFACE	INPUT DL2	TOTAL DL1	TOTAL DL2
0.6891	0.3042	0.0000	0.0000	0.0000	0.9932	0.0000

DEAD LOAD AND LIVE LOAD REACTIONS

DL1 REACTION	DL2 REACTION	IMPACT FACTOR	LL+I H20 REACTION	LL+I HS20 REACTION	LL+I ML80 REACTION	LL+I TK527 REACTION
33.4	0.0	1.261	23.9 L	30.7 T	29.7 T	30.4 T

LL+I P-82 REACTION
54.4 T

PRESTRESSING FORCE (STRAND PATTERN UNKNOWN)

INITIAL	LOSS %	EFFECTIVE	NO. OF STRANDS	ECCENTRICITY	C.G.S.
305.200	20.00	244.160	16	14.639	2.875

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********************************************************************************
*                               RATING SUMMARY                                 *
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		FLEXURAL RATINGS (BASED ON MOMENT)			SHEAR RATINGS (1979 I)		
LOAD		FACTOR	TONS	LOCATION FROM CL BRG	FACTOR	TONS	LOCATION FROM CL BRG
H20	IR	0.147	2.95	33.300	IR 2.813	56.26	16.650
	OR	1.227	24.54	33.300	OR 4.696	93.91	16.650
HS20	IR	0.105	3.78	33.300	IR 2.017	72.62	16.650
	OR	0.874	31.47	33.300	OR 3.367	121.22	16.650
ML80	IR	0.089	3.28	33.300	IR 1.800	65.97	16.650
	OR	0.745	27.28	33.300	OR 3.005	110.11	16.650
TK527	IR	0.093	3.70	33.300	IR 1.792	71.68	16.650
	OR	0.770	30.82	33.300	OR 2.991	119.66	16.650
P-82	IR	0.057	5.83	33.300	IR 1.062	108.37	16.650
	OR	0.472	48.19	29.970	OR 1.773	180.90	16.650

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*                            CONTROLLING RATINGS                               *
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VEHICLE TYPE	IR	OR
H20 LOADING (TONS)	2.95 F	24.54 F
HS20 LOADING (TONS)	3.78 F	31.47 F
ML80 LOADING (TONS)	3.28 F	27.28 F
TK527 LOADING (TONS)	3.70 F	30.82 F
P-82 LOADING (TONS)	5.83 F	48.19 F

F = FLEXURAL RATING   S = SHEAR RATING