THE PORT AUTHORITY OF NY & NJ
Engineering Department

Structural Design Guidelines

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TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

TABLE OF CONTENTS

DOCUMENT CONTROL

Document History			
Version	**Issue Date**	**Author**	**Details of Changes**
Version 2018 v1.1	1/23/2019	Z. Whiteman	Updated document reference links

1.0 OVERVIEW

These guidelines are provided as an overview of the Port Authority's design standards. Design details and associated documents outlined in these documents will be provided to the success client.

The Guidelines shall not replace professional design analyses nor are the Guidelines intended to limit innovative design where equal performance in value, safety, and maintenance economy can be demonstrated. The design team shall be responsible for producing designs that comply with the Guidelines in addition to all applicable codes, ordinances, statutes, rules, regulations, and laws. Any conflict between the Guidelines and an applicable code, ordinance, statute, rule, regulation, and/or law shall be addressed with the respective functional chief. The use and inclusion of the Guidelines, specifications, or example drawing details as part of the Contract Documents does not alleviate the design professional from their responsibilities or legal liability for any Contract Documents they create. It is also recognized that the Guidelines are not universally applicable to every project. There may be instances where a guideline may not be appropriate. If the design professional believes that a deviation from the Guidelines is warranted, such a deviation shall be submitted in writing for approval to the respective functional chief.

The Structural Design Group is solely dedicated and uniquely qualified to deliver on the mission and vision of the Port Authority of New York & New Jersey. We take great pride in being members of the Port Authority of New York & New Jersey family and feel a true sense of ownership in the facilities that we help design and maintain. Our staff of structural engineers has more than 300 years' worth of combined institutional knowledge of all Port Authority of New York & New Jersey facilities and procedures. We provide a total structural design and analysis solution for each stage of project development, as well as expert evaluations of existing structures, life-cycle and cost-benefit analyses, and contract document preparation. Possessing an unparalleled level of institutional familiarity and professional expertise in structural design solutions, we pride ourselves on the ability to develop innovative solutions for the repair and upgrade of existing structures, as well as new facility designs, ensuring our clients and the Port Authority of New York & New Jersey operating departments and their patrons the highest levels of service, comfort, and safety while maintaining the maximum level of operational continuity possible during any such work.

The Structural Design Group consists of highly skilled and motivated structural engineers with unsurpassed institutional knowledge of all Port Authority of New York & New Jersey facilities. All Structural Design staff is organized into groups according to facility, enabling them to be exclusively dedicated to servicing all of the Structural Engineering needs for their Port Authority of New York & New Jersey facilities. Supplementing in-house structural staff is call-in consulting firms who work under our supervision as an extension to our staff. Our services are applied to designing and maintaining a wide variety of our facilities, including marine terminals, rail systems, airports, tunnels, bridges, highways, buildings, and security. When we retain outside consultants, we also audit their designs for reasonableness, conformance with code, and Port Authority of New York & New Jersey practice. Areas of specific expertise include bridge design, rehabilitation and retrofit work, seismic upgrades, waterfront facility design, infrastructure support and building design, progressive collapse and blast design considerations, and blast hardening.

2.0 TECHNICAL AND CODE STANDARDS/REGULATIONS

2.1 AVAILABLE STRUCTURAL TECHNICAL CODES AND STANDARDS

All codes referenced in below are available in the Structural Department on the 20th Floor at 4 World Trade Center, NY, New York. The codes reside in the structural library and with the structural principals.

2.1.1 BUILDINGS (1)

2.1.2 BRIDGES (2)

2.1.3 FEMA (SEISMIC) (3)

2.1.4 HELIPORT (4)

2.1.5 PORTS (5)

2.1.6 RAIL (6)

2.2 AMERICAN SOCIETY FOR TESTING MATERIALS (ASTM) (7)

2.3 FACTORY MUTUAL INSURANCE COMPANY (FMRC) (8)

2.4 AMERICAN CONCRETE INSTITUTE (ACI) (9)

2.5 NEW YORK CITY BUILDING CODES (10)

2.6 NFPA INTERNATIONAL (11)

2.7 OSHA REGULATIONS, DOCUMENTS, AND TECHNICAL INFORMATION (12)

2.8 PORT AUTHORITY STANDARD SPECIFICATIONS (13)

2.9 NJDOT STANDARD SPECIFICATIONS FOR ROAD AND BRIDGE CONSTRUCTION (14)

2.10 NJDOT BRIDGE AND STRUCTURES DESIGN MANUAL (15)

2.11 NYSDOT BRIDGE MANUAL (16)

3.0 Design Criteria and Special Requirements

3.1 Design Criteria — PA Specific

3.1.1 Codes, Regulations, and Standards

Refer to the Tenant Construction Review Manual. (17)

3.1.2 Seismic Criteria

3.1.2.1 Seismic Policy

Regarding existing Port Authority of NY & NJ transportation facilities in general, it is the policy of the Chief Engineer's Office to seismically upgrade them to meet modern code when the opportunity arises, such as during a major renovation or expansion.

When considering additions or modifications to existing structures, the extent of the proposed work shall be considered in the context of the entire existing structure. If the extent of the proposed modifications do not require upgrade of the entire existing structure by code, then it becomes the Agency's business decision whether to use the opportunity to fund the upgrade.

3.1.2.2 Seismic General Guidelines

All new structures shall conform to the applicable seismic requirements of the governing building code.

All additions and modifications to the existing structures shall conform to the applicable seismic requirements of the governing building code. Seismic retrofit of the existing base structure, if not required by the code, should be considered with input from the Line Department Director.

In existing PA facilities, where new components, elements, ceilings, utilities, equipment, and other appurtenances are to be installed, the structural support of all said items shall conform to the applicable seismic requirements as if the base building were new.

3.1.2.3 Applicable Seismic Codes, Regulations, and Standards

3.1.2.3.1 Buildings in New York City

Buildings in New York City: Comply with the latest New York City Construction Codes and their Reference Standards.

3.1.2.3.2 Buildings in New Jersey

Buildings in New Jersey: Comply with the latest New Jersey Uniform Construction Code (NJUCC), its bulletins, and the sub-codes with their Supplements and Reference Standards.

3.1.2.3.3 Vehicular Bridges in New York

Vehicular bridges in New York: Comply with the more stringent of the latest American Association of State Highway and Transportation Officials (AASHTO) and the New York State Department of Transportation (NYSDOT).

3.1.2.3.4 Vehicular Bridges in New Jersey

Vehicular bridges in New Jersey: Comply with the more stringent of the latest AASHTO and the New Jersey Department of Transportation (NJDOT).

3.1.2.3.5 Bi-State Vehicle Bridges

Bi-state vehicular bridges that span from New York to New Jersey: Comply with the more stringent of the latest AASHTO, NYSDOT, and NJDOT.

3.1.2.3.6 Railroad Bridges

Railroad bridges: Comply with the latest American Railway Engineering and Maintenance-of-Way Association (AREMA).

3.1.2.4 FEMA 414 INSTALLING SEISMIC RESTRAINTS FOR DUCT AND PIPE

3.1.3 TUNNEL WIND

3.1.3.1 PATH TUNNEL WIND PRESSURES

In general, wind loads used for design in the PATH tunnel areas are 30 psf for pressure and suction normal to the surface, plus a lateral load of 2.5 psf or 8.5% of the positive wind pressure, whichever is greater, acting parallel to the surface of the structural component. For lightweight ceiling design, use an additional non-concurrent single concentrated live load of 200 lb at all ceiling supports and connections. A 33% increase in allowable stresses is permitted for loading combination DL + LL + WL only. This criterion also applies to the station areas in the tunnels.

3.1.4 CEILINGS

3.1.4.1 RESTRICTIONS ON ACCESSING CEILINGS

No one is permitted to enter an existing ceiling of any kind or impose any other additional loading on an existing ceiling unless and until one of the following is submitted to the authority for review and approval:

- ❒ Analysis indicating that the ceiling system can support the additional live load.
- ❒ An acceptable ceiling shoring system.

3.1.4.2 GYPSUM ROOF PLANKS – CONDITION EVALUATION CRITERIA

A. Policy

The recommendations shall be either 1 or 2:

(Monitoring or repair on a priority basis is not acceptable).

1. Repair on an immediate basis
2. No action required

B. Engineering Assessment Criteria

1. Conditions for Immediate Repairs:
    a. Large holes through planks
    b. Large spalls

c. Extensive map cracking

d. Water stained and powdery composition

e. Hollow and delaminated

2. Conditions where no action is required:

a. Minor hairline

b. Minor corrosion of metal edges

c. Small spalls

d. Small holes

C. Options for Immediate Action

1. Temporary Measures:

a. Cordon-off roof area as required to maintain a safe condition for a minimum of 6 ft. around the perimeter of the damaged planks

b. Protect the area below the damaged planks by one of the following actions:

- ❒ Cordon off the area below the roof
- ❒ Install netting below roof
- ❒ Install a protective shield below roof of metal decking or wooden planking

2. Permanent Measures:

a. Replace gypsum planks with concrete planks. (If an excessive amount of planks are replaced in one area, an engineering analysis should be performed).

b. Install structural supports underneath planks

Previous & Present Priority Table Sample

Status of Previous and Present Priority Repairs					
Repair Item #*	Location	Previous Priority Repair	Current Status	Present Priority Repair	Remarks & Recommendations

*** Number only those items that remain or are presently recommended as priority repairs.**

3.1.4.3 PATH TUNNEL WIND PRESSURES

3.1.4.4 CEILINGS – HEAVYWEIGHT & LIGHTWEIGHT CRITERIA (18)

3.1.5 SIGNS

3.1.5.1 SIGN STRUCTURE LOADING

A. Highway signs and luminaries shall conform to the latest AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals.

B. Exterior signs adjacent to vehicular traffic such as streets, highways, trains and light rail vehicles or open terrain shall be designed in accordance with the latest AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries, and Traffic Signals. All other exterior signs shall be designed as per the latest applicable local building code.

C. Interior signs located in the entrance lobbies, entrance vestibules or boarding platforms of transportation terminals shall be designed for an incidental wind gust pressure of 15 psf. All other interior signs shall be designed for a minimum lateral pressure of 5 psf.

D. Interior signs shall also be designed for earthquake loads as per the latest applicable local building code.

E. All signs (exterior or interior) that require routine cleaning or servicing, i.e., variable message, internally lighted, etc., whether or not specifically designed for a servicing device, shall be designed for all anticipated additional loads, but not less than a 100-lb concentrated horizontal load and a 300-lb concentrated vertical load applied at the point of assumed or most eccentric loading. The additional concentrated loads shall be applied in combination with the sign dead load (not concurrent with wind or earthquake loads.

3.1.5.2 OVERHEAD SIGN STRUCTURE NOTES [19]

3.1.5.3 OVERHEAD SIGN STRUCTURE DESIGN MANUAL, NYSDOT [20]

3.1.6 TEMPORARY LOADS (SAMPLE NOTES)

3.1.6.1 TEMPORARY PLATFORMS BELOW BRIDGES (SAMPLE NOTES)

A. Decking shall be designed using the Allowable Stress Design method, for a live load of 100 psf.

B. Deck framing shall be designed using the Allowable Stress Design method, for a live load of 70 psf.

C. Hangers and hanger connections shall be designed using the more stringent of:

- Allowable Stress Design method, for a live load of 70 psf or
- Ultimate Loading, for a load of 4 times the dead and live loads.

D. The requirement for uplift design can be waived for areas of the platform that are completely enclosed by the arch, spandrel walls and rib walls.

E. Sway bracing must be installed along any platform edges that are not in contact with a wall. They will not be required along walls.

F. OSHA approved railings must be installed along any platform edges that are not in contact with a wall. They will not be required along walls.

G. Existing bridge members shall be analyzed using the Allowable Stress Design method, for a construction live load of 70 psf. In addition to the global shear and bending stresses, local shear and bending stresses must be computed for existing members at hanger connection locations. If shear and bending stresses produced by the construction live load do not exceed 3000 psi, it is not necessary to compute total stresses. If stresses produced by the construction live load exceed 3000 psi, total stresses from dead load, vehicle load plus impact and construction load must be computed. The AASHTO HS-20 truck should be used for computing vehicle live load stresses. Total stresses, including construction live load, must not exceed 150% of AASHTO inventory allowable stresses.

H. In addition to the wire rope hangers, timber posts will be installed at locations where it is possible to have uplift from wind.

I. Diagonal sway brace ropes and railings will be installed at all edges of platforms that are not connected to the walls.

3.1.7 PLATFORMS

3.1.7.1 TEMPORARY PLATFORMS BELOW BRIDGES (SAMPLE NOTES) (21)

3.1.8 VEHICULAR (22)

3.1.8.1 ELEVATED ROADWAYS AND RAMPS

Elevated roadways shall be designed for all loadings, including seismic effects, in accordance with the latest AASHTO Standard Specifications for Highway Bridges or latest AASHTO LRFD Bridge Design Specifications and the relevant state DOT standards.

The loading for the departure and arrival ramps servicing airport passenger terminals shall be HS 20-44 AASHTO highway loading per the latest AASHTO Standard Specifications for Highway Bridges or HL93 AASHTO Vehicular Live Loading per the AASHTO LRFD Bridge Design Specifications. All other ramps servicing cargo facilities or road overpasses shall be designed for HS 25 AASHTO highway loading per the AASHTO Standard Specifications for Highway Bridges or HL93 AASHTO Vehicular Live Loading per the AASHTO LRFD Bridge Design Specifications.

3.1.8.2 PABT BUS LOADING 2008 (23)

3.1.8.3 GWBBS BUS LOADING 2008 (24)

3.1.8.4 GWB (SAMPLE RFP) WIM & FATIGUE ANALYSIS

Expert Professional Services for the George Washington Bridge Main Span Upper Level Structural Steel Rehabilitation:

Truck Load Determination and Life Cycle Analysis

This task shall be performed concurrently with the previous tasks, as appropriate.

The Consultant shall submit a plan to execute the following required items for this study (Items 1 through 8, below). The submission schedule is in Section IV of this document.

The Consultant shall perform the following engineering analyses to determine the remaining useful life of the GWB Upper Level orthotropic deck under a) HS-25 vehicle loading and b) actual (site-specific) vehicular loading and the most economical remediation strategies. Today, the GWB is subject to higher and more frequent overload conditions than those assumed in the original design of the orthotropic deck.

A. Gather any available vehicular traffic data for the Upper Level to establish a baseline of the vehicular loading throughout the past life of the orthotropic deck. Data may include toll classification counts, which indicate the number of axles on trucks, and Weigh-in-Motion (WIM) data already collected from adjacent roadways.

B. Perform a temporary Weigh-in-Motion (WIM) study to supplement existing vehicular usage data. The purpose of the study is to determine current truck traffic loadings and frequencies. Perform temporary video surveillance to establish multiple presence patterns (i.e. how frequently trucks travel side-to-side). Establish the site-specific truckload and multiple presence criteria for the GWB Upper Level roadway.

C. Perform a limited field inspection of the upper level deck and its supporting steel to verify problem areas documented in recent condition survey reports. (Note: this may be combined with field inspections.)

D. Perform a finite element analysis of an orthotropic deck panel under a) HS-25 vehicle loading and b) the observed site-specific vehicle loading and take into account fatigue effects.

E. Perform a load test of an orthotropic deck panel with strain gauges to validate the finite element analysis. The Consultant shall develop the load test procedure, procure the necessary equipment, install the equipment and perform the load tests.

F. Project the remaining useful life of the deck based on available past life loading history, along with a) HS-25 vehicle loading and b) the observed site-specific vehicle loading, and

G. Perform an economic analysis to determine the best remediation/replacement scenario. Perform life cycle cost analyses in accordance with NCHRP Report 483 methodology taking into consideration Agency costs and User (the traveling public) costs. Order of magnitude cost estimates are acceptable). For each scenario, estimate costs and remaining useful life of the deck and perform a life cycle cost analysis based on a) HS-25 vehicle loading and also for b) the actual (site-specific) vehicular loading. Scenarios might include:

1. Do priority repairs only and replace the deck in “X” years.
2. Do a comprehensive retrofit and enhancement of the orthotropic deck and superstructure steel and replace the deck in “Y” years. For the purpose of this study, describe the nature and extent of the retrofit and enhancement and deck replacement concepts.
3. Replace the deck now with a more robust construction for greater expected life.

H. Prepare and submit 50% draft, 90% draft and 100% complete copies of reports, summarizing the above study for review. All calculations shall be included in the Appendix of the Report.

3.1.9 SUSTAINABLE DESIGN GUIDELINES (25)

3.1.9.1 STRUCTURAL FIRE PROTECTION (26)

Aircraft loading walkways (airports) design loads and the worse case loads combination shall be prescribed in NFPA 415, Chapter 6.

3.1.10 BRIDGE SAFETY RECOMMENDATIONS

The National Transportation Research Board (NTSB) in collaboration with the Federal Highway Administration (FHWA) is currently investigating the cause of the August 1, 2007, collapse of a 1000-foot section of the 1,900-foot long Interstate 35W (1-35W) highway bridge over the Mississippi River in Minneapolis, Minnesota. Approximately 110 vehicles were on the bridge at the time, 13 people died and 145 people were injured. The bridge's roadway deck was supported by a steel truss, not unlike other bridges built during the 1960s.

Although the investigation is ongoing, and no determination of probable cause has been reached, an interim report, *"Adequacy of the U10 & U11 Gusset Plate Designs for the Minnesota Bridge No. 9340 (1-35W) over the Mississippi River,"* was issued on January 11, 2008. The concern of the investigators related to the design of the gusset plates, which are steel plates that connect two or more members of a truss structure at a joint, prompted the NTSB to issue "Safety Recommendations," dated January 15, 2008. Highlights of the interim report and safety recommendations memorandum are as follows:

- ❒ The 1-35W bridge is considered "fracture critical" because the load paths were non-redundant, i.e., failure of any primary structural element in the bridge would cause collapse.
- ❒ There was apparently an error in the original design of some gusset plates resulting in them being roughly half the required thickness. Since the original design calculations for the gusset plates could not be found, it is not known whether the error was in the calculations, drafting or some other design process. Unquestionably, there was some breakdown in the design process and in the design QA/QC process.
- ❒ Bridge owners frequently perform load-rating calculations to determine if their bridges can accommodate heavy vehicles. The computer programs commonly used to perform load-rating analyses typically check the stresses in truss members, but not gusset plates. The assumption being that the gusset plates were properly sized in the first place and have greater capacity than the truss members. It is, therefore, highly unlikely that an error in the original design of the gusset plates would be found by performing typical load rating analyses after the fact.
- ❒ Since the time that the bridge was built, it had undergone at least two major rehabilitations, including an increase to the thickness of the deck and addition of median and outside barrier walls. These rehabilitations significantly added to the weight of the bridge.
- ❒ The 1-35W Bridge was inspected biennially as required by the FHWA. The FHWA's National Bridge Inspection Standards (NBIS) are aimed at detecting conditions such as cracks and corrosion. They do not, and are not intended to, check for errors in the original design.
- ❒ At the time of the collapse, roadway construction was being conducted. Machinery and paving materials were being stockpiled on the center span.

The NTSB has no evidence that the deficiencies in the design process related to the 1-35W Bridge are widespread, however, they have recommended that the owners of non-redundant load path bridges conduct load capacity calculations to verify that the stress levels in all structural elements, including gusset plates, remain within acceptable limits whenever planned modifications or operational changes may significantly increase stresses.

In response to the NTBS Safety Recommendations memorandum, implement the following enhanced actions:

- ❒ Perform load capacity calculations for all major PANYNJ bridges, including the GWB (suspension), Bayonne (arch), Goethals (cantilever truss), and Outerbridge (cantilever truss). Primary structural elements, including gusset plates, check considering their current state and service conditions. These analyses will be performed as the bridges receive their biennial condition survey evaluations, and they will be completed within a three-year time frame.
- ❒ Audit the size and thickness of primary structural elements, including gusset plates, to verify that the sizes installed are consistent with the original design dimensions and those assumed above. This audit will also be performed during the respective biennial condition survey.
- ❒ The Engineering Department has an initiative underway to further enhance our QA/QC procedures. While our existing procedures are sound, we are taking extra steps to clarify and systematize our QA/QC methodology. Engineering Department designers and our consultants will be instructed that, for any construction work, which results in a significant

increase in stress in structural elements, including gusset plates, a structural analysis and verification of the safe load carrying capacity of these affected elements will be required. It should be noted that this instruction is not new, but rather the instructions will be reiterated to all designers. Furthermore, such instructions will be included in our design guidelines/standards currently being redrafted.

- ❒ Designers have been reminded that they must consider likely construction operations, which may involve temporary overload of structural elements and specify appropriate restrictions, if necessary, on the contract drawings.
- ❒ The Departments' construction inspectors will continue to review contractor construction procedures and operations, which may result in an increase of stress in structural elements, with the contractor and with the designer of record, if necessary.

3.1.11 NYDOT TECH ADVISORY – TRUSS GUSSET PLATE ANALYSIS

3.1.12 GUIDELINE FOR DESIGN LOAD INFORMATION REQUIRED ON CONSTRUCTION DOCUMENTS FOR BUILDINGS

Effective immediately, all construction documents for buildings shall include design loads and other information pertinent to structural design in conformance with the code listed for the jurisdiction below.

Jurisdiction	Code
New York City	New York City Building Code
New Jersey	New Jersey Uniform Construction Code, its bulletins and the sub-codes with their Supplements and Reference Standards (International Building Code, New Jersey Edition)
New York	Building Code of New York State

Information pertinent to the structural design shall include live loads, roof snow loads, wind design data, earthquake design data, flood loads and flood design data, and special loads. Live loads shall include machinery, equipment, and other concentrated loads in excess of 1,000 lbs. along with footprint of load or support layouts.

The construction documents shall also include superimposed floor dead loads such as, but not limited to, ceilings, hung utilities, wear courses, future wear courses, waterproofing, partitions, and finishes.

3.1.13 CLIMATE RESILIENCY

All structural elements shall be designed in accordance with the flood protection levels provided in “Design Guidelines - Climate Resilience chapter” and with the appropriate codes for the flood elevation (including freeboard) established for a project for hydrostatic and hydrodynamic forces and shall be designed as required and appropriate for collision impact from debris, wave load, and buoyancy.

3.2 Design Criteria — Facility Specific

3.2.1 Design Live Loads for Existing Port Authority of New York & New Jersey Facilities

3.2.1.1 Instructions for Use of Design Live Loads

The loads listed are the known design live loads for existing Port Authority of New York & New Jersey facilities. The loads are the original live loads the existing structures were designed for except as noted herein.

For the Port Authority Bus Terminal (PABT), design bus live loads have been increased above the original bus loads to a 45-foot bus loading. Non-destructive testing in the 1990s revealed increased load capacities of the bus levels.

There are no documented design live loads (allowable uniform live loads) for the berths listed for Port Facilities in Subsection 3.2.1.7 unless otherwise noted. The design live loads provided for these berths are based on facility practice and maximum loads historically allowed on the berths.

Design live loads are listed in Subsection 3.2.1.9 for the four-major long span bridge structures of the Port Authority:

- George Washington Bridge
- Bayonne Bridge
- Outerbridge Crossing
- Goethals Bridge

In addition, note that the TB&T Structural Group processes overweight vehicles on a routine basis for all four of these bridge structures.

3.2.1.2 George Washington Bridge Bus Station

The third floor, bus level is approved for the loading of the 45-foot-long bus shown in Figure 3-A. See below for AASHTO H-15 truck loading.

AASHTO H-15 Truck Loading

Axle Location	Axle Load
Front Axle	14,400 lbs.
Drive Axle	20,000 lbs.
Tag Axle	10,000 lbs.
Total Weight = 44,000 lbs.	

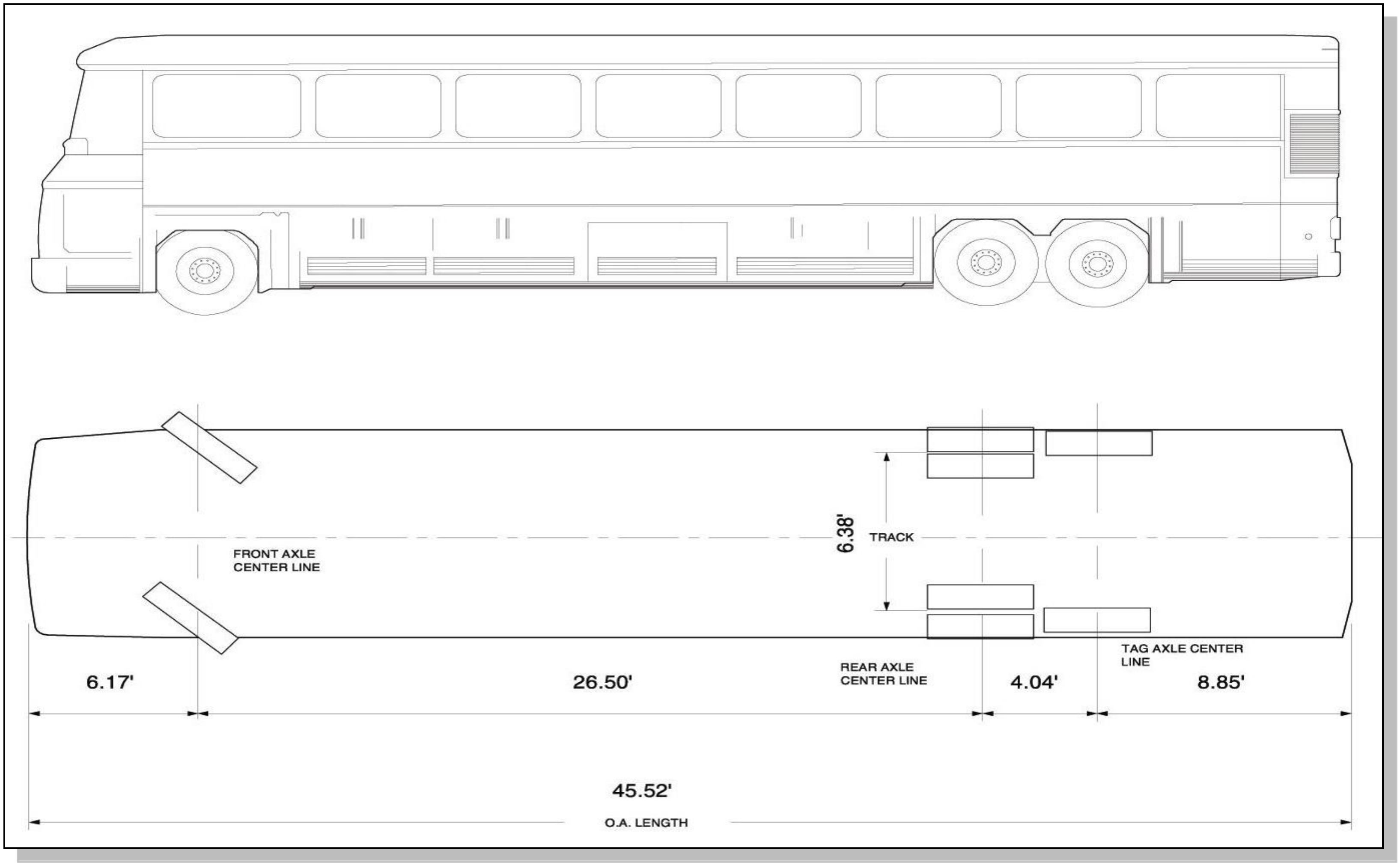
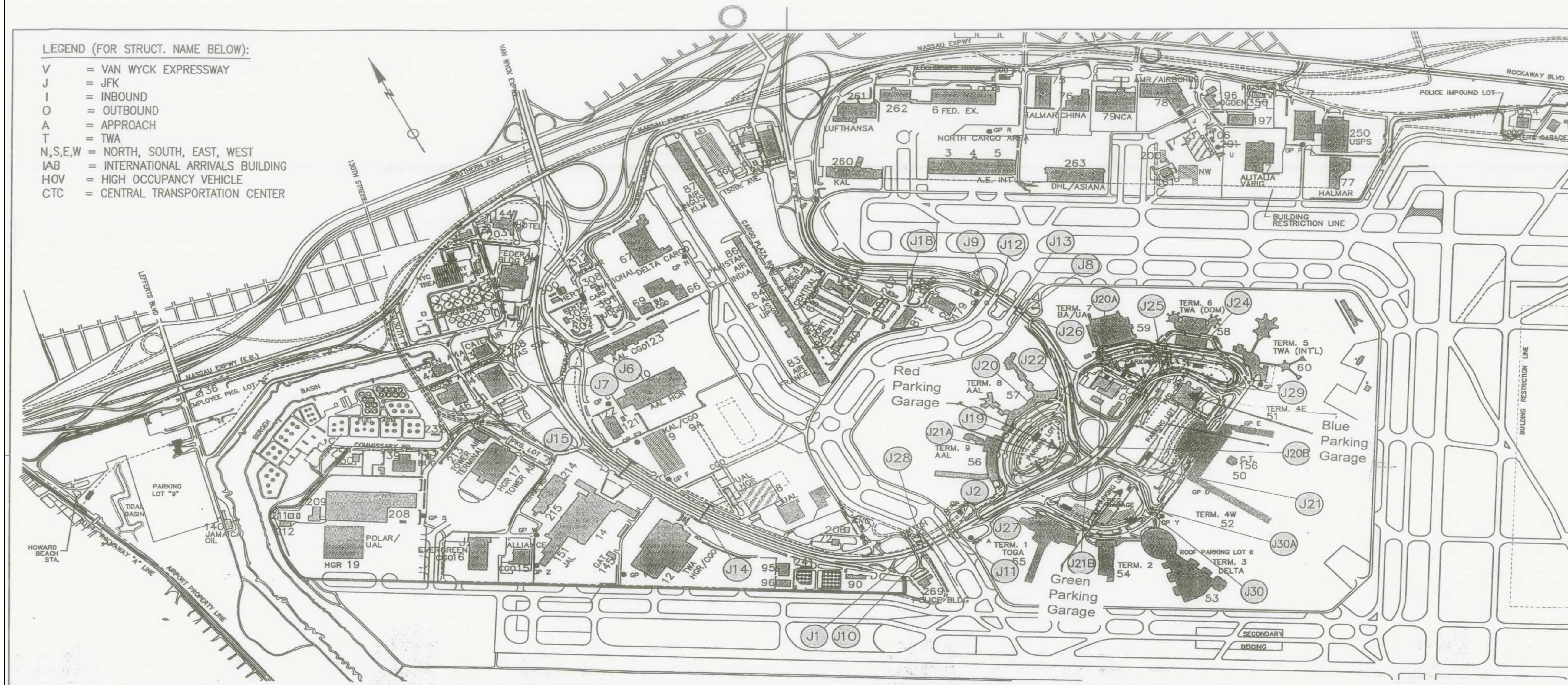


Figure 3-A
Proposed 45-Foot-Long Bus Axle Loads

3.2.1.3 JFK INTERNATIONAL AIRPORT

Bridge No.	Design Load	Remarks
J1	AASHTO HS20-44	
J2	AASHTO HS20-44	
J6	AASHTO HS20-44	
J7	AASHTO HS20-44	
J8	AASHTO HS20-44	
J9	AASHTO HS20-44	
J10		Taxiway Bridge Designed for NLA Aircraft (A380-800)
J11		Taxiway Bridge Designed for NLA Aircraft (A380-800)
J12		Taxiway Bridge Designed for NLA Aircraft (A380-800)
J13		Taxiway Bridge Designed for NLA Aircraft (A380-800)
J14		Pedestrian Bridge
J15		Pedestrian Bridge
J18	AASHTO HS20-44	
J19	AASHTO HS20-44	
J20	AASHTO HS20-44	
J20A	AASHTO HS20-44	
J20B	AASHTO HS20-44	
J21	AASHTO HS20-44	
J21A	AASHTO HS20-44	
J21B	AASHTO HS20-44	
J22	AASHTO HS20-44	
J23	AASHTO HS20-44	
J23A	AASHTO HS20-44	
J23B	AASHTO HS20-44	
J24	AASHTO HS20-44	
J25	AASHTO HS20-44	
J26	AASHTO HS20-44	
J27	AASHTO HS20-44	
J28	AASHTO HS20-44	
J29	AASHTO HS20-44	
J30	AASHTO HS20-44	
J30A	AASHTO HS20-44	
Blue Garage	Per NYC Building Code: LL = 50 psf Snow Load = 30 psf Designed for seismic	4-Story building, foundation has provision for a 7-story hotel
Green Garage		3-Story building
Red Garage		4-Story building

See Figure 3-B, JFK International Airport Roadway Bridge Location Plan.



LOCAL NUMBER	STRUCTURE NUMBER	NAME
J1	5522110	N. PERIMETER SERVICE RD. / V.W.E. EXT.
J2	5522140	S. PERIMETER SERVICE RD. / V.W.E. EXT.
J6	5522079	VAN WYCK EXP. / NORTH FED. CIRCLE
J7	5522089	VAN WYCK EXP. / SOUTH FED. CIRCLE
J8	5522190	S. PERIMETER SERVICE RD. / 150 TH. ST.
J9	5522220	N. PERIMETER SERVICE RD. / 150 TH. ST.
J10	5522121	TAXIWAY "O" / VAN WYCK EXP. EXT.
J11	5522122	TAXIWAY "I" / VAN WYCK EXP. EXT.
J12	5522210	TAXIWAY "O" / 150 TH. ST.
J13	5522200	TAXIWAY "I" / 150 TH. ST.

LOCAL NUMBER	STRUCTURE NUMBER	NAME
J14	5522100	EAST PED. BR. / VAN WYCK EXP. EXT.
J15	5522090	WEST PED. BR. / VAN WYCK EXP. EXT.
J18	5523870	SERVICE RD. / 150 TH. ST.
J19	5522640	AJO OVERPASS / JFK & VW RAMP
J20	5522650	VO OVERPASS / JFK RAMP
J20A	552265A	CTCWVO RAMP / JFK
J20B	552265B	IABVO / JFK
J21	5522230	VICTCE / JFK RAMP
J21A	552223A	VICTCE RAMP / JFK
J21B	552223B	VICTCE TO JICTCE RAMP / JFK

LOCAL NUMBER	STRUCTURE NUMBER	NAME
J22	5522470	IABJO / JFK
J24	5524320	VIT OVERPASS
J25	5524330	IABVO OVERPASS
J26	5524340	HOV OVERPASS
J27	5524590	HOV OVERPASS
J28	5524730	S-W RESTRICTED SERVICE ROAD
J29	5524720	DEP / ARR - PK2
J30	5524770	JI - ARR
J30A	552477A	VI - ARR RAMP

THE PORT AUTHORITY OF NY & NJ
Engineering Department
Quality Assurance Division

JFK
INTERNATIONAL AIRPORT
ROADWAY BRIDGE
LOCATION PLAN

BASED ON AVIATION JFK OPER-PL.DWG 10-2-98

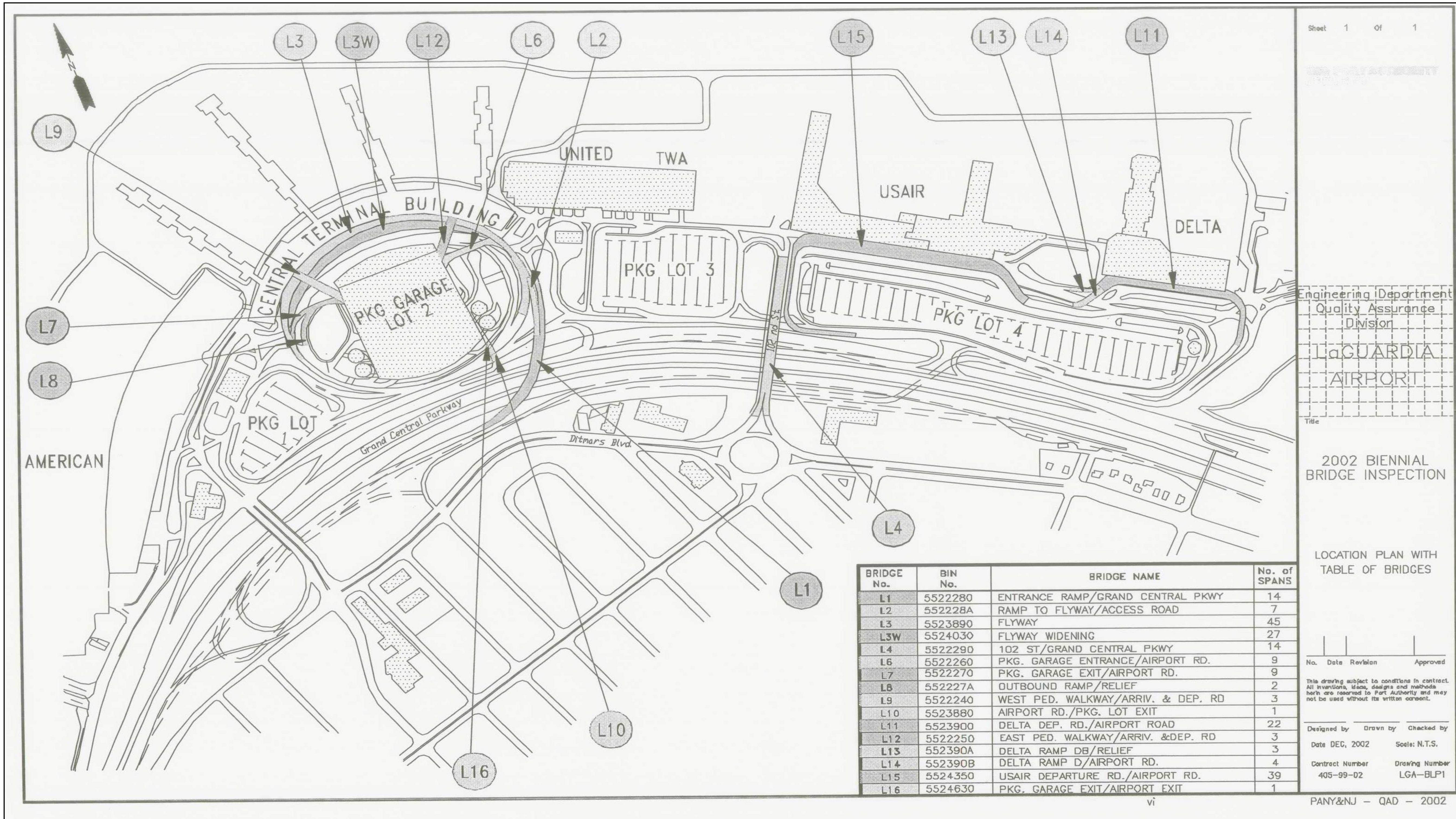
VI PANY NJ-QAD-2004

Figure 3-B
JFK International Airport Roadway Bridge Location Plan

3.2.1.4 LaGuardia Airport

Bridge No.	Design Load	Remarks
L1	AASHTO HS15-44	
L2	AASHTO HS20-44	
L3	AASHTO HS15-44	
L3W	AASHTO HS20-44	
L4	AASHTO HS20-44	
L6	AASHTO HS20-44	
L7	AASHTO HS20-44	
L8	AASHTO HS20-44	
L10	AASHTO HS20-44	
L11	AASHTO HS20-44	
L12		East Pedestrian Walkway/Arrival & Departure Road
L13	AASHTO HS20-44	
L14	AASHTO HS20-44	
L15	AASHTO HS20-44	
L16	AASHTO HS20-44	
Parking Garage	Per NYC Building Code: LL = 50 psf Snow Load = 30 psf Not designed for seismic	

See Figure 3-C, LaGuardia Airport Location Plan of Bridges



BRIDGE No.	BIN No.	BRIDGE NAME	No. of SPANS
L1	5522280	ENTRANCE RAMP/GRAND CENTRAL PKWY	14
L2	552228A	RAMP TO FLYWAY/ACCESS ROAD	7
L3	5523890	FLYWAY	45
L3W	5524030	FLYWAY WIDENING	27
L4	5522290	102 ST/GRAND CENTRAL PKWY	14
L6	5522260	PKG. GARAGE ENTRANCE/AIRPORT RD.	9
L7	5522270	PKG. GARAGE EXIT/AIRPORT RD.	9
L8	552227A	OUTBOUND RAMP/RELIEF	2
L9	5522240	WEST PED. WALKWAY/ARRIV. & DEP. RD	3
L10	5523880	AIRPORT RD./PKG. LOT EXIT	1
L11	5523900	DELTA DEP. RD./AIRPORT ROAD	22
L12	5522250	EAST PED. WALKWAY/ARRIV. &DEP. RD	3
L13	552390A	DELTA RAMP DB/RELIEF	3
L14	552390B	DELTA RAMP D/AIRPORT RD.	4
L15	5524350	USAIR DEPARTURE RD./AIRPORT RD.	39
L16	5524630	PKG. GARAGE EXIT/AIRPORT EXIT	1

Figure 3-C
LaGuardia Airport Location Plan of Bridges

3.2.1.5 PATH (OPEN AREA STRUCTURES)

Structure Name	Bridge Type	Design Live Load *	Comment
Passaic River Bridge (8.05B)	Rail	E63	Deck girders of tower spans all other remaining members
	Rail	E38	
Hackensack Rive Bridge	Rail		Conrail owned
Tichenor St. Crossing	Rail	E50	Amtrak owned
Pennington St. Crossing	Rail	E50	Amtrak owned
Chestnut St. Crossing	Rail	E50	Amtrak owned
Oliver St. Crossing	Rail	E50	Amtrak owned
East Kinney St. Crossing	Rail	E50	Amtrak owned
Cottage St. Crossing	Rail	E50	Amtrak owned
Walnut St. Crossing	Rail	E60	Amtrak owned
Elm St. Crossing	Rail	E60	Amtrak owned
Green St. Crossing	Rail	E60	Amtrak owned
Lafayette St. Crossing	Rail	E60	Amtrak owned
Foot Bridge to Sub Station #6	Pedestrian		PATH owned
Bridge 1½ (OG 2.35)	Rail	E40	Bridge owned by others
Van Wagen Avenue Foot Bridge	Pedestrian		Orphan bridge. PATH repair limited to conditions that impact our operations
Tonnelle Avenue Bridge	Vehicular		Orphan bridge. PATH repair limited to conditions that impact our operations
Summit Avenue Bridge	Vehicular		Orphan bridge. PATH repair limited to conditions that impact our operations
Baldwin Avenue Bridge	Vehicular		Orphan bridge. PATH repair limited to conditions that impact our operations
Chestnut Avenue Bridge	Vehicular		Orphan bridge. PATH repair limited to conditions that impact our operations
Waldo Avenue Foot Bridge	Pedestrian		Orphan bridge. PATH repair limited to conditions that impact our operations
PATH Flyover	Rail	E60	PATH owned
North Dock Trestle	Rail	E60	Bridge owned by others
South Dock Trestle	Rail	E60	Bridge owned by others
Bridge 6.16	Rail		Conrail Freight Line
Bridge 6.63	Rail		Amtrak Main
Wallis Avenue Bridge	Rail		Bridge owned by others

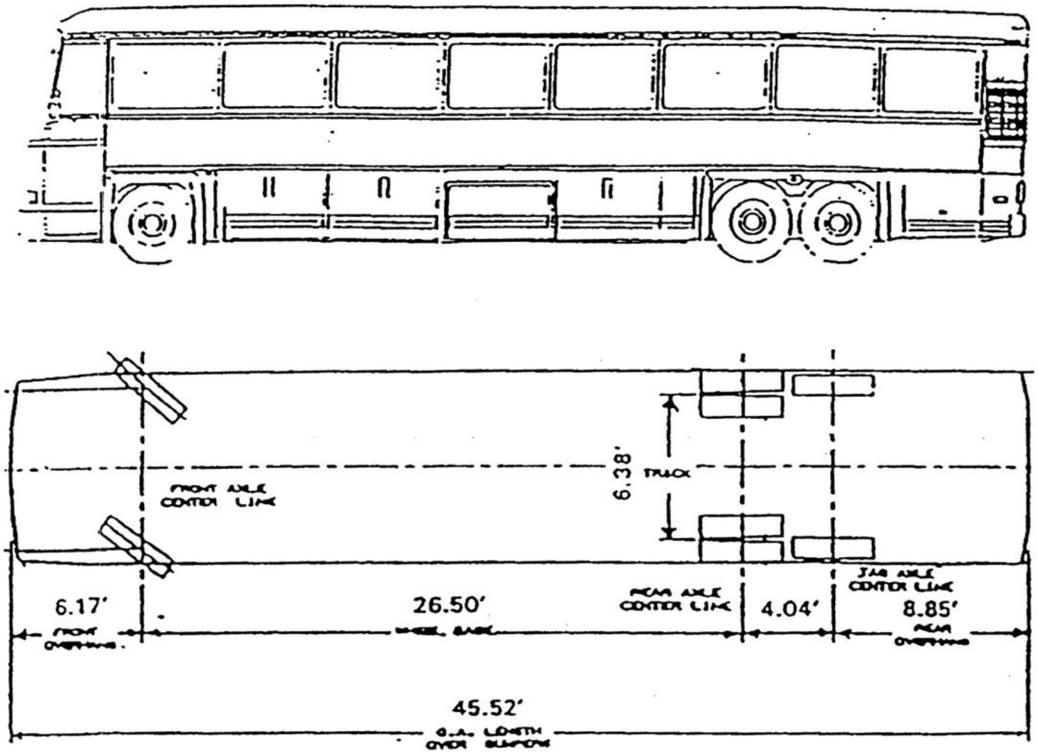
*** Add steam impact to Cooper live loading.**

3.2.1.6 PORT AUTHORITY BUS TERMINAL

Floor	North Wing Structure				South Wing Structure	
		Floor Construction	Live Load		Floor Construction	Live Load
					Car Parking Floors	
7th (Upper Level Parking)	3rd Entity Built in 1980			2nd Entity Built in 1963	Precast concrete channel slabs	50 PSF
6th (Intermediate Level Parking)						
5th (Lower Level Parking)						
		Bus Floor Area			Bus Floor Area	
4th (Upper Bus Level)		3” LWC wearing surface membrane waterproofing 7” LWC slab 2” Metal deck Composite beam system	45’ Bus Plus 10% or 100 PSF	1st Entity Built in 1950	4” Wear course ¾” Asphalt plank Membrane waterproofing 7½” Stone concrete slab Non-composite beam system	45’ Bus Plus 10% or 88 PSF
3rd (Suburban Bus Level)						
		(Public) Pedestrian & Store Area			(Public) Pedestrian & Store Area	
2nd (Suburban Concourse & Subway Mezzanine)		½” Terrazzo finish 4” to 5” LWC slab (5,000 PSI) 1½” Metal deck Composite deck system	100 PSF		2½” Terrazzo finish 3” Cinder fill 4” Lightweight cinder concrete slab (1,000 PSI) Non-composite deck system	100 PSF
1st (Main Concourse)		2½” Terrazzo finish 4” LWC slab (5,000 PSI) 1½” Metal dec Composite deck system				

Notes:

1. LWC denotes Lightweight Concrete
2. Only typical floor construction and live loading is given in this table. Floor construction, dead loading, and live loading at individual areas may vary.
3. See Figure 3-D for live loading of 45’ bus plus 10%.



Gross Vehicle Weight	Axle Loads	Axle Loads Plus 10%
Front Axle (lbs.)	14,400	15,840
Drive Axle (lbs.)	20,000	22,000
Tag Axle (lbs.)	10,000	11,000
Total (lbs.)	44,400	48,840

Figure 3-D

PABT Bus Loading

3.2.1.7 PORTS

3.2.1.7.1 Port Newark Marine Terminal

Berth No.	Allowable Uniform Live Load (PSF)	Remarks	References
2	500		
3	500		
4	500		
5	500		
6	500		
7	500		
8	1,000		PN-354.042 (2006). Dwg G004
9	500		
10	1,000	Portion only	PN-354.042 (2006). Dwg G004
11	500		
12	TBD		
13	500		
14	TBD		
15	500		
16	500		
17	500		PN-110.042 (1986). Dwg 2
18	500		
19	100		Inactive MFP-920.507 WO No. 8
20	500		PN-280.020 (1984). Dwg 2
21	TBD		
22	500		
23	Exceed 1,000		
24	500		
25	500		
26	500		
28	500		
30	500		
32	500		
34	500		
36	500		PN-800.219

3.2.1.7.2 Port Elizabeth Marine Terminal

Berth No.	Allowable Uniform Live Load (PSF)	Remarks	References
50	TBD		
51	TBD		
52	TBD		
53	TBD		
54	TBD		
55	TBD		
56	500		TAA EP-0249-LC-U28 (2007). Berth 56 to 62 Modernization Project
57	500		
58	500		
59	500		
60	500		
61	500		
62	500		
63	500		PN-800.219
64	TBD		
66	TBD		
68	TBD		
70	TBD		
72	TBD		
74	TBD		
76	TBD		
78	TBD		
80	TBD		
82	500	For half of Berth 82	TAA EP-248-LC-A3 (2004). Wharf Improvements-Phase 5. Other loads shown on Dwg P5-G3
84	500		
86	500		
88	500		TAA EP-248-LC-A1 (2001). Wharf Improvements-Phase 1. Other loads shown on Dwg P1-G3
90	500		
92	500		TAA EP-248-LC-A2 (2007). Wharf Improvements-Phase 2. Other loads shown on Dwg P2-G3
94	500		TAA EP-248-LC-A4 (2007). Wharf Improvements-Phase 3 and 4, Berths 94, 96, 98, and Turntable
96	500		
98	500		

3.2.1.7.3 Howland Hook Marine Terminal

Berth No.	Allowable Uniform Live Load (PSF)	Remarks	References
North Ext.	500		HH-334.009
Existing	250		
South Ext.	500		HH-334.009

3.2.1.7.4 Auto Marine Terminal

Berth No.	Allowable Uniform Live Load (PSF)	Remarks	References
1	TBD		
2	TBD		

3.2.1.7.5 Red Hook Marine Terminal

Berth No.	Allowable Uniform Live Load (PSF)	Remarks	References
5	TBD		
6	TBD		
7	TBD		
8	TBD		
9A	500		
9B	TBD		
10	TBD		
11	TBD		
12	TBD		

3.2.1.8 NEWARK LIBERTY INTERNATIONAL AIRPORT

				Inventory Rating							Operating Rating						
Local No.	Struct. No.	No. of Spans	Control Element	HS20-44 (36 Tons)	H-20 (20 Tons)	HS-15 (27 Tons)	H-15 (15 Tons)	Type 3 (25 Tons)	Type 3S2 (40 Tons)	Type 3-3 (40 Tons)	HS20-44 (36 tons)	H-20 (20 Tons)	HS-15 (27 Tons)	H-15 (15 Tons)	Type 3 (25 Tons)	Type 3S2 (40 Tons)	Type 3-3 (40 Tons)
N1	3800033	1	Beam 6, for moment	53				50	64	73	88				84	107	122
N2	3800034	2	Span 2 beam 7, for moment	45				43	54	62	76				72	91	103
N3	3800035	1	Beam G13, for shear	61				55	81	107	102				92	135	178
N4	3800036	3	Shear cap beams, for shear	36				40	42	44	60				66	70	74
N5	3800037	1	Beam 2, for moment	50				47	62	72	83				79	103	120
N6	3800038	3	Shear cap beams, for shear	25				25	28	29	42				42	46	49
N7	3800039	3	Shear cap beams, for shear	29				29	31	33	48				48	51	55
N8	3800040	3	Shear cap beams, for shear	32				32	35	38	51				51	55	60
N9	3800041	3	Span 3 interior beam, for shear	41				40	85	81	68				67	142	135
N10	3800042	3	Span 2 interior beam (on ramp), for shear	38				37	68	94	63				62	114	156
N11	3800043	3	Span 3 interior beam, for shear	38				36	54	64	63				60	90	107
N12	3800044	3	Span 2 interior beam, for shear	41				43	63	82	68				72	106	136
N13	3800045	3	Span 2 interior beam, for shear	41				39	72	83	69				64	121	138
N14	3800046	2	Stringer S21, for moment	39				38	44	48	65				63	74	80
N15	3800047	2	Stringer S30, for moment	39				38	45	48	65				64	74	81
N16	3800048	3	Stringer S8, for moment	32				31	36	40	53				52	61	66

Note: Control element based on HS20 – Inventory Rating – Load Factor Method
( ) = Working Stress Method

				Inventory Rating							Operating Rating						
Local No.	**Struct. No.**	**No. of Spans**	**Control Element**	**HS20-44 (36 Tons)**	**H-20 (20 Tons)**	**HS-15 (27 Tons)**	**H-15 (15 Tons)**	**Type 3 (25 Tons)**	**Type 3S2 (40 Tons)**	**Type 3-3 (40 Tons)**	**HS20-44 (36 tons)**	**H-20 (20 Tons)**	**HS-15 (27 Tons)**	**H-15 (15 Tons)**	**Type 3 (25 Tons)**	**Type 3S2 (40 Tons)**	**Type 3-3 (40 Tons)**
N17	3800049	3	Stringer S21, for moment	27				26	33	38	45				43	55	62
N18	3800050	25	Cellular structure No. 1, fascia beam, for shear	34				32	42	48	57				55	70	80
N19	3800051	30	Cellular structure No. 3, fascia beam, for shear	33				32	42	48	55				53	69	80
N20	3800052	30	Cellular structure, fascia beam, for shear	34				32	42	48	57				55	70	80
N21	3800053	12	Various 27”-deep beams, for shear	39				38	51	61	65				63	85	102
N22	3800054	13	Various 27”-deep beams, for shear	36				32	47	53	60				54	78	88
N23	3800055	12	Various 33”-deep beams, for shear	41				36	46	66	69				60	76	110
N24	3800056	3	Fascia beam (original section), for neg. moment	40				46	54	58	67				76	91	96
N25	3800057	3	Span 2 interior beam (widened section), for shear	38				37	48	55	64				61	80	91
N26	3800058	4	Fascia beams on spans 2 & 3, for moment	42				37	58	75	71				62	97	120
N27	3800059	9	Timber cap beam, for shear	35				41	72	84	47				56	98	113
N29	3800061	3	Stringer S11, for moment	31				29	37	41	51				48	61	69
N38	3800065	2	Typical for negative moment	30				28	37	45	50				62	57	75
N39	3800066	2	Typical for negative moment	30				28	37	45	50				62	57	75
N40	3800074	2	Typical for negative moment over the pier	36				38	53	54	61				43	88	90
N42	**3800080**	**8**	**Girder 1 - Span 5 (bottom flange)**	38				64	67	70	64				106	112	116

Note: Control element based on HS20 – Inventory Rating – Load Factor Method
( ) = Working Stress Method

3.2.1.9 TB&T BRIDGES

3.2.1.9.1 George Washington Bridge

Live Load					Live Load Reduction Equation	Impact
Type of LL		Lb. per Square Foot	Lb per Linear Foot	Lb per Linear Foot		
1	Roadway (80 ft.)	**250**	**-**	**20,000**	L = 46000 x (0.5+(2/(n+3))) x (0.2+(160/(200+l)))	I = (150/(200+l) x 4/(3+n)) x L
4	Electric Railway Tracks	**-**	**6,000**	**24,000**		
2	Sidewalks (10 ft.)	**100**	**-**	**2,000**		
Total Unreduced P =				**46,000***		

* Using the Live Load Reduction Equations, the Design Live Load for 14 lanes loaded on the center span is 8,000 pounds per linear foot and 28,000 Kips total for the 3,500-foot span.

Note: All live loads of a length 1,500 feet or less or the stresses there from shall be increased by an impact allowance of "I"
n = Number of loaded lanes of traffic
l = Loaded length of bridge
K = 0.2 + 160/(200+l)
C = 0.5 + 2/(n+3)
1 Kip = 1,000 pounds

3.2.1.9.2 Bayonne Bridge

Live Load					Live Load Reduction Equation	Impact
Type of LL		Lb. per Square Foot	Lb per Linear Foot	Lb per Linear Foot		
2	Rapid Transit Lines	**-**	**6,000**	**12,000**	L = 23200 x (0.5+(2/(n+3))) x (0.2+(160/(200+l))) L = 3671 + (71.113/(4.8424+m))	I = (150/(200+l) x 4/(3+n)) xl = 2.1156/(4.8424+m) x L
4	Roadway Lanes	**-**	**2,500**	**10,000**		
2	Sidewalks	**-**	**600**	**1,200**		
Total Unreduced P =				**23,200***		

* Using the Live Load Reduction Equations, the Design Live Load for 4 lanes loaded on the center span is 5,250 pounds per linear foot and 8,800 Kips total for the 1,675-foot span.

Note: All live loads of a length 1,500 feet or less or the stresses there from shall be increased by an impact allowance of "I"
m = Number of loaded panels (a panel is the bay between two consecutive floor beams)
n = Number of loaded lanes of traffic
l = Loaded length of bridge
K = 0.2 + 160/(200+l)
C = 0.5 + 2/(n+3)
1 Kip = 1,000 pounds

3.2.1.9.3 Outerbridge Crossing

Live Load					Live Load Reduction Equation	Impact
Type of LL		Lb. per Square Foot	Lb per Linear Foot	Lb per Linear Foot		
1	Roadway (42 ft.)	**250**	**-**	**10,500**	L = 12375 x (0.5+(2/(n+3))) x (0.2+(160/(200+l)))	I = (150/(200+l) x 4/(3+n)) x L
2	Sidewalks (9.375 ft.)	**100**	**-**	**1,875**		
Total Unreduced P =				**12,375***		

* Using the Live Load Reduction Equations, the Design Live Load for 4 lanes loaded on the center span is 3,600 pounds per linear foot and 2,700 Kips total for the 750-foot span.

Note: All live loads of a length 1,500 feet or less or the stresses there from shall be increased by an impact allowance of "I"
n = Number of loaded lanes of traffic
l = Loaded length of bridge
K = 0.2 +160/(200+l)
C = 0.5 + 2/(n+3)
1 Kip = 1,000 pounds

3.2.1.9.4 Goethals Bridge

Live Load					Live Load Reduction Equation	Impact
Type of LL		Lb. per Square Foot	Lb per Linear Foot	Lb per Linear Foot		
1	Roadway (42 ft.)	**250**	**-**	**10,500**	L = 12427 x (0.5+(2/(n+3))) x (0.2+(160/(200+l)))	I = (150/(200+l) x 4/(3+n)) x L
2	Sidewalks (9.635 ft.)	**100**	**-**	**1,927**		
Total Unreduced P =				**12,427***		

* Using the Live Load Reduction Equations, the Design Live Load for 4 lanes loaded on the center span is 3,800 pounds per linear foot and 2,600 Kips total for the 672-foot span.

Note: All live loads of a length 1,500 feet or less or the stresses there from shall be increased by an impact allowance of "I"
n = Number of loaded lanes of traffic
l = Loaded length of bridge
K = 0.2 +160/(200+l)
C = 0.5 + 2/(n+3)
1 Kip = 1,000 pounds

3.2.1.10 AIRCRAFT LOADING WALKWAYS

Aircraft loading walkways (airports) design loads and the worse case load combinations shall be as prescribed in NFPA 415, Chapter 6.

3.2.2 JFK

3.2.2.1 JFK AIRCRAFT DESIGN LOADING (27)

3.2.3 PABT

3.2.3.1 PABT TENANT GUIDELINES (28)

3.2.3.2 PABT (DESIGN FEATURES & GUIDELINES, STANDARDS, PRECAUTIONARY, FIELD WELDING, ETC. - AL BOCH REPORT) (29)

3.2.3.3 SPECIAL REQUIREMENTS FOR UNDERDECK ATTACHMENTS AT THE SOUTH WING

Floors in certain areas of the Port Authority Bus Terminal South Wing are of lightweight low-strength concrete construction (Figure 3-E). Concrete anchors are not permitted in these lightweight slabs for the attachment of hangers for supporting ducts, utilities, ceilings, and other miscellaneous loads. These loads shall be supported directly from floor beams or supplementary framing connected to the floor beams.

In areas where concrete inserts are permitted, only approved-type stainless-steel anchors rated for shock and vibration loads and elevated temperature shall be used.

3.2.3.4 PABT FLOOR CONSTRUCTION AND LIVE LOADING (30)

3.2.4 GWBBS

3.2.4.1 GWBBS REDEVELOPMENT PROJECT (SAMPLE DESIGN CRITERIA) (31)

3.2.5 PATH

3.2.5.1 NY FACILITY BOUNDARIES REQUIRING PA REVIEW FOR CRANE PERMITS (32)

3.2.5.2 PATH TRACK MAINTENANCE STANDARDS MANUAL (1980) (33)

3.2.5.3 PATH BOLT REPLACEMENT

3.2.5.3.1 Guide for Tunnel Bolt Replacement

The guideline is intended to be used by the Field Engineer to direct and inspect bolt replacement work in the cast iron ring sections of the PATH tunnels. It is not intended for the contractor's use.

A. Responsibilities of the Field Engineer

1. Interpret the bolt replacement criteria to direct the contractor as to which bolts will be replaced.
2. Perform close up inspection of all bolts using flat car provided by PATH.
3. May determine a special sequence or method of bolt replacement to be followed by the contractor within the scope of the guidelines.
4. Record all replacement work performed.

5. Ensure that contractor adheres to the specifications for all bolt replacements.
6. Notify Engineer of any unsafe conditions encountered in the field.
7. Other normal responsibilities for the contract construction phase as defined in the contract book.

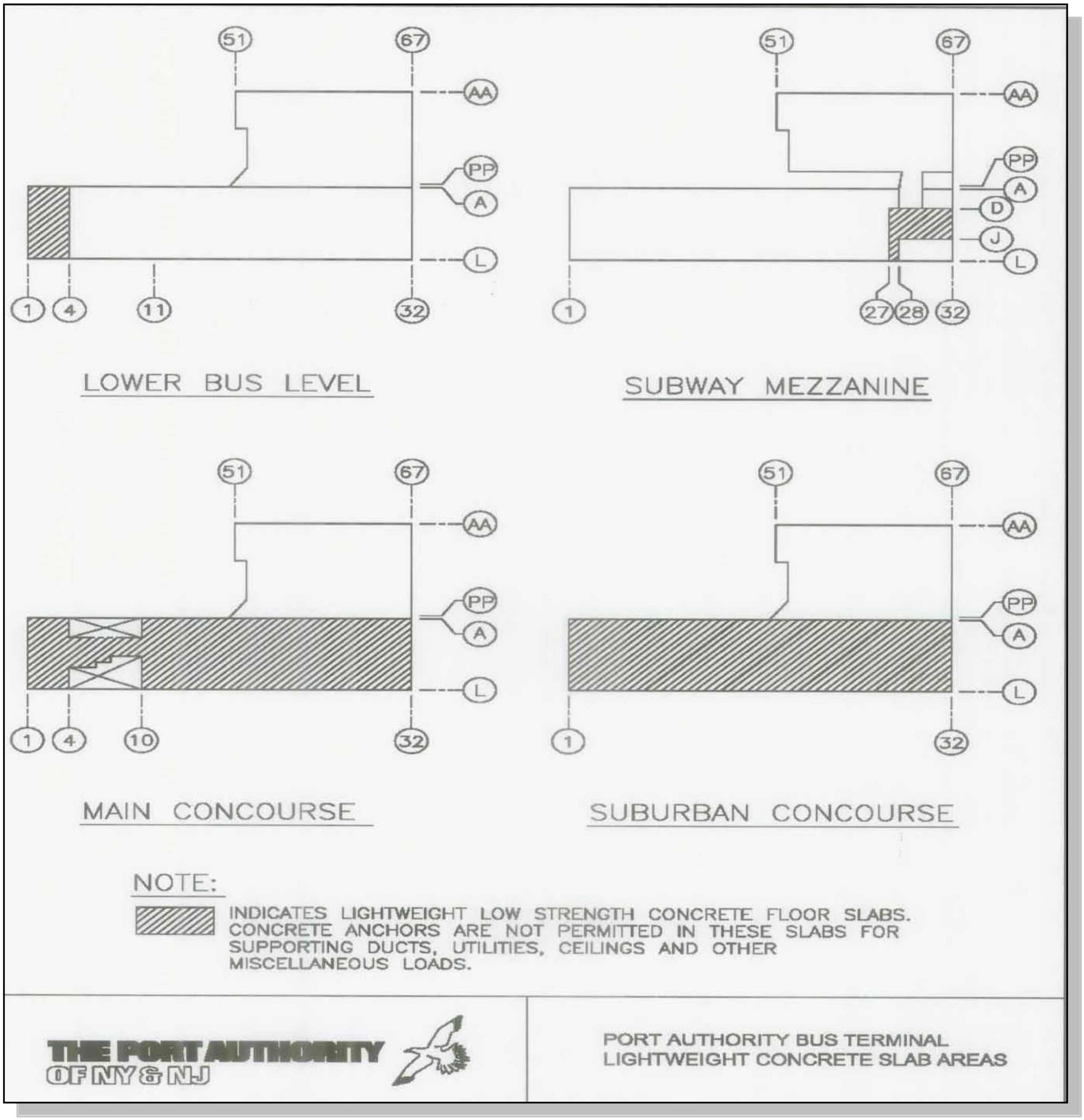


Figure 3-E
Port Authority Bus Terminal Lightweight Concrete Slab Areas

B. General Inspection Guideline

All bolts will be visually inspected from within arm's distance from the flat car in order of the schedule of work and in accordance with the methods contained herein. All bolts will be closely inspected for loss of metal and will be replaced by the specified bolts.

1. Inspection Method

    a. Cleaning—Areas to be inspected will be cleaned before commencement of inspection.

    b. Visual—Heavily corroded bolts need only visual inspection.

    c. Measurement—The inspector should use template or gages to measure bolt head dimensions.

3.2.5.3.2 General Criteria for Nut and Bolt Replacement

The contractor will replace only those nuts and bolts that are identified by the Engineer in the field. The following guidelines will be followed by the inspector to locate and mark bolts for replacement.

A. Replace all missing nuts and bolts. Where alignment of the flanges does not permit a 1¼" diameter bolt, use a bolt size that will fit.

B. Replace nuts and bolts as follows:

1. Bolts where the head and/or nut of the corroded bolt has bearing loss of more than 50%, as directed by the Engineer.
2. Bolts where the head and nut of the bolt are so corroded that the bearing areas is less than 50%.
3. Bolts where the head and/or nut are severely corroded and there are leaks through the cast iron lining flanges or bolt holes.
4. Short bolts – 50% or more of nuts thread not engaged.
5. Bolts where the head exhibits a loss of 50% or more of its projection from the shank at any point. Similar for nuts.

C. In all cases replace the complete assembly, nut, bolt, and washers.

D. Tighten loose nuts in accordance with the method contained herein.

E. Bolts shall be made "Snug Tight," i.e., a tightness obtained by the full effort of a man using an ordinary spud wrench.

F. Record Replacement.

G. Identify locations at which work is performed by the contractor.

3.2.6 LGA

3.2.6.1 AIRCRAFT RESTRICTIONS ON LAGUARDIA AIRPORT RUNWAY EXTENSIONS

Dated 09/05/2007

Aircraft	Allowable Operating Weights (Pounds)				Speed Restrictions (Knots)		Comments
	Take-Off		Landing		Straight Taxiing (Parallel to Runways) on Taxiways; Turning on Runways	Turning and Taxiing at 45 or 90 Degrees (to Runways) on Taxiways	
	Gross Weight @ MAC	Max. Main Gear Load	Gross Weight @ MAC	Max. Main Gear Load			
Airbus							
A300 (all series) Standard bogie	290,000 @ 24%		270,000				
A300 (all series) 38.5” x 60” bogie	298,000 @ 24%		280,000				
A310-200	284,640 @ 23%		271,170				
A310-300							
A319	155,204	71,336	134,500	62,700	15	5	
A320-200	162,037 @ 43%		142,195 @ 45%				Tire pressure not greater than 205 psi
A320-211							
A321-200	188,600 @ 29%	87,350	166,200 @ 38.4%	80,580			
A321-211							
BAE Systems							
Bae-146-200	93,500		81,000		14	5	
Boeing							
B727-100F	169,500		137,500		10	5	
B727-200	176,000 @ 20%	81,830	161,000		10	5	
B737-300	135,500		116,000		12	6	
B737-400	143,000		121,000		12	6	

Aircraft	Allowable Operating Weights (Pounds)				Speed Restrictions (Knots)		Comments
	Take-Off		Landing		Straight Taxiing (Parallel to Runways) on Taxiways; Turning on Runways	Turning and Taxiing at 45 or 90 Degrees (to Runways) on Taxiways	
	Gross Weight @ MAC	Max. Main Gear Load	Gross Weight @ MAC	Max. Main Gear Load			
B737-500	139,000		110,000		12	6	
B737-600	144,000		120,500		12	6	
B737-700	154,500		129,200		12	6	
B737-800 B737-900	173,000 @ 12% to 21.4% varying linearly to 168,600 @ 31%	80,000	146,300		12 (Taxiing on T/W) 15 (Turning on R/W)	6	
B757-200	255,000 @ 19.1% varying linearly to 243,800 @ 34.1%	113,200	198,000	93,000	20	9	
B757-212 B757-232 B757-26D	241,000	113,200	198,000	93,000	20	9	
B767-200 B767-232	312,000 @ 32%	144,800	296,000	142,600	10	5	
B767-300 B767-332ER B767-3P6ER B767-324 B767-332 B767-36N	317,00 @ 23%	144,800	296,000	142,600	10	5	
B767-400ER	370,000 @ 7% to 26.5% varying linearly to 360,500 @ 37%	172,000	323,000	148,000	10	5	

Aircraft	Allowable Operating Weights (Pounds)				Speed Restrictions (Knots)		Comments
	Take-Off		Landing		Straight Taxiing (Parallel to Runways) on Taxiways; Turning on Runways	Turning and Taxiing at 45 or 90 Degrees (to Runways) on Taxiways	
	Gross Weight @ MAC	Max. Main Gear Load	Gross Weight @ MAC	Max. Main Gear Load			
Bombardier							
CRJ-200LR	53,250		47,000		14	5	
CRJ-700 (all series)	82,750		73,500		14	5	
Embraer							
Embraer 145	162,000 @ 12.1%		144,000		12	6	
Embraer 175	85,870		74,957		14	5	
Embraer 190	114,199		97,003		12	6	
ERJ 170ER	82,365		72,312		14	5	
Fairchild-Dornier							
Dornier 328-300	34,524		31,724		12	6	
Fokker							
F-100	98,500						
Lockheed							
C141B	257,900	122,500	250,800	119,100	10	5	
L1011-1	364,000 @ 21%	165,600	358,000		None	None	
L1011-500	355,200 @ 25%	164,300	348,000		None	None	
McDonnell Douglas							
DC-9-15	90,700		81,700				

Aircraft	Allowable Operating Weights (Pounds)				Speed Restrictions (Knots)		Comments
	Take-Off		Landing		Straight Taxiing (Parallel to Runways) on Taxiways; Turning on Runways	Turning and Taxiing at 45 or 90 Degrees (to Runways) on Taxiways	
	Gross Weight @ MAC	Max. Main Gear Load	Gross Weight @ MAC	Max. Main Gear Load			
DC-9-30	173,000 @ 12.1%		144,000		12	6	
DC-9-31	105,000		95,300		14	5	
DC-10-10 DC-10-30	366,600 @ 16%	163,200	347,800		None	None	Center gear must be retracted at all times
MD-80 (all series)	160,000		139,500		14	5	
MD-90-30 MD-90-30ER	168,500 @ -1% varying linearly to 159,050 @ 30%	76,680	142,000		14	5	

3.2.6.2 LGA DECK ANALYSIS FILES (GT STRUDL REQUIRED) (34)

3.2.6.3 STEEL DECKING – CONCRETE ANCHORS & ATTACHMENTS

In the LaGuardia Airport Central Terminal Building, concrete anchors or attachments to the steel decking are not permitted for connection of hangers for ducts, utilities, ceilings, bracing, signage, and other miscellaneous loads in areas where the floor is constructed of non-structural concrete fill over cellular decking (Figure 3-F). All such loads in these areas shall be directly supported from the existing structural floor framing or supplementary framing shall be provided to transfer the loads to the existing framing members.

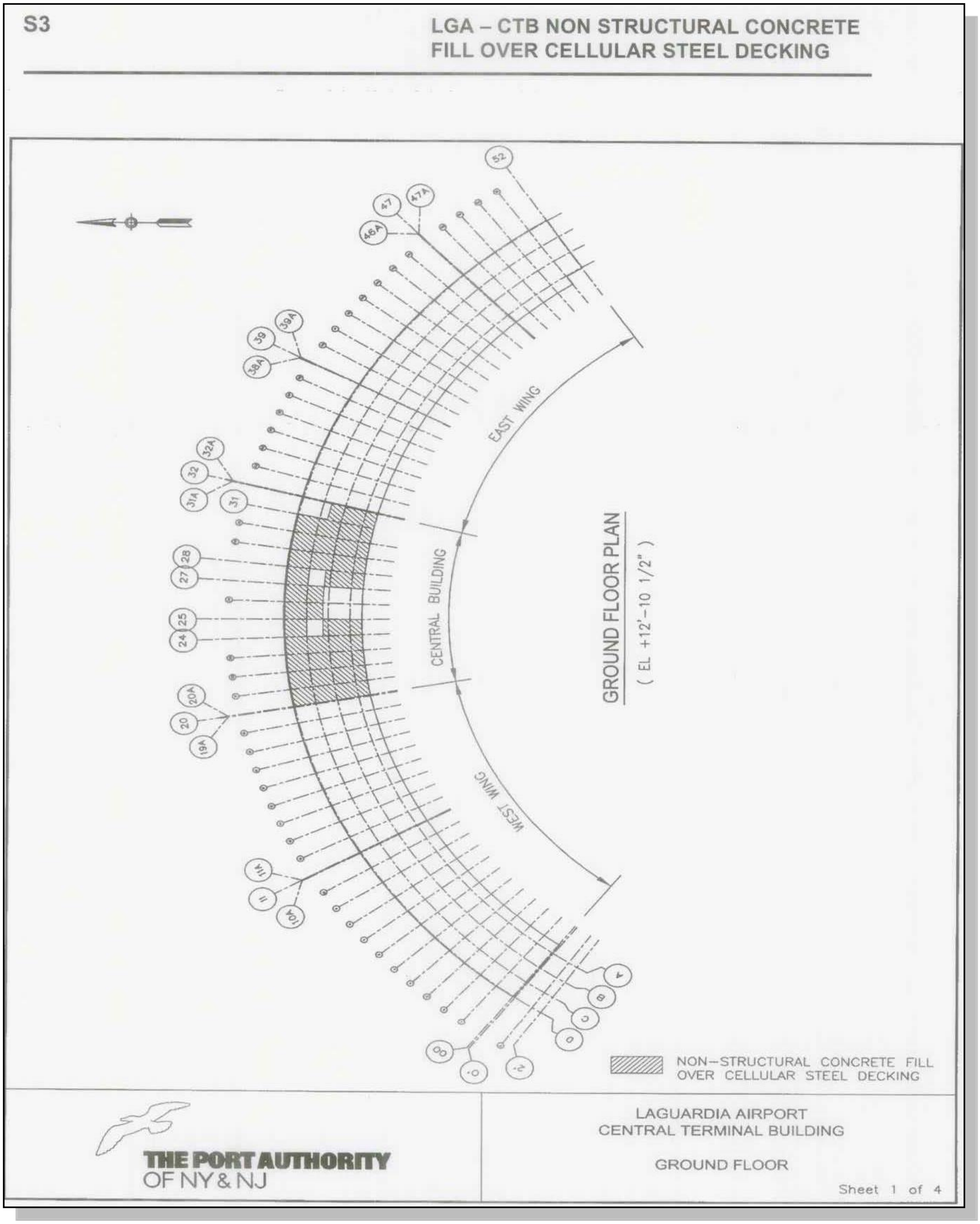


Figure 3-F
LaGuardia Airport Central Terminal Building

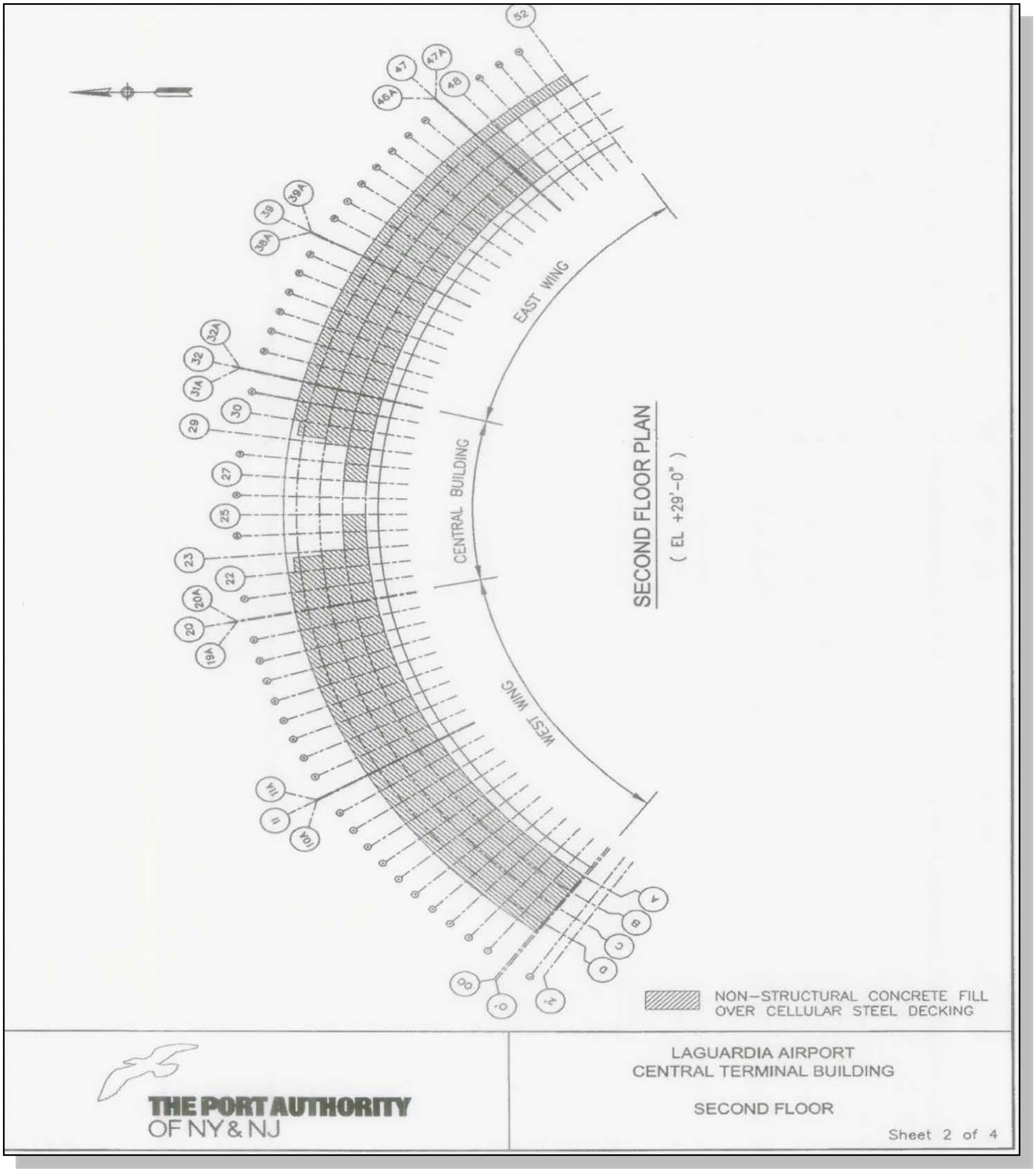


Figure 3-F, Cont'd.
LaGuardia Airport Central Terminal Building

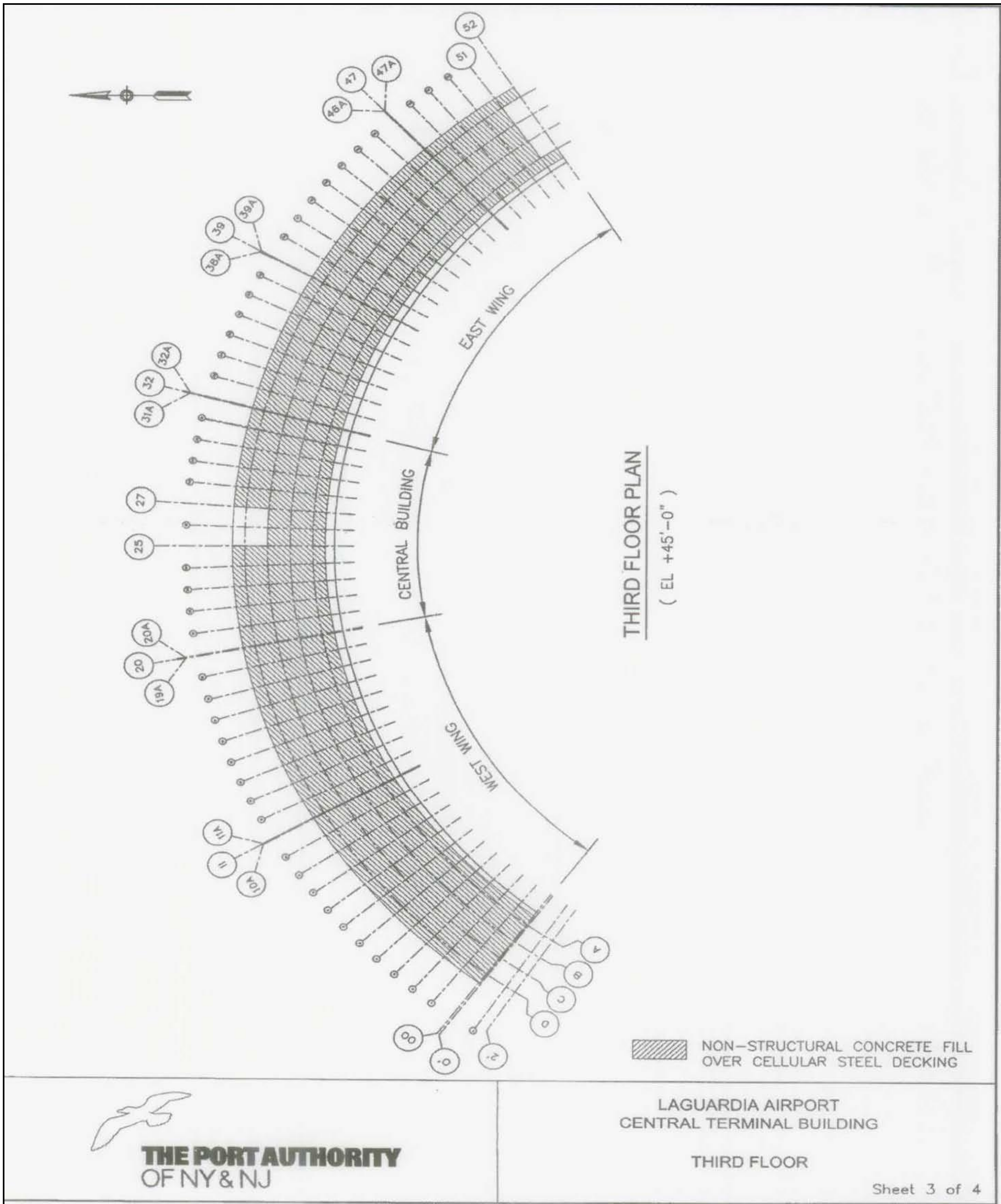


Figure 3-F, Cont'd.
LaGuardia Airport Central Terminal Building

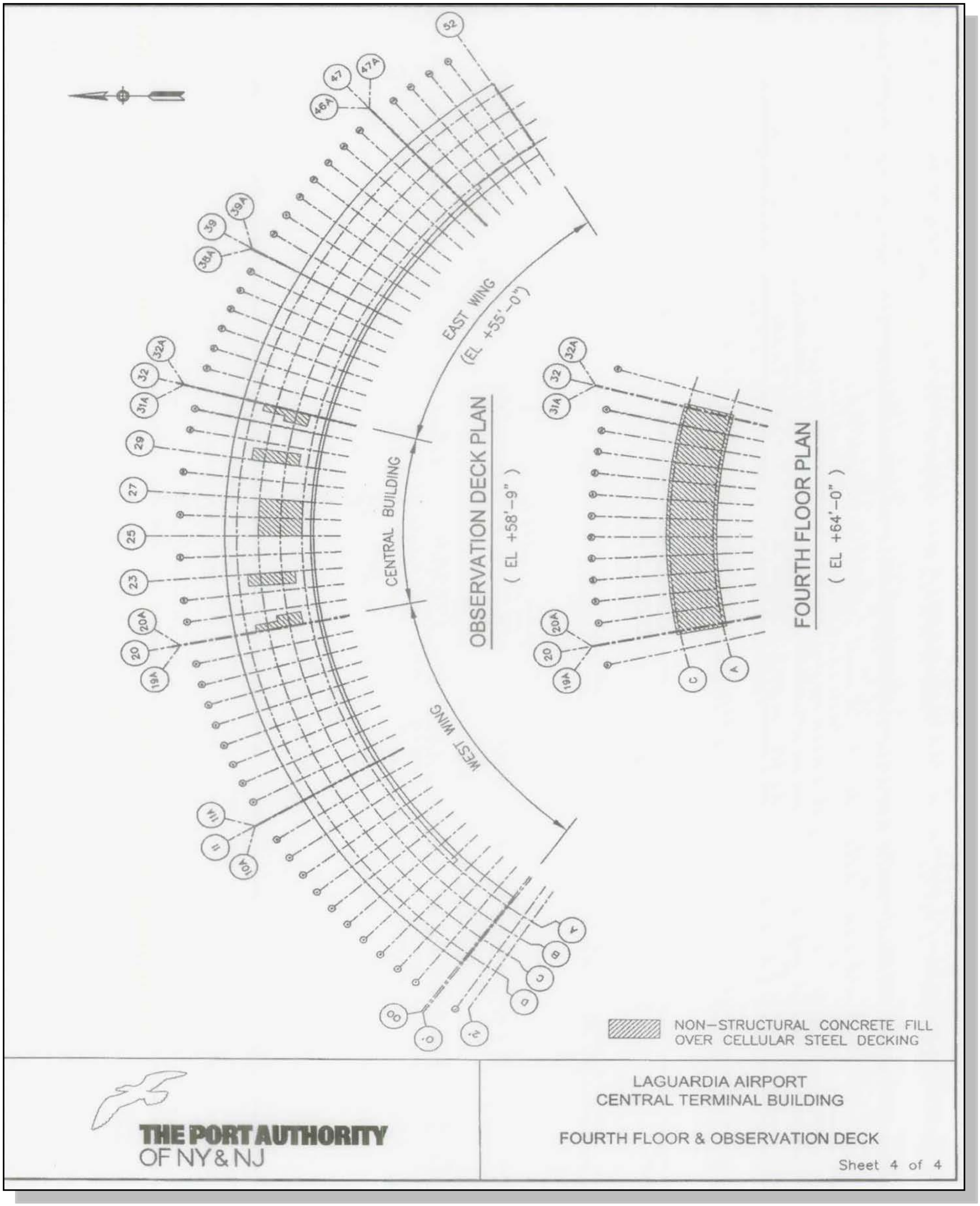


Figure 3-F, Cont'd.
LaGuardia Airport Central Terminal Building

3.2.7 GOETHALS BRIDGE

3.2.7.1 GOETHALS BRIDGE MODERNIZATION (SAMPLE GEOTECHNICAL CRITERIA [2007])

The preliminary geotechnical criteria for designing the proposed bridge foundations are based on previous site investigation together with design analyses performed in 1996 for the Goethals Bridge Expansion (i.e., “twinning”). Additional site investigation will be required during subsequent design stages to determine on-site nature and sequence of the strata at locations not defined by prior boring information. The bridge will be supported by foundation piers to rock, drilled shafts, and concrete filled pipe piles. The preferred method of support will depend on the stratigraphic sequence encountered at localized pier locations.

CODES AND STANDARDS

The latest provisions of the following codes, interims, and standards shall be applicable:

- ❒ AASHTO LRFD Bridge Design Specifications, with current Interims (LRFD)
- ❒ AASHTO Standard Specifications for Highway Bridges
- ❒ NYSDOT Seismic Design Specifications
- ❒ The Port Authority of New York & New Jersey Standard Specifications
- ❒ NYSDOT Bridge Manual, LRFD
- ❒ NJDOT Bridge and Structures Design Manual LRFD

DESIGN METHODOLOGY

Selection of foundation geotechnical capacities for sizes, number of caissons or piles, and evaluation of foundation deformation will be based on Service Load design criteria using the AASHTO Standard Specifications.

Foundations

Allowable capacities for piles, caissons, and piers will be developed during Stage II/Phase I. For preliminary design, the following values shall be assumed:

A. Allowable Bearing Capacity of Rock – 20 ksf

B. Caisson (Drilled Shaft):

1. Axial Capacity—To develop preliminary estimates of caisson capacity, we ignore shaft friction for the overburden soil above the bedrock and assume all loads to be resisted by the side shear of rock socket.
2. Design rock socket shear capacity = 60 psi and Ultimate shear capacity = 120 psi, to be confirmed by load tests.
3. Uplift Capacity—As opposed to compression loading, in which the transfer of load from the shaft tends to compress and close the joints in the rock, tension loading on the shaft tends to open the joints and may result in failure occurring along joint planes rather than as a shear failure along the socket. The uplift capacity of drilled shafts was calculated based on the weight of a wedge of rock around the shaft, the weight of soil above the wedge, and the weight of the shaft.

   The shaft uplift capacities used for preliminary design are summarized below:

   - ❒ New York: Design capacity = 8 psi and Ultimate capacity = 16 psi.
   - ❒ New Jersey: Design capacity = 4 psi and Ultimate capacity = 8 psi.

4. Lateral Capacity—The critical failure mode for lateral capacity is governed, as in tension capacity, by failure along joints and bedding planes rather than by the strength of the intact rock. In examining the rock cores obtained for prior project, it was noted that the predominant joint plane in the rock was inclined at approximately 20° to 25° to the horizontal. Based on this geometry, the required depth of embedment of the rock sockets was determined by applying the calculated horizontal shear and bending moment at the top of rock (based on the structural analyses) to the shaft. The required depth of embedment that would provide passive resistance in front and in back of the shaft that satisfies horizontal force and moment equilibrium was then determined. In earlier studies, the width of the passive wedge was estimated to be about 18 feet or equal to the spacing between the shafts, whichever was less.

   The following additional considerations will be included in the analysis:

   a. The moments at the top of rock were developed considering the actual point of application of the structural load and the lateral soil flow loads. Magnitude of loading will depend on the size of foundations, the magnitude of design earthquake, and will be discussed in the final report.

   b. Depending on the depth and spacing of the shafts in a given row, there may be insufficient room to develop the full passive wedge in front of the second and third row of shafts. In this case, a modified analysis of the passive pressure based on a truncated wedge that extends only to the next row of shafts would be required.

   c. Capacity of Driven Piles (Concrete Filled Steel Filled Pipe Pile).

Axial Capacity

Pile Diameter (in)	Wall thickness (in)	Capacity, tons (design/ultimate)
20	0.5	200/400
30	0.5	300/600

Uplift Capacity

Pile Diameter	Wall Thickness	Uplift Capacity (tons)			
		NY		NJ	
		Design	Ultimate	Design	Ultimate
20	0.5	40	80	30	60
30	0.5	60	120	45	90

Lateral Capacity (Assumed fixed head condition)

Pile Diameter	Wall Thickness	Lateral Capacity (tons)			
		NY		NJ	
		Design	Ultimate	Design	Ultimate
20	0.5	12	24	25	50
30	0.5	25	50	40	80

The above capacities shall be confirmed by load test.

Earth Pressures

A. Above Grade Retaining Walls

1. Criteria for design of retaining walls under static and seismic load conditions are given in the Guidelines and Parameters for Analysis and Design of Retaining Walls and Abutments table.
2. For retaining walls supporting inclined backfills, an analysis considering the site-specific geometry shall be done.

B. Design Ground Motions

The initial time histories to be spectrum matched shall be representative actual recorded motions, which are scaled, as needed, to the approximate level of rock spectrum in the period range of significance for the structure's response.

Sets of ground motion time-history components shall be selected from no fewer than 3 earthquakes contained in the NYSDOT seismic criteria for a critical bridge. Their duration shall be consistent with the magnitude, fault distance, and source characteristics of the design earthquake. Their amplitude shall be scaled using the following procedure:

For each pair of horizontal ground motion components, the 5% damped response spectrum for each component shall be created. An SRSS component for that earthquake shall be constructed by taking the square root of the sum of the squares of the two horizontally orthogonal components. Then an ensemble spectrum shall be formed by taking the average of the SRSS spectra for the individual earthquakes. The ensemble earthquake shall be frequency scaled so that it does not fall below 1.4 times the 5% damped response Safety Evaluation/Design Level NYSDOT design spectrum for a critical bridge in the range of 0.2 T to 1.5 T seconds; where T is the natural period of the fundamental mode of the structure. The scale factor required to do so shall be applied to the individual ground motion components.

Site response analyses along the bridge alignment, using a one dimensional column of site specific soil shall be conducted to develop varying free field motions at pier and tower foundations.

To enable a multiple support excitation analysis of the bridge, the time histories shall be phased horizontally. In addition to the site response analysis, above, the spatially varying motions shall consider, as a minimum, the following factors:

- ❒ Wave passage effect.
- ❒ Wave scattering incoherency effect.
- ❒ Soil shear strength degradation and liquefaction, where applicable.
- ❒ Attenuation of shaking amplitude with distance from the earthquake source.

❒ Cross correlation issues between the two orthogonal horizontal components.

At least 3 three component time history sets, representing the SEE, and 1 three component time history set representing the FEE shall be developed for use in the nonlinear dynamic analysis of the bridge. The effect of directionality from the expected source shall be taken into account.

C. Design Response Spectrum

The design response spectrum shall be structure specific in contrast to the conventional mudline surface spectrum. It shall be developed at the pile-cap level using an equivalent linear SSI analysis, which incorporates the kinematic interaction between soil and foundation, taking into consideration the pile stiffness, the soil stiffness and the pile group layout.

The response spectra shall be developed for both, the horizontal and the vertical ground motions for the FEE and the SEE event.

The response spectra shall be developed with the assumption of 5% damping for concrete components.

D. Liquefaction and Lateral Ground Movement

An evaluation of the potential for and consequences of soil liquefaction within near surface soil shall be made.

1. Liquefaction Potential Analysis—Numerical modeling procedures involving the use of nonlinear dynamic effective stress methods, such as the one-dimensional effective stress site response analysis performed by the program DESRA, shall be used to evaluate the potential for liquefaction.

   Soil springs (p-y, t-z, q-u) shall be furnished to enable dynamic analysis in both, the liquefied configuration and the non-liquefied configuration for both the FEE and SEE events.

2. Lateral Ground Movement Analysis—Numerical modeling procedures that evaluate the liquefaction- related permanent ground movement including lateral spreading, lateral flow and dynamic settlement, shall be used. Methodology for Lateral Spread Impact Assessment shall be implemented. The effects of altered site conditions, such as fills and the beneficial effects of “pinning” piles shall be considered.

   Mitigation of liquefaction effects shall be addressed either in the form of soil improvement or acceptance of predefined inelasticity in the piles. (**CRITERIA TO BE DEVELOPED**)

   Liquefaction and liquefaction-related ground movement shall be treated as separate and independent load cases.

Allowable Height of New Embankments will be evaluated according to thickness of organics and previous surcharge history at specific locations.

Case I: Free standing abutments or retaining walls: sliding bearings

Case II: Abutments or retaining walls restrained by anchors or batter piles.

Utilities

Underground utilities such as storm and sanitary drainage lines and electrical conduits, in general may be installed without pile support. However, where these lines connect to pile supported structures, the connections should incorporate some flexibility to accommodate ground settlements, movements, and/or allow readjustment in the future. Gas line details shall be approved by the Port Authority of NY and NJ.

If any utility lines pass under new embankments, the effect of settlement on such utility lines should be evaluated. If the settlement is deemed to be unacceptable, the cost of relocating the lines versus the cost of shielding the lines from the effects of settlement would have to be evaluated.

Seismic Design

A. For loads on retaining walls, see Guidelines and Parameters for Analysis and Design of Retaining Walls and Abutments table.

B. For design load combinations refer to the Structural Design Criteria.

C. Use NYCDOT soil type criteria. A site-specific analysis will be performed to establish seismic parameters for liquefaction analysis, seismic earth loads, and design response spectrum. For preliminary design, prior to completion of the site-specific analysis, use Soil Type D for the New Jersey approaches and Soil Type E for the Main span and New York approaches.

D. Liquefaction evaluation shall be based on the Seed-Idriss simplified method (1983). The cyclic stress ratio may be determined based either on Seed's simplified equation or on the results of a "Shake" Type Analysis.

Guidelines and Parameters for Analysis and Design of Retaining Walls and Abutments

Parameter	Value
Soil Moist Unit Weight Soil Saturated Unit Weight	115pcf 120 pcf
Equivalent Fluid Weight (static analysis) Foundation stability, pile loads Retaining wall structure	 35pcf acts at 0.4H 55pcf acts at 0.4H
Equivalent Fluid Weight (seismic analysis) Case I Case II	 45pcf acts at H/2 55pcf acts at H/2
Seismic coefficient, kh (used in calculating inertia forces of abutment & block of soil above abutment base)	A/2 (Case I) 1.5A (Case II)
Pile Geotechnical Capacity for Seismic Loading: Axial Loading: Compression Uplift Lateral Loading	2.0 x design capacity. 1.5 x design capacity To be established based on allowable deflections but □□2.0 x design capacity

Cellular Cofferdam Protection Dolphin

A cellular cofferdam protection dolphin, if required, should be designed based on concepts of energy dissipation. The cofferdam should be capable of safely deflecting a sufficient amount to dissipate the energy of a ship impact. The design of the cofferdam should address the issue of long term durability and should include adequate factors of safety against:

- Vertical shear (Terzaghi)
- Horizontal shear (Cummings)
- Interlock tension
- External stability (sliding, overturning, bearing capacity)

Use of tie down anchors may be necessary to provide the required stability.

Every effort should be made to locate the main piers on land, or at locations where depth of water is shallow at high tide, to minimize vessel collision during inclement weather, or to prevent the bridge elements from being struck by uncontrolled ships.

3.2.7.2 GOETHALS BRIDGE MODERNIZATION (SAMPLE STRUCTURAL CRITERIA [2007])

The purpose of these design criteria is to document the specifications, design approaches, design methodology and governing criteria to be used for the design of a new Cable Stayed Goethals Bridge over the Arthur Kill waterway spanning between the City of Elizabeth, NJ and Staten Island, NY and its approaches structures. The design criteria is based on the basic design requirements and philosophies used by the PANYNJ, AASHTO, New Jersey DOT and New York State DOT, with criteria for the main span utilized by the profession for special and complex bridges. The specifications, design approach, design methodology, and governing criteria described here are applicable for the design of single or parallel Goethals Bridge structure(s) whether in New Jersey or New York. Where differences arise between the AASHTO LRFD provisions and the AASHTO Guide Specifications, the more stringent provisions shall be used. The NJDOT and NYSDOT specifications shall supersede the AASHTO provisions for the specific provisions. Where differences exist between the NJDOT and the NYSDOT provisions, the more stringent state specification shall be used. When other design differences exist between the specifications below, the PANYNJ shall provide final documented direction.

DESIGN SPECIFICATIONS

- ❒ AASHTO LRFD Bridge Design Specifications, Latest Edition, with current Interims (LRFD).
- ❒ For geotechnical capacities only - AASHTO Standard Specifications for Highway Bridges, Latest Edition
- ❒ AASHTO Guide Specifications for Design and Construction of Segmental Concrete Bridges, Latest Edition.
- ❒ ANSI/AASHTO/AWS Bridge Welding Code D1.5, Latest Edition.
- ❒ NYSDOT, Bridge Manual, LRFD, Latest Edition.
- ❒ NYSDOT, Division I-A of the Standard Specifications for Highway Bridges (Blue Book), Latest Edition with current amendments.
- ❒ NJDOT, Bridge and Structures Design Manual LRFD Latest Edition.
- ❒ AASHTO Guide Specifications for Seismic Isolation Design, Latest Edition.
- ❒ CEB/FIP Model Code for Concrete Structures, latest edition, Appendix E, Time-Dependent Behavior of Concrete, Creep and Shrinkage.
- ❒ Post-Tensioning Institute (PTI) Recommendations for Stay Cable Design, Testing, and Installation, Latest Edition.
- ❒ Post-Tensioning Institute (PTI) Guide Specification for Grouting of Post-Tension Structures, Latest Edition.
- ❒ For ice loading only - ANSI/ASCE, Minimum Design Loads for Buildings and Other Structures, Latest Edition.
- ❒ AREMA, Manual for Railway Engineering, Latest Edition.
- ❒ ACI, Analysis and Design of Reinforced and Prestressed Concrete Guideway Structures, ACI 358.1R-92, Latest Edition.

- ❒ NFPA 130 Standard for Fixed Guideway Transit and Passenger Rail Systems, Latest Edition.
- ❒ AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges, Latest Edition with current Interims.
- ❒ AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals, Latest Edition with current interims.
- ❒ AASHTO Guide Specification for Fracture Critical Non-Redundant Steel Bridge Members, Latest Edition with current Interims.
- ❒ Design of Bridge Deck Drainage, Publication No. FHWA-SA-92-010.
- ❒ FHWA Hydraulic Engineering Circular No. 18, Evaluating Scour at Bridges, FHWA-IP-90-017, Latest Edition.

UNITS

All structures shall be designed and detailed using the U.S. customary English units of Inches, Feet, Pounds, Kips, Tons, Degrees Fahrenheit, etc.

DATUM

- ❒ Vertical Datum is based on the National Geodetic Vertical Datum of 1929 (NGVD) of EL. 0.00.
- ❒ Coordinates shall be based on the State of New Jersey Mercator System (NAD 1983).

BRIDGE CLEARANCES

A. Navigational Clearance: A minimum horizontal navigational clearance of 500 ft and a minimum vertical navigational clearance of 135 ft from Mean High Water El. +3.09 (based on USC&G Mean Sea Level at Sandy Hook, NJ of. El. 0.0). These are the same clearances currently provided by the existing bridge.

B. Aviation Clearance: The pylons of the main span structures shall not exceed an elevation to be determined by the Port Authority an approved by the FAA. This elevation is to be determined based on the structure's proximity to Newark International Airport. Temporary exceedance during construction will require FAA permits.

C. Desirable vertical clearance on major roadways or access roads to/from HHMT: 16'-6".

D. Vertical clearance on local streets: 14'-6".

E. Vertical clearance for railway side tracks and industrial tracks without overhead electrification: 23'-0" per AREMA; vertical clearance for electrified LRT shall be determined based on the final system chosen and shall be no less than 16'-0" from top of rail to catenary. Catenary standoff shall be based on the voltage with a minimum 6" provided.

F. Railroad horizontal clearance: Piers less than 25'-0" from the centerline of railroad track require a crash-wall in accordance with the AREMA Manual.

ROADWAY GEOMETRICS

A. Roadway layout

1. 3-12-foot lanes
2. 12-foot right hand shoulder
3. 5-foot left hand shoulder

B. Sidewalk-bikeway layout

1. For single bi-directional bridge: 10-foot on north side of bridge
2. For dual parallel bridges: 10-foot on north side of northern most bridge only

C. LRT layout

1. 27-foot corridor (between barriers) for dual track LRT
2. 14-foot corridor (between barriers) for single track LRT

D. New Jersey type concrete barrier curbs shall be used. Typical widths shall be

1. 1'-6" at edge of roadways
2. 2'-0" for barriers separating directions of traffic
3. 1'-6" for barriers separating traffic from LRT or sidewalk-bikeways

BRIDGE ELEMENT DESIGN LIFE EXPECTANCY

Approach Spans	
Deck Slab Wearing Surface Overlay	25 Years
C.I.P Deck Slab with overlay	100 Years
C.I.P Deck Slab	100 Years
Pre-cast prestressed Deck Slab with overlay years	100 Years
Pre-cast prestressed Deck Slab	100 Years
Superstructure Elements	
(Concrete Box Girders, Precast Prestressed Concrete Girders or Steel Girders)	100 Years
Bearings	50 Years
Expansion Joints	25 Years
Piers and Foundations	150 Years

BRIDGE ELEMENT DESIGN LIFE EXPECTANCY, CONT'D.

Main Spans	
Deck Slab Wearing Surface	25 Years
C.I.P Deck Slab with overlay	75 Years
C.I.P Deck Slab	75 Years
Pre-cast Prestressed Deck Slab with overlay	100 Years
Pre-cast Prestressed Deck Slab	100 Years
Superstructure Elements	
(Concrete Box Girders, Precast Prestressed Elements or Steel Girders)	100 Years
Bearings	50 Years
Expansion Joints	25 Years
Stay Cables	100 Years
Towers, Piers and Foundations	150 Years

DESIGN METHODS

The bridge superstructure and column substructure elements shall be designed using AASHTO LRFD design specifications. Deflections, rotations, cables vibrations, sidewalk vibrations, accelerations, fatigue limits, concrete crack control, crack opening, etc., shall be checked to ensure serviceability of the bridge components. For segmental designs, Joint Type B (dry joints) shall not be permitted. Permanent (Stay-in-place - SIP) steel deck forms shall NOT be used. Precast concrete panels that are cast composite and used as forms are permitted and may remain, all forms shall be removed. Barriers and overlay wearing surfaces shall not be considered structural members.

Pile caps, piles, drilled shafts and caissons shall be designed as structural members using the AASHTO LRFD design specifications.

The capacity of piles, drilled shafts and caissons to transfer their load to the appropriate stratum shall be based on service load design methodology (AASHTO 17th Ed.). The loads used to verify the foundation capacities shall the AASHTO LRFD Bridge Specification loadings without the application of load and/or resistance factors.

DESIGN LOADINGS

Dead Loads

Self Weight	
Normal weight reinforced concrete and prestressed concrete	150 lb/ft^3
Normal weight of post tensioned concrete with reinforcing	155 lb/ft^3
Tremie concrete	145 lb/ft^3
Structural steel	490 lb/ft^3
Unit weight for earth	120 lb/ft^3
Timber treated	60 lb/ft^3

Dead Loads, Cont'd.

Superimposed Dead Load	
Future wearing surface (t=2”)	30 lb/ft^2
Concrete Parapets (ea.)	
1’-6” Single Side Jersey Barrie	460 lb/ft
2’-0” Double Sided Jersey Barrier	500 lb/ft
Transit and Fastening per Track (2 rails)	200 lb/ft
Utilities (TBD in Stage II – Phase II, minimum value provided)	1000 lb/ft
Pedestrian Railing	100 lb/ft
Light Rail Transit (LRT)	

Self-weight of track work and appurtenances and other secondary elements supported by the structure after construction of the basic structure shall be considered as superimposed dead load. Concrete pads (second pour) for direct fixation of LRT rails shall also be considered as superimposed dead load

LIVE LOADS

A. Vehicular

1. Standard truck or tandem and lane loads shall be AASHTO HL-93. All design live loads shall be positioned both transversally and longitudinally so as to produce the maximum influence on the structure in accordance with the AASHTO LRFD Design Specifications.

   Multiple presence factors shall be applied to highway lanes only. Up to two (2) LRT tracks and sidewalk loading shall be combined with highway traffic loading to produce the most critical loading condition in the members. Refer to section below for LRT loads. The multiple presence factor of highway lanes shall be in accordance with the AASHTO LRFD Design Specifications.

   Fatigue design shall be in accordance with AASHTO LRFD Specifications, where, p = 0.85. ADTT and ADTTSL shall be finalized in Stage II upon completion of 2004 traffic studies.

   Traffic Design Speed = 50 mph

   Dynamic Load Allowance (IM) shall be in accordance with AASHTO LRFD Design Specifications.

   a. Impact shall be applied to all superstructure elements, towers, stay cables and cable anchorages, bearings (except elastomeric), and substructure elements, (pier caps, piers and pile caps) not entirely below ground.

   b. Impact shall not be applied to foundation components that are entirely below ground level. This includes but is not limited to piles and drilled shafts.

   c. Impact shall not be applied to the Permit Vehicles.

2. Longitudinal Forces

   a. Highway braking forces shall be computed in accordance with AASHTO LRFD Design Specifications.

   b. See Section below for Light Rail Transit braking forces on tracks.

3. Permit Vehicle

Permit vehicles shall be in accordance with the AASHTO LRFD and their appropriate agency (NJDOT, NYSDOT). Two types of permit vehicles are defined, Routine and Special. These definitions are based on NCHRP 454 "Calibration of Load Factors for LRFD Bridge Evaluation."

B. Routine Permit Vehicle

PA Standard Permit Vehicle (to determine with database provided by PA). Structure to accommodate PA Standard Permit Vehicle with unrestricted use (no lane closure, etc.).

C. Special Permit Vehicle (Overload):

1. NJ Permit Vehicle

   a. The NJDOT LRFD Permit Vehicle and the NYSDOT LRFD Permit Vehicle. The permit vehicle shall be used only with the Strength II limit state calculation.

   b. When analyzing for the NJDOT Permit Vehicle, the configuration shall be centered across two design lanes, precluding an additional design vehicle from traveling in an adjacent lane. The permit vehicle shall only be considered once on a structure. The remaining design lanes shall be considered occupied by the HL-93 design vehicle.

c. In addition to the HL-93 analysis, a strength II limit state calculation shall be made for the following NJDOT permit vehicle configuration:

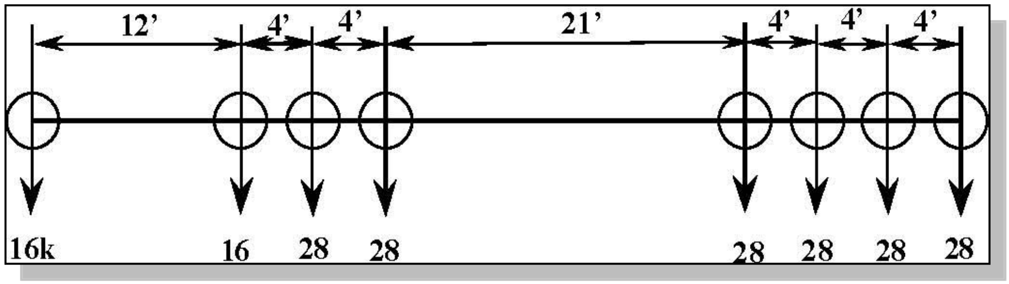


Figure 3-G
NJDOT LRFD Permit Vehicle, (200 Kips)

d. When analyzing for the NJDOT Permit Vehicle, the configuration shall be centered across two design lanes, precluding an additional design vehicle from traveling in an adjacent lane. The permit vehicle shall only be considered once on a structure. The remaining design lanes shall be considered occupied by the HL-93 design vehicle.

D. NY LRFD Permit Vehicle:

NY Permit Vehicle shall be in accordance with NYSDOT criteria.

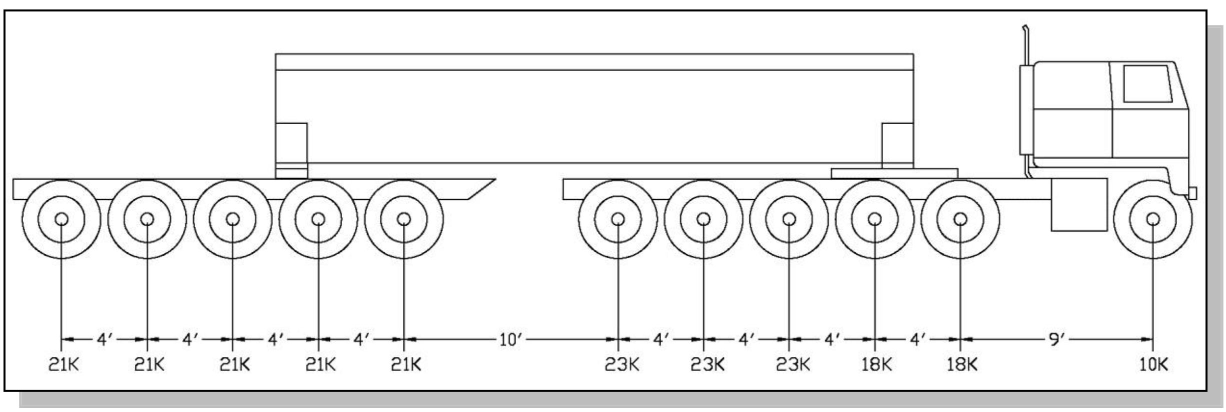


Figure 3-H
NYSDOT LRFD Permit Vehicle, (220 Kips)

E. Light Rail Transit System Loads:

For these design criteria, the Light Rail Vehicle (LRV) used was adopted from the NJ Transit Light Rail Manual of Design Criteria. The final LRV design loads are dependent on the manufacturer and shall be confirmed under later design stages. The assumed LRV has the following design load characteristics: 6 axles per car for a total weight of 147.6 kips. An LRV train may consist of one, two or three cars. Figure 3-I depicts the position and load of each vehicle axle.

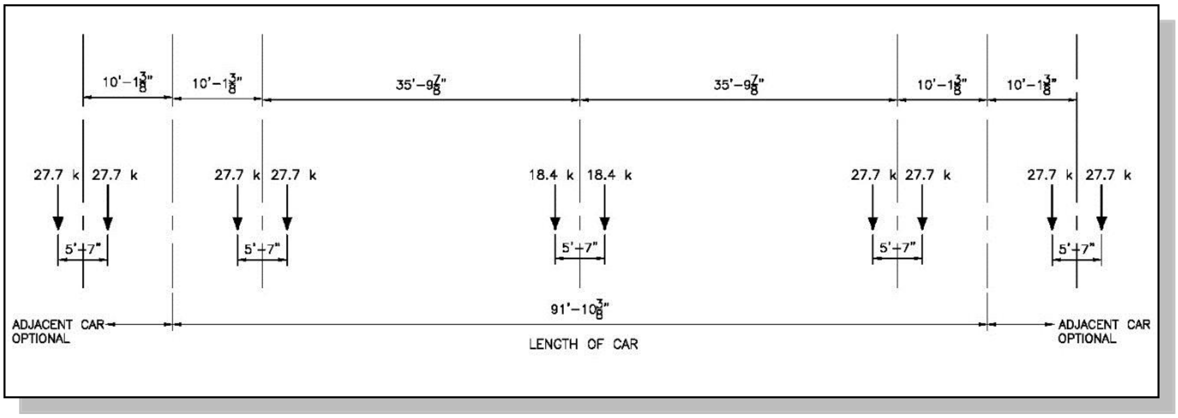


Figure 3-I
Light Rail Vehicle (LRV) adopted from NJ Transit Light Rail

In addition to the LRV live loads, the following vehicle forces are also applied to the bridge structure:

1. Impact (I)

   For structures with longitudinal girders, including cantilever systems, the following impact factors shall be applied to the moments, shears and reactions resulting from the LRV live loads:

   - I = 40 percent of the total LR vehicle loading for girders in regions of negative bending, including cantilevers and for the supports.
   - I = 30 percent in regions of positive bending.

   The loading and impact for the case of a second LRV or maintenance vehicle pushing a stalled train shall be considered. This case shall be evaluated under final design when further LRV details and operations are known.

2. Rolling or Hunting Force (RF)

   A force equal to 10 percent of the LRV loading on one track shall be applied at the top of the rail and transverse to the rail.

3. Longitudinal Braking and Tractive Force (LF)

   A force equal to 15 percent for normal braking and 30% for emergency braking of the LRV loading per track shall be applied longitudinally 5 feet above the top of rail on all tracks. Consideration shall be given to combinations of acceleration and deceleration forces where more than one track is used.

   For double-track structures, three longitudinal loading cases shall be considered:

- ❒ Single track loaded - Longitudinal force acting, applicable forces on supporting structure.
- ❒ Both tracks loaded - One train accelerating, one decelerating; maximum longitudinal forces acting, applicable forces on supporting structure.
- ❒ Both tracks loaded - Both trains accelerating or decelerating; longitudinal forces acting in opposite directions, applicable forces on supporting structure.

4. Derailment Forces

LRT structures shall be designed for a derailment loads in accordance with NJ Transit Light Rail Transit “Manual of Design Criteria” and ACI 358.1R caused by a misdirected LRV oriented with its longitudinal axis parallel to the track. The transverse position shall be assumed to be a minimum of 1.5 feet to a maximum of 3 feet from the centerline of the track. However, upon determining the final LRT track details and if shown to be justified, these minimum transverse offsets maybe revised.

The derailment load shall consist of standard vehicles with a modified dynamic load allowance. A derailment dynamic load allowance equal to 100 percent of the axle load shall be applied to any two adjacent axles at a time, with a normal dynamic load allowance factor applied to the remaining axles. The 100 percent dynamic load allowance axles shall be selected to produce the critical loading condition for the structure.

When checking any component of a superstructure or substructure that supports two or more tracks, only one train on one track shall be considered to have derailed, with the other track being unloaded and loaded with a stationary train.

The magnitude and line of action of a horizontal derailment load on a barrier wall is a function of a number of variables. These include the distance of the tracks from the barrier wall, the vehicle weight and speed at derailment, the flexibility of the wall, and the frictional resistance between the vehicle and the wall. In lieu of a detailed analysis, the barrier wall should be designed to resist a lateral force equivalent to 50 percent of a standard vehicle weight distributed over a length of 15 ft (5 m) along the wall and acting at the axle height. This force is equivalent to a deceleration rate of 0.5g.

When designing prestressed concrete members for derailment loads, the steel stress shall not exceed 85 percent of the ultimate tensile strength (0.85 f's) and the concrete stress shall not exceed 60 percent of the 28-day compressive strength (0.60 f'c). For LRFD design, the load combination used with derailment forces shall be:

- ❒ Extreme Event: γ_p(DC+DD+EL) + 1.4(LL+IM) + 1.0(TU+CR+SH) + 1.4DR + 1.0(WL+WS)

5. Collision Forces (CT)

Collision forces shall be as defined in the AASHTO LRFD Bridge Specifications.

6. Rail/Structure Interaction Force (RS)

It is assumed that all track work is constructed with continuous welded rail (CWR) with rails fastened directly to the bridge’s deck. To reduce the interactive force between the CWR and the structure, expansion rail joints can be used. However, the expansion joints shall be minimized.

During the design of non-ballasted aerial structures, an analysis shall be made of the forces resulting from rail/structure interaction.

The design of the decks, girders, bearings, frames, pier caps, piers and foundations shall include the forces resulting from the interaction between the rails and the structure when:

a. The superstructure expands and contracts beneath the rail

b. One rail breaks

c. The structure restrains the rail from displacing radially on horizontal curves

d. Any combination of the above

e. Service/Safety Walk Loading

   - ❒ LRT service and safety walks shall be designed for a loading of 75 psf.

Pedestrian

- ❒ A pedestrian load of 75 psf shall be applied to all sidewalks wider than 2'-0" and considered simultaneously with the vehicular design live load.
- ❒ Where maintenance or inspection vehicles will use sidewalks, the sidewalk deck shall be designed for an H-15 wheel load including impact (to confirm with facility).

Wind Loads

- ❒ Approach Structures
  - Wind load for approach structures shall be calculated based on the AASHTO LRFD Bridge Specifications.
- ❒ Main Span Structure
  - Under Stage II – Phase I, site-specific climatology study will be performed to determine the design wind speeds for final design and construction stage return periods defined below.
  - During Stage II – Phase II, wind tunnel model studies for the main span shall be conducted to determine wind design parameters and aerodynamic stability. Scaled models and/or section models of the pylon with deck and deck cross-section will be used in the wind tunnel tests.
  - Wind design return periods:

    For structural design of the bridge in its final condition, the 100-year return period wind speed shall be used.

    For aeroelastic stability of the bridge in its final condition, the 10,000-year return period wind shall be used.

    For structural design of the bridge during construction, the 10-year return period wind speed shall be used in design.

    For aeroelastic stability during construction, the 1,000-year return period wind shall be used in design.

Thermal Forces and Effects

The bridge shall be designed for all expected thermal movements, forces and effects of a cold climate:

- ❒ The design mean setting temperature shall be 60ºF
- ❒ Thermal coefficient for concrete structures 6.0 x 10-6 per ºF

- ❒ Thermal coefficient for steel structures 6.5 x 10^{-6} per ºF
- ❒ For deck movements for design of bearings and expansion joints:
  - Steel = -30°to 120°F
  - Concrete = 0° to 80°F
- ❒ The additional combined movement effects of creep, shrinkage and elastic shortening combined with temperature shall be considered as follows:
  - 1.2 (Temperature fall + creep + shrinkage elastic shortening)
  - 1.2 (Temperature rise)
- ❒ Temperature Gradient (TG)
  - The load factors and load combinations for temperature gradient (TG) under the strength, service and extreme limit states shall be as defined in AASHTO LRFD section 3.4.
  - A positive and negative temperature gradient (TG) per AASHTO LFRD section 3.12 shall be used for superstructure design. Transverse thermal effects for wide structures shall be accounted for in the design.
  - Between cables and superstructure, a -10ºF and a +18ºF differential temperature shall be assumed.
  - Between opposite faces of towers a differential temperature + 10ºF shall be used.

Creep and Shrinkage

- ❒ Creep effects shall be calculated in accordance with CEB/FIP 1990 Model Code for Concrete Structures.
- ❒ The relative humidity shall be taken as 70% per AASHTO LRFD Design Specifications, Fig. 5.4.2.3.3-1.

Differential Settlement

- ❒ Differential settlement between the piers shall be considered in load combinations, which include temperature, creep and shrinkage.
- ❒ Differential settlement between the piers shall be in accordance with the Geotechnical Report.

Stream Flow

- ❒ Forces due to stream current shall be based on a channel velocity defined in the scour report for the Arthur Kill. As reference, the 1995 Goethals Bridge Modernization Stage II – Phase II study defined the mean water velocity as 3 ft/sec.

Vessel Impact

AASHTO Guide Specifications and Commentary for Vessel Collision Design shall be used to determine the vessel impact force.

The bridge is classified as CRITICAL and the accepted annual frequency of collapse shall be equal to or less than 0.01 in 100 years. The design vessel has been selected based on Method II.

The following vessel impact loads provided as reference and are based on the design criteria set forth in the 1996 Goethals Bridge Modernization Stage II – Phase II study. The vessel data for the Arthur Kill and collision loads are in the process of being collected and determined under the Stage 2 – Phase I preliminary design. The final design collision loads may be subject to removal from the general design criteria and defined under security design criteria.

A. Ship collision loads

1. Condition 1: Three oversize tank barges plus a towboat at M.H.W. elevation 3.09 striking a pier. Each barge is 290 feet long by 53 feet wide with loaded displacement of 4,300 tons. The weight of towboat is assumed to be 600 tons.
2. Condition 2: A container ship at M.H.W. elevation 3.09 striking a pier. The ship is 903 feet long by 129 feet wide with fully loaded displacement of 84,500 tonnes and ballasted displacement of 31,700 tonnes.

   For both conditions 1 and 2 the velocity of the vessel is assumed to be 14.5 feet per second (still water vessel speed of 11.8 fps combined with current speed of 2.7 fps).
3. Condition 3: As determined by The Port Authority of NY & NJ by further study.

Vessel impact forces shall be applied to the bridge structure as defined in AASHTO LRFD section 3.14

Scour

A. Scour depth calculated by FHWA Hydraulic Engineering Circular No. 18 (HEC-18).

B. The main span structure shall be designed for the following scour events calculated in part a) above:

1. The more critical of AASHTO LRFD Section 2.6.4.4, NJDOT LRFD “Design Manual for Bridges and Structures” Section 45.6
2. Based on NCHRP Report 489 “Design of Highway Bridges for Extreme Events:”
   a. 1.25DC+1.5DW+1.75(LL+IM)+1.0FR+TG+SE+1.80SC(scour)
   b. 1.25DC+1.5DW+1.0WA+1.0WS+1.0FR+TG+SE+0.70SC(scour)
   c. 1.25DC+1.5DW+1.0WA+1.0FR+1.0(IC+CT+CV)+0.60SC(scour)
   d. 1.45DC+1.5DW+1.0WA+1.0FR+1.0EQ+0.25SC(scour)

Potential future dredging of the shipping channel shall be considered in the evaluation of scour.

Ice Load

Ice loads shall be designed but not limited to:

A. Substructure

Substructure ice loads shall be designed in accordance with AASHTO LRFD. A pier ice load generated by a 6” thickness and a strength of 32 KSF shall be applied to the substructure at elevation 0.0.

B. Superstructure

An ice load shall be applied simultaneously to each stay cable in accordance with the ANSI/ASCE, “Minimum Design Loads for Building and Other Structures” latest edition. In addition, the ice load shall be applied simultaneously to the perimeter of the portion of the pylon above the roadway deck.

LOAD MODIFIERS

The resistance factor will be computed as per AASHTO 1.3.2.1. The load modifiers used in the computation are given below.

A. Ductility Factor

The ductility factor, η_D=1 for all limit states for all bridge components.

B. Redundancy Factor

For redundant members, whose fracture would cause a significant loss of functionality of the bridge, a redundancy factor of ηR = 1.05 at the strength limit state will be applied.

C. Importance Factor

The Goethals Bridge is considered an important bridge, an Importance Factor, ηI = 1.05, for all strength limit states will be applied.

COMBINATIONS OF LOADS

Load combinations shall be in accordance with the AASHTO LRFD Bridge Design Specifications and as defined within these criteria, whichever results in a higher demand.

The seismic design shall combine the Earthquake Load with Live Load, using a factor for live load γEQ of 0.50, if it is more critical.

LIVE LOAD DEFLECTIONS

A. For structures supporting only light rail, girders of simple or continuous spans shall be designed so that the deflections due to live load plus impact shall not exceed 1/1,000 of the span length.

B. For structures not supporting light rail, the live load deflection provisions in the AASHTO LRFD Bridge Design Specifications shall be used.

C. For structures supporting both light rail and highway vehicles or that may support both in the future the more restrictive limit of 1 or 2 above shall apply.

D. Sidewalk deflections and vibrations shall be checked for serviceability.

MATERIALS

A. Concrete—All concrete properties shall be in accordance with the following minimum 28-day design compressive strength (f'c):

Location	f'c (psi)
Cast-in place deck slab	5,000
Pre-cast deck slab panels (prestressed and/or post-tensioned) and C.I.P closure pours	6,000
Prestressed girders	10,000
Prestressed concrete box girder segments & cast-in-place closure pours	6,000
Pylons	6,000
Pre-cast piers	6,000
Cast-in-place piers	5,000
Footings and Abutments	4,000
Barriers	4,000

Location	f'c (psi)
Drilled Shafts	4,000
Tremie Seals	3,000

B. Prestressing Steel

1. Strand

All strands shall conform to the requirements of AASHTO M203 (ASTM A416), Grade 270 for low-relaxation, weldless strand.

Prestressing Parameters for Strand (f_{pu}=270 ksi, f_{py}=243 ksi):

Modulus of Elasticity	28,500 ksi
Maximum jacking stress at anchorage	0.90 f_{py}
Maximum tendon stress at anchorage immediately after anchor set	0.70 f_{pu}
Max stress at end of seating loss zone immediately after anchor set	0.74 f_{pu}
Maximum stress at service limit state after long term losses	0.80 f_{py}
Anchor set	0.25 in
Friction coefficient (μ)	
Galvanized steel ducts	0.25
Polyethylene ducts	0.23
Wobble coefficient (K)	
Internal galvanized steel ducts	0.0002 /ft
Internal polyethylene ducts	0.0002 /ft
Strand size	0.6 in diameter

2. Bars

All bars shall conform to the requirements of AASHTO M275 (ASTM A722), Grade 150, Type 2, deformed.

Prestressing Parameters for Bars (fpu=150 ksi, fpy=120 ksi):

Modulus of Elasticity	29,000 ksi
Maximum jacking stress at anchorage	0.90 f_{py}
Maximum steel stress at anchorage immediately after anchor set	0.70 f_{pu}
Max stress at end of seating loss zone immediately after anchor set	0.70 f_{pu}
Maximum stress at service limit state after long-term losses	0.80 f_{py}
Anchor set	0.0625 in
Friction coefficient (μ)	0.15
Wobble coefficient (K)	0.0002/ft

3. Post-tensioned tendons and bars

All permanent external and internal post-tensioning shall be grouted per the AASHTO LRFD Design and Construction Specifications and the PTI Grouting Specifications for Post-Tension Structures.

C. Structural Steel

1. Structural steel shall conform to requirements of AASHTO M270M (ASTM A709) for:
   - ❒ Grade 36
   - ❒ Grade HPS 50
2. High Performance Steel Grade 70 shall conform to requirements of ASTM A709 Grade HPS 70.
3. All structural steel shall be painted, unless otherwise specified.

D. Stay Cables

1. Acceptance testing of stay cables shall be in accordance with the AASHTO LRFD and the latest edition of the PTI Recommendations for Stay Cable Design, Testing, and Installation.
2. Stay cable strand shall consist of 0.6-inch diameter, low-relaxation, weldless seven-wire strand conforming to the requirements of AASHTO M203 (ASTM A416), Grade 270.
3. The space between the stay strand and the outer HDPE sheath along the free length of the duct shall not be filled with cement grout or any other material.
4. The stay design shall be such that the replacement of any cable can be done, if required, strand by strand, in order to reduce to a minimum any traffic disruption.

E. Reinforcing Steel

1. Reinforcing steel shall conform to the requirements of AASHTO M31, Grade 60.
2. Weldable reinforcing steel shall conform to the requirements of ASTM A706, Grade 60.
3. All reinforcement above the top of footing or extending out above of the top of footing shall be epoxy coated.
4. Minimum clear cover to reinforcing steel shall as noted below:

Top of structural slab where wearing surface (ws) will be added		2 in + ws
Top of structural slab where no wearing surface will be added		3 ½ in
Bottom of deck slab		1 ½ in
External underside of superstructure concrete box segments		1½ in
Internal to superstructure concrete box segments		1 in
Precast Prestressed Concrete Girders		1 in
Towers above deck		2 in
Towers below deck		3 in
Pier caps (Main Steel)		3 in
CIP Piers and Precast Pier segments	(external surface)	3 in
	(internal surface)	2 in
Precast Pier Segments		3 in
Pier Footings		3 in

F. Fasteners

7/8-inch diameter high-strength bolts conforming to ASTM A325 Type 1 (slip critical connections) shall be used unless otherwise specified in the plans.

G. Bearings

High Load Multi-Rotational bearings or Steel Reinforced Elastomeric bearing pads.

H. Anchor Bolts

- Anchor bolts shall be in accordance with ASTM F1554, Grade 50 or Grade 105 as appropriate.
- Anchor bolts, nuts and washers shall be hot-dipped galvanized in accordance with ASTM A153.

DESIGN PROCEDURES

A. Concrete Deck Slabs

1. The slab shall be designed in accordance with AASHTO LRFD Design Specifications and checked for serviceability.
2. Steel Stay-In-Place (SIP) forms shall not be permitted.
3. The wearing surface shall not be considered as effective in resisting axial or bending forces.
4. The limiting distribution of flexural reinforcement “Z” for crack control calculations shall be 130 kips/inch.

B. Prestressed Concrete Elements

1. Prestressed Concrete Stress Limits
2. Temporary stresses before losses due to creep and shrinkage at the time of transfer:

   Compression: 0.60 f’ci

   Tension shall be per Table 5.9.4.1.2-1 of the AASHTO LRFD Design Specification. Joint Type B shall not be permitted.

3. Stresses at service level after losses have occurred:

   Compression shall be per Table 5.9.4.2.1-1 of the AASHTO LRFD Design Specification.

   Tension shall be per Table 5.9.4.2.2-1 of the AASHTO LRFD Design Specification. Joint Type B shall not be permitted.

C. Anchorage Stress Limits

Bearing stress under anchorage plates shall be in accordance with AASHTO LRFD Design Specifications.

D. Construction Stress Limits

1. A minimum compressive strength of 4000 psi shall be attained before releasing forms on non-prestressed concrete.
2. A minimum compressive strength of 4000 psi shall be attained before stressing transverse and longitudinal post-tensioning.

E. Steel Composite Girders, Stringers and Floor beams

1. Design shall be in accordance with the AASHTO LRFD Bridge Design Specifications.

2. Effective width of the deck slab shall be determined in accordance with AASHTO LRFD. Girders shall be designed in accordance with Article 4.6.2.6.2 and floor beams in accordance with Article 4.6.2.6.

F. Stay Cable Design

1. Stay cables shall be designed in accordance with the latest edition of the AASHTO LRFD and the PTI Recommendations for Stay Cable Design, Testing, and Installation.

2. Resistance Factors:

For the limit states referred to in Section 3.4 of the AASHTO LRFD, the following resistance factors shall apply:

Strength Limit State	ϕ□ = 0.65
Service Limit State	$\phi = 0.85$
Extreme Event	$\phi = 1.00$
Fatigue	$\phi = 1.00$
Construction	$\phi = 0.75$

3. Provision shall be made for the replacement of any individual stay cable while maintaining two lanes of traffic in the direction of travel adjacent to the cable being replaced. The lanes of traffic shall be shifted as far as possible away from the cable being replaced.

4. Provision shall be made for the adjustment of the cable force and deck profile in any individual stay cable.

5. The design shall provide for the loss of any single stay cable under full live load in their striped lanes.

6. In addition to AASHTO LRFD, NJDOT LRFD and NYSDOT LRFD Specifications, the following special load factors and combinations as suggested by the PTI reference above shall be used for the following stay cable conditions:

G. Stay replacement

1.2DC + 1.4DW + 1.5(LL* + IM) + Cable Exchange Forces

H. Loss of cable

1.2DC + 1.4DW + 0.75(LL** + IM) + Cable Loss Dynamic Forces

It should be noted that a current draft ballot edition of the "Recommendations for Stay Cable Design, Testing and Installation" proposes to revise the load factors for the loss of cable load group. It is recommended that the status of this revision be followed and evaluated for adoption under the final design.

* At least one lane of live load shifted away from the cable under exchange.

** Full live load placed in their actual striped lanes.

I. Substructure Design

1. Piers and Towers

2. All piers and towers shall be designed using AASHTO LRFD Bridge Design Specifications.

3. For hollow sections, wall slenderness ratios shall not exceed 15, using Section 5.7.4.7 of the AASHTO LRFD.

4. Piers shall be designed as tied. Minimum ties shall be #4 bars. Tie spacing shall be as required by design, however shall not exceed 12-inches.
5. Where sufficient ductility for seismic plastic hinging has been demonstrated, precast segmental piers and towers may be designed with Type A joints using the seismic provisions for zones 3 and 4.

J. Foundations

1. Foundations shall be designed as defined under the Design Methods section.
2. Bottom of foundations shall be placed below the frost line.
3. Drilled shafts shall be embedded a minimum of 12 inches into footings. Reinforcing steel shall be fully developed into the footing.
4. Minimum center-to-center spacing for drilled shafts shall be 3 shaft diameters unless defined otherwise in the final geotechnical report.
5. All unsuitable material within the limits of the foundations shall be removed and disposed of as directed by the engineer.
6. Basic allowable rock bearing pressure for spread footings shall be in accordance with the final geotechnical report.

SEISMIC DESIGN

Seismic Design shall comply with:

- ❒ AASHTO LRFD Bridge Design Specifications (Ref. 1)
- ❒ New York Seismic Design Specifications (Division 1A for Downstate Bridges)

Importance Classification

The new Goethals Bridge shall be classified as “Critical”. It is a lifeline structure that must remain open to all traffic, including emergency vehicles, and be usable for security/defense purposes immediately after a safety level seismic event.

Design Earthquake Levels and Seismic Zone

The bridge shall be designed for two earthquake levels:

- ❒ A lower level, Functional Evaluation Earthquake, “FEE”, defined as an event having a 10% probability of exceedance in 50 years. (500-year return period).
- ❒ An upper level, Safety Evaluation Earthquake, “SEE”, defined as an event having a 2% probability of exceedance in 50 years. (2500-year return period).

  The seismic performance zone shall be classified as Zone 3 as defined in AASHTO LRFD Section 3.10.4.

Performance Criteria

The performance objective for this structure is to provide:

- ❒ FEE Event—No collapse, no damage to primary structural elements with minimal damage to other components. Immediate access to all traffic after a couple hours for inspection.
- ❒ SEE Event—No collapse, repairable damage. Access limited access to emergency vehicles within 48 hours with full service to general traffic within months.

Consistent with NYSDOT Seismic Specifications, this classification necessitates the following performance characteristics:

❒ Functional Evaluation Earthquake (FEE):

- Essentially elastic response (minimal damage) in exposed ductile elements and elastic response in buried or submerged ductile elements.
- The response is associated with minimal damage and is virtually not visible.
- No permanent deformations.
- Damage to expansion joints that can be temporarily bridged with steel plates.

❒ Safety Evaluation Earthquake, (SEE)

- Inelastic response may occur in exposed ductile elements but only elastic response in buried or submerged ductile elements.
- This response is associated with repairable damage such that repairs can be made without replacement of the structural element. The damage is visible in the form of concrete cracking, spalling, reinforcement yielding, minor yielding of steel elements.
- Post-earthquake permanent deformations are small enough, that they do not interfere with the serviceability of the bridge.
- Damage to expansion joints that can be temporarily bridged with steel plates.
- Any repairs could be made under non-emergency conditions with the exception of repairs to the expansion joints.

Seismic Load Path

The bridge shall have a clearly defined inelastic mechanism for response to seismic loads.

A. The principal seismic force resisting elements shall be limited to the piers, tower legs and the tower struts. Inelastic behavior in the superstructure, and foundations, shall not be permitted.

B. In the elements anticipated to behave in elastically, and all other elements that may participate in forming a plastic mechanism, seismic detailing that ensures full ductility, shall be implemented.

C. Elements connecting with the principal seismic force resisting elements shall be designed on the principles of capacity design.

Design Ground Motions

Refer to the Geotechnical Design Criteria for the development of Design Ground Motions.

(**INSERT FINAL MOTIONS**)

Design Response Spectrum

The New Jersey approaches shall be designed for the following FEE and SEE spectrum.

(**INSERT SPECTRUM**).

The main span and New York approaches shall be designed for the following FEE and SEE spectrum (**INSERT SPECTRUM**).

Liquefaction and Lateral Ground Movement

An evaluation of the potential for and consequences of soil liquefaction within near surface soil shall be made.

Soil springs (p-y, t-z, q-u) shall be furnished to enable dynamic analysis in both, the liquefied configuration and the non-liquefied configuration for both the FEE and SEE events.

Liquefaction and liquefaction-related ground movement shall be treated as separate and independent load cases.

(**INSERT LOADS FROM GEOTECH**)

Seismic Design of Ductile Members

- ❒ Preliminary Design—Force based design in conjunction with an elastic response spectrum analysis may be used for the preliminary design stages of the Goethals Bridge structure. The response modification factors shall be taken from NYSDOT Seismic, Table 6B. 4-1 for a bridge of critical importance.
- ❒ Final Design—The approach structures shall be analyzed using a multimode spectral analysis augmented by a non-linear static pushover displacement analysis. (Deformation based design)

  The main span shall be analyzed, either by a multimode spectral analysis augmented by a non-linear static pushover displacement analysis, or by a detailed nonlinear dynamic time history analysis.

Deformation Based Design

Deformation based design may be used in conjunction with either an elastic response spectrum analysis with cracked properties and secant springs or with a detailed non-linear dynamic time history analysis.

When pushover analysis is used to calculate the deformation capacity of the structure, the calculated deformation demand shall not cause the material strains to exceed the limitations given in Section 14.15. No yielding of stay cables shall be permitted.

Multimodal Response Spectrum Elastic Analysis

Seismic demands shall be determined by a response spectrum analysis using a 3D finite element model.

The response spectrum analysis shall incorporate the following:

A. Equivalent linear springs shall be used to model the stiffness of the soil. Both, upper bound and lower bound values of springs, appropriate for the design level ductility requirements, representing liquefied and unliquefied soil configuration shall be investigated.

B. Any reinforced or prestressed concrete members, which are expected to crack in an earthquake, shall be modeled with adjusted material and section properties to represent the cracked section.

C. “Compression type” and “tension type” models shall be used if gap widths at the expansion joints are such that adjacent frames become longitudinally coupled during a seismic action.

D. The maximum seismic force due to seismic load shall be based on the CQC combination of modal responses. The maximum force due to three orthogonal ground motion components - longitudinal, transverse and vertical - shall be obtained by a “100%-40%-40% Combination Rule”

E. Loads shall be combined in accordance with the Extreme Event I load case as defined by Table 3.4.1-1of the AASHTO LRFD code. □eq of 0.5 shall be used. The dead loads used for combination with seismic loads shall fully consider the effect of construction sequence.

Nonlinear Dynamic Time History Analysis

Seismic demands for the main span shall be determined by a nonlinear multi-support dynamic time history analysis. This analysis shall incorporate the following:

A. Three orthogonal components of the phased time histories shall be applied simultaneously. Each time history shall be applied simultaneously to the model in the longitudinal, transverse and vertical direction considering the effects of phasing.

B. If three time history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more time history analyses are performed, the average value of the response parameter may be used for design.

C. The structural model shall explicitly consider the geometric and material non-linearities of all pertinent elements.

D. The structural model shall account for nonlinear stiffness of soil by explicitly modeling non-linear p-y, t-z and q-u soil springs. The properties of the springs will be determined from local models, and shall include group effects. The model may also account for local damping in soil.

E. Nonlinear beam elements shall be modeled using “moment-curvature elements” which are based on expected material properties as defined in article 14.15.

F. Expansion joints shall be modeled using “gap” elements.

G. Damping shall be carefully applied using the analyst’s judgment such that the dominant frequencies of concrete elements are damped to 5% of critical damping and the dominant frequencies of the steel elements are damped to 3 % of critical damping. Fourier analysis may be used to identify the dominant frequency modes.

H. For selected members deemed as “controlling the design” by the designer, the effect of construction sequence on dead load distribution shall be accounted for.

I. Time history analysis shall be performed on a model representing the liquefied configuration of soil.

Construction Stage Earthquake

Construction Stage effects shall be evaluated in accordance with NYSDOT Seismic Criteria, Section 3.12.

Determination of Capacities

The Seismic Design Requirements of the AASHTO LRFD Specifications and the NYSDOT for a critical bridge category shall apply.

Stay Cables

The load capacity of stay cables shall be in accordance with AASHTO LRFD Specification and with PTI’s 4th Edition “Recommendations for Stay Cable Design, Testing, and Installation”.

Where bridge designs result in computed dynamic tensions less than 5% of GUTS, the Engineer shall note the minimum cable forces computed from dynamic analysis on the plans. The Engineer shall specify any special requirements for qualification testing of anchorages, including net minimum force, minimum number of strands, wires or bars for the test specimen, maximum length of the test specimen, frequency of the dynamic test load, and a minimum number of loading cycles for the test.

Reinforced and Prestressed Concrete Component Capacities

The reinforced concrete capacities shall be calculated in accordance with AASHTO LRFD Section 5.10.11.4 and Section 3.10.9.4.1.

A. Design Flexural Strength

For seismic loads the nominal moment capacity of the ductile reinforced and prestressed concrete elements and of the capacity protected elements shall be based on expected material strengths as follows:

Expected concrete compressive strength	f'_{ce} = 1.3 * f'c
Expected reinforcement yield strength	f_{ye} = 1.1 * fy
Expected prestressing strand yield strength	f_{pye} = 0.9*fpu
Expected prestressing bar yield strength	f_{pye} = 0.9*fpu

Maximum concrete strains at the design flexural strength shall not exceed 0.003.

Capacity protected members, as indicated AASHTO LRFD specification shall be designed for forces derived from maximum plastic hinging moments.

B. Maximum Plastic Moment Capacity

Plastic hinging moments shall be determined directly from a moment-curvature analysis described in paragraph d below. The value used for design shall correspond to the peak seismic curvature demand. The material properties for this analysis shall be:

f'_{co} =	1.7 * f'c
f_{yo} =	1.25 * fy
f_{pe} =	TBD

C. Deformation Capacity

The deformation capacity of reinforced concrete and prestressed elements shall be calculated using the plastic hinge lengths and the rotational capacities corresponding to the allowable material strains of paragraph d and e below. (**PLASTIC HINGE LENGTH FORMULA SHALL BE PROVIDED HERE**)

D. Strain Limits for Concrete

Where necessary, moment curvature plots shall be developed using Mander's stress-strain model for confined concrete. (Mander et al, J. Struct. Engineering, ASCE, 1988 114(8), 0 1804-1849. A computer program that takes into account Mander's concrete model, confinement steel, strain hardening of longitudinal and transverse steel and the spalling of concrete cover shall be used. Allowable concrete strains shall be:

Functional Evaluation Earthquake	$\varepsilon_c = 0.004$
Safety Evaluation Earthquake	$\varepsilon_c^{safety} = 2/3\ \varepsilon_{cu}$
where ε_{cu} is the ultimate concrete strain given by the Mander model.	

E. Strain Limits for Column/Pylon Reinforcement Steel

Allowable Strain Limits for Grade 60 (A706) reinforcing used in yielding elements shall be taken as follows:

Average extreme fiber strains in potential plastic hinge zones.

	#8 and smaller	#9	#10	#11	#14	#18
Functional Evaluation Earthquake, ε_s^{FEE}	0.015	0.0125	0.0115	0.0115	0.0075	0.0050
Safety Evaluation Earthquake, $\varepsilon_s^{SEE} = 2/3\ \varepsilon_{su}$	0.080	0.080	0.080	0.060	0.060	0.060
Steel Strain at Ultimate Stress, $\varepsilon_{s\ su}$	0.12	0.12	0.12	0.09	0.09	0.09

F. Strain Limits for Prestressing Strand

Allowable strains in the 270 Ksi, low relaxation strand shall be as follows:

Elastic prestress steel strain, $\varepsilon_y = 0.0086$

Ultimate prestress steel strain, $\varepsilon_{pu}=0.0300$

$$\varepsilon_{ps} \le 0.0086 \Rightarrow f_{ps} = 28500 \times \varepsilon_{ps}$$

$$\varepsilon_{ps} > 0.0086 \Rightarrow f_{ps} = 270 - \frac{0.04}{(\varepsilon_{ps} - 0.007)}$$

G. Reinforced and Prestressing Concrete Detailing

The requirements of AASHTO LRFD for Seismic Zones 3 and 4 apply.

H. Design for Joint Shear

Joints between ductile flexural members shall be proportioned, reinforced and detailed in accordance with AASHTO LRFD. Joints shall be checked for principal tension failure and forces shall be rationally calculated appropriate using strut and tie models.

I. Column Shear Design

Shear demand shall be the maximum shear obtained from the maximum plastic moment capacity of the column and pylon sections.

J. Concrete Superstructure Component Capacities

All structural elements above the soffit of the superstructure, including cables and the portion of the pylon above the deck, shall be designed to remain elastic during an SEE event.

Structural Steel Superstructure Component Capacities

The steel superstructure will be designed to remain elastic. All connections shall be designed to be capacity protected.

The limiting plate width/thickness ratios for axial compression will conform to AASHTO LRFD Section 6.9. Non-compact sections may be used in the girder but the maximum factored strains at the top or bottom of the girder shall not exceed the yield point strain.

Strain Limits for Structural Steel Pile Shells (casings – if used)

The strains in the steel casing shall not exceed the yield point strain in either the FEE or the SEE event.

Bearings

Bearing plates shall be sized such that a movement equal to 1.5 x the peak relative movement caused by the SEE could be accommodated without unseating of the bearing plate.

Expansion Joints

TBD

ERECTION AND CONSTRUCTION DESIGN

A. Miscellaneous

1. A suggested construction sequence shall be indicated on the plans and, if utilized by the Contractor, shall be verified for accuracy and applicability by the Contractor. It is presented for information only and, if used, is to be fully developed by the Contractor.
2. In addition to the camber indicated in the final design plans, the contractor shall provide camber to the structure to account for the method of construction chosen. The final grades and cross-slopes as indicated in the plans shall be achieved and confirmed by a final survey.
3. The contractor shall submit complete stress and camber calculations in each erection stage, detailed shop and erection drawings, and the proposed erection method to the Engineer for review.
4. The contractor shall insure that the structure remains safe and stable under all anticipated loading conditions during construction.

B. Minimum Construction Loadings

1. Dead Loads

   Applied loads shall include, but are not limited to the actual weight of form travelers (including forms), construction equipment, materials, lifting devices, etc. A minimum 10% impact shall be applied to all dead loads being lifted.

2. Live Loads

   A minimum construction live load per AASHTO LRFD Bridge Specifications shall be applied to the superstructure to produce the maximum construction effects.

3. Seismic Loads

   Seismic loading during construction shall be as defined by Section 3.10.10, "Requirement for Temporary Bridges and Stage Construction" of the AASHTO LRFD Bridge Design Specifications.

4. Thermal Forces

   Temperature effects shall be considered during construction.

5. Construction Wind Loads

   Wind loads during construction shall be as defined under the Wind Load section of these criteria.

5. Combinations

   Construction loads shall be combined in accordance with AASHTO LRFD Design Specifications load combinations with an equivalent, if any, overstress allowance of 25% for temporary condition used in accordance with AASHTO LRFD Specifications. Tensile prestressed concrete stresses under erection or construction service loads shall not exceed the tensile limits of Sections 5.14.2.3.3 and 5.9.4 of the AASHTO LRFD or the requirements of NJDOT LRFD or the NYSDOT LRFD Specifications.

MISCELLANEOUS

A. Bearing Replacement

Provision shall be made for jacking of the superstructure for replacement of the bearings. Jacking locations and loads shall be shown on the plans.

B. Expansion Joints

An appropriate durable, water-tight expansion joint shall be chosen to meet the movements and loads of the approach and main span layouts. Future replacement of the expansion joints should be considered in the initial design.

C. Utilities

Utility needs shall be coordinated with all disciplines, outside agencies and utility companies under Stage II and III design phases. An appropriate equivalent linear load shall be selected to represent the utilities in the design. Gas lines shall be reviewed and approved by the PA.

D. Lighting

Lighting loads and pedestals shall be included in the final design of the structures.

E. Inspection Access

Permanent Inspection access requirements will be discussed with the Port Authority.

F. Signing

Sign loads and pedestals shall be included in the final design of the structures.

G. Coordination

The Contractor shall coordinate all parts of the work in the contract including mechanical and electrical work; the requirements for said work are shown throughout the contract documents.

3.2.8 GWB

3.2.8.1 GWB (SAMPLE DESIGN CRITERIA FOR ORTHOTROPIC DECK)

3.2.8.1.1 General Provisions

A. Objectives & Scope

The purpose of this document is to establish the design criteria and specifications to be used for the for GWB upper level deck rehabilitation project will be used for the design of the priority deck repairs.

B. Limits of Applicability

These criteria apply to the upper level orthotropic deck and floor system of the George Washington Bridge suspended spans. Elements of the orthotropic deck and floor system will be evaluated and designed to accommodate the demands imposed by AASHTO and NYSDOT standard design vehicles, based on current design practice, as well as site-specific Weigh-In-Motion (WIM) data for the George Washington. Fatigue considerations will comply with the proposed revisions to the AASHTO LRFD code, which is anticipated to become available in 2007.

C. Design References and Governing Specifications

The documents indicated below form the basis for the evaluation/design of the upper level deck system replacement, listed in descending order of governance. Where conflicts exist between these Design Criteria and other references, these Design Criteria will control.

1. Structural Design Criteria for the George Washington Bridge Upper Level orthotropic deck, latest revision (Design Criteria).
2. AASHTO Standard Specifications for Highway Bridges, Latest Edition (AASHTO) and applicable NYSDOT and NJDOT modifications to AASHTO.
3. AASHTO LRFD Bridge Design Specifications – Customary US Units, Latest Edition (AASHTO LRFD) with Interims, as specifically indicated herein, as well as proposed revisions to fatigue considerations for orthotropic bridges.
4. Standard Specifications of ASTM International (ASTM).
5. AASHTO/AWS D1.5 Bridge Welding Code
6. AASHTO Manual for Condition Evaluation of Bridges Latest Edition
7. Applicable NYSDOT and NJDOT Standard Specifications for Highway Bridges

D. Design Approach

The AASHTO Standard Specifications (ASD) Method will be used for design of all steel and concrete structural members except as noted herein.

Both AASHTO Standard Specifications (ASD) and proposed revisions to the AASHTO-LRFD will be used to determine the maximum governing fatigue stress range in the orthotropic deck plate and ribs

For existing members that may not pass an ASD design check for new loadings imposed on the structure, the AASHTO-LRFD specification may be used, and shall be clearly called out in the calculations.

3.2.8.1.2 Design Loading

A. Structural Dead Loads

Structural dead loads shall be based on unit weights of materials and the computed volumes of the structural elements. The following unit weights shall be used:

1. Reinforced concrete: 160 PCF
2. Orthotropic deck overlay: 150 PCF
3. Steel: 490 PCF
4. Utilities: Actual computed weight based on conduit, conductor and support system.

B. Live Load

The orthotropic deck on the George Washington Bridge will be evaluated for both local and global loadings.

1. Local Loading (Floor beam, Stringers, Subfloorbeams and Deck)
    a. The following criteria for live load lane configurations in the upper deck shall apply with up to six (6) lanes of H-10 considered for the lower deck (Figure 3-J):
        1) Up to eight (8) lanes concurrently loaded with AASHTO HS25 (HS20 increased by 25%) or,
        2) Loading to be determined by the WIM study
        3) Not less than the local loading design criteria for GWB Contract 170.003 for the original orthotropic deck construction or the design loads and impact factors from the Port Authority Specifications for Design of Bridges (July 1929).

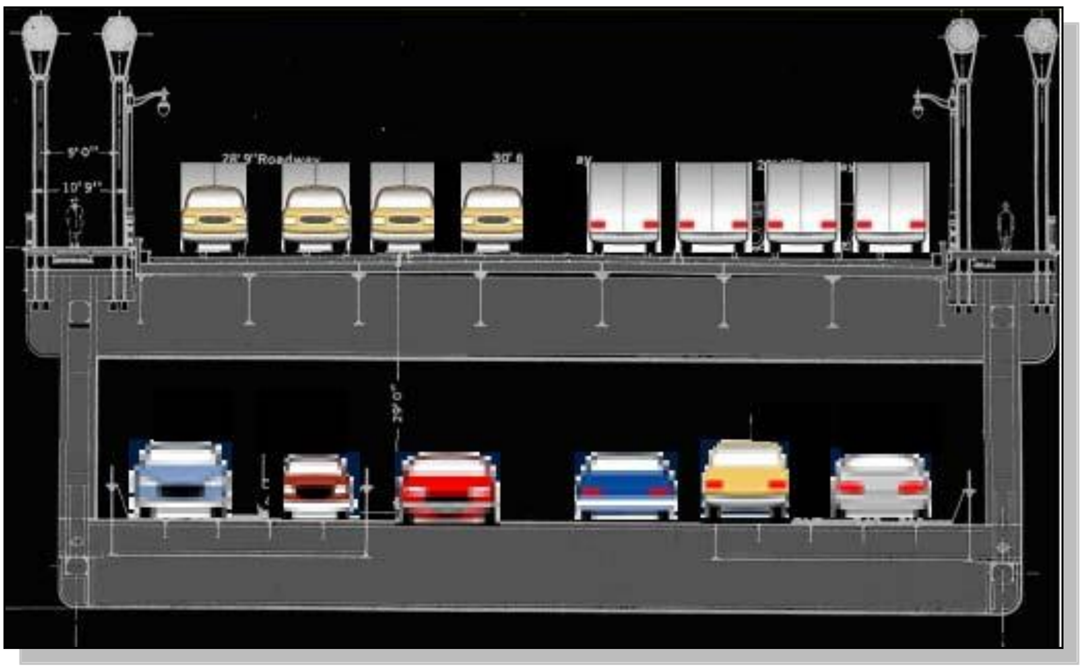


Figure 3-J
Design Vehicles and Placement on the Upper Level

        4) A single NYSDOT Design Permit Vehicle as defined in NYSDOT's LRFD Blue Pages (Figure 3-K) will be used as an overload case (AASHTO Group II loading with a 25% increase in allowable stresses). This vehicle is defined as follows, and appropriate Imperial Units will be used:

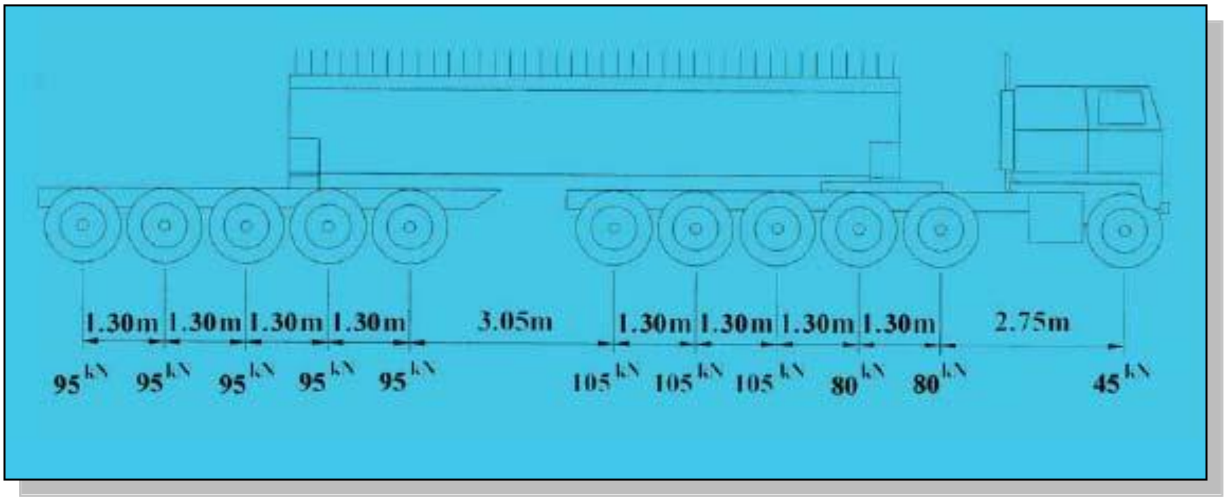


Figure 3-K
NYSDOT Design Permit Vehicle

b. Reduction factors for multiple lane presence for up to six lanes shall be in accordance with AASHTO. For multiple presence of 7-12 lanes, 0.6 shall be used, which is the same as the original design factor and is consistent with current AASHTO-LRFD of 0.65 for three or more lanes

c. The fatigue resistance of the orthotropic deck will be investigated using the provisions of the latest edition of AASHTO LRFD Specifications. The fatigue design vehicle shall have the axle configuration shown in Figure 3-L. The fatigue truck will be applied to the orthotropic deck with 15% impact. This loading was developed in conjunction with Dr. John Fisher. This modified fatigue loading considers a "3 x HS-15" truck, in lieu of the "2 x HS15" as indicated in the 3rd Edition AASHTO LRFD Specifications.

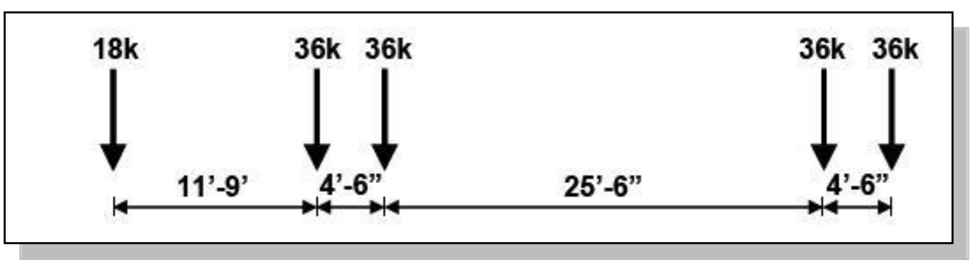


Figure 3-L
Effective Fatigue Vehicle Axle Spacing and Loading

d. The tire pressures will be applied over contact areas as defined in AASHTO. These will be used to model local fatigue and service load effects on the deck plate, ribs, sub-floor beams, welds and other local orthotropic deck details.

As a point of reference, the original bridge was designed for the live load configuration as indicated in Figure 3-M.

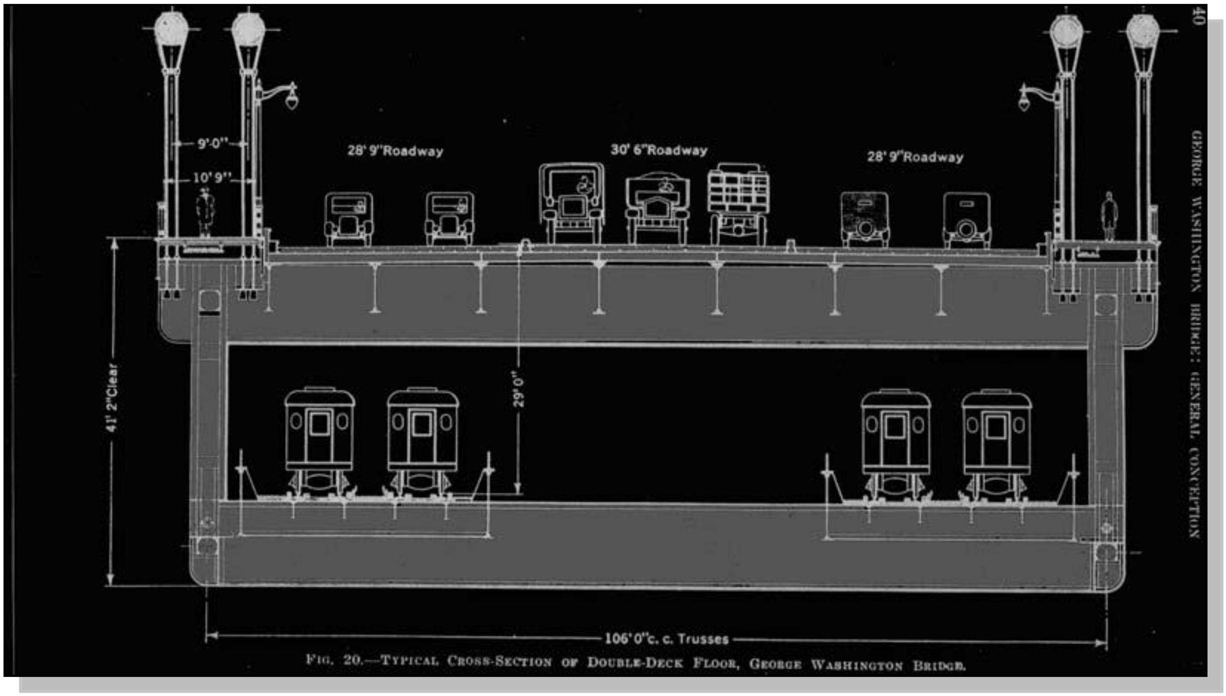


Figure 3-M
Original Loading Configuration

2. Global Loading

   The original design criteria provided for 8000 plf of bridge shall be used.

3. Thermal Loading

   The thermal loads design temperature range shall correspond to NYSDOT requirements for Region 11:

	Mean Temperature	64° F	
	Rise	Fall	Range
Steel	56° F	64° F	0 - 120
Concrete	16° F	54° F	10 – 80

A differential temperature gradient of 20° F between the deck plate elements and the remainder of the structural system including ribs, sub-floor beams, stringers, floor truss, top and bottom chord of the truss will be considered for the design of the orthotropic deck.

3.2.8.1.3 Materials

A. New Steel

1. Orthotropic Deck -Steel ASTM A709, Grade 50
2. Connections -Steel ASTM A709, Grade 50
3. Steel Fingers -Steel ASTM A709, Grade 50
4. High Strength Bolts -Steel ASTM A325, Galvanized

B. Original Steel

1. Tension (Net Section)

Carbon Steel	20,000 psi allowable
Silicon Steel	27,000 psi allowable
Nickel Steel	33,000 psi allowable

2. Compression (Gross Section)

Carbon Steel	17,000 psi allowable
Silicon Steel	23,000 psi allowable
Nickel Steel	28,000 psi allowable

3. Shear

Carbon Steel	12,500 psi allowable
Silicon Steel	17,000 psi allowable
Nickel Steel	20,000 psi allowable

4. Bearing

Carbon Steel	30,000 psi allowable
Silicon Steel	40,000 psi allowable
Nickel Steel	50,000 psi allowable

5. Power-driven rivets and turned bolts -30,000 psi allowable
6. Hand-driven rivets and unfinished bolts -20,000 psi allowable
7. Orthotropic Deck Steel

   Deck Plate and ribs -A588

3.2.8.1.4 Cost Estimating

A. Construction Cost Estimating

1. Port Authority Construction Estimating Standards

B. Bridge Life-Cycle Cost Analysis

1. NCHRP Report 483 – Bridge Life-Cycle Cost Analysis - Methodology for bridge life-cycle cost analysis (BLCCA) for use by transportation agencies.

3.2.8.1.5 Miscellaneous

A. Miscellaneous Design Criteria

1. NFPA 14 – Installation of Standpipe and Hose Systems
2. NFPA 502 – Road Tunnels, Bridges and Other Limited Access Highways
3. OHSA (Occupational Safety and Health Administration) – Safety and Health Standards – 29 CFR 1926

3.2.8.2 GWB PIP Ramps to Lower Level (Sample RFP Attachment A - 2002)

RFP Attachment A

Expert Professional Services for George Washing Bridge
Palisades Interstate Parkway Connector Ramp to
Lower Level Roadway

A. Background

The project is to construct a new ramp connection for the southbound Palisades Interstate Parkway (PIP) to the eastbound Lower Level roadway of the George Washington Bridge. The proposed roadway begins at the exiting lanes from the PIP Toll Plaza, continues southerly as a combined roadway with the present ramp to the Upper Level over the GWB mainline roadways, diverges on-grade before the overpass at Hudson Terrace, and continues in a westerly direction above Hudson Terrace and below grade under the local streets (Central Road, Hoyt Avenue and Service Street) where the ramp then merges with the eastbound Lower Level roadway.

B. Scope of Work

The services of the Consultant consist generally of the following:

- ❒ Performing verification of field conditions and retrieval of previous details:
- ❒ Preparing structural details of specialized drainage structures, structural and foundation details, structural geometries, interfacing and coordinating with PA Engineers and other consultants from other disciplines on this project.
- ❒ Performing constructability and construction staging study for entire project to be incorporated in the contract drawings and specifications.
- ❒ Preparing detailed construction drawings and procedure for the modification of the Upper Level Tolls Building. Drawings to clearly show all stages of construction for the Tolls Building including permanent and temporary bracing and support of building, abutment installation, and removal of existing bent wall. Temporary bracing and temporary support of the Tolls Building to be fully designed and shown on the contract drawings by the consultant. Procedure to include detailed description of all jacking and load transfer operations along with recommendations for equipment and instrumentation.
- ❒ Delivering to the Authority Contract Drawings, Specifications and final Construction Cost Estimate and Support Services during Construction based on the preliminary design of the project. (See Part V., Section B.1 and B.2) The consultant shall include an estimated quantity or drawings required, including a breakdown per structure in their proposal.
- ❒ Preparing seismic design of bridges and overpasses including retrofitting of existing 5 span bridge over the GWB Approach to resist earthquake loads elastically, so that structures remain open to normal traffic after a seismic event in accordance with the critical classification of these structures per the NJDOT and AASHTO.
- ❒ Preparing seismic retrofit on the existing Tolls Building to resist earthquake loads elastically to ensure that the ramp remains open to normal traffic after a seismic event.
- ❒ Preparing seismic design of all other structures of the ramp such as, but not limited to the retaining walls and open cut structures in accordance with the applicable codes.
- ❒ Prepare details and specifications for architectural concrete finishes.

- ❒ Preparing detailed performance criteria to aid contractor in designing temporary structures, except as noted above for the Tolls Building. Performance criteria to be in accordance with applicable codes and standards and to include rough sketches and conceptual drawings as required
- ❒ Perform Construction Support Services, but not construction inspection or supervision.

C. Description of Consultant Tasks

1. Task A. Meetings

   Attend kickoff meeting and subsequent review meetings, including preparation and submission of minutes for all meetings and inspections.

2. Task B. Field Verification

   a. Review all available documents included herewith as appropriate and research and collect any additional available data (i.e. contract drawings, shop drawings, reports, calculation books, etc.) from the Authority's files at the Port Authority Technical Center, the George Washington Bridge facility and at the Journal Square Transportation Center (Jersey City, NJ) to assist in completing the project. If specific data is not available, then a field trip to the site is warranted in order to retrieve data needed to complete the project. The Consultant shall estimate 96 hours of staff time for performance of this research. All documents shall be subject to the conditions defined in Section VI below.

   b. Conduct a site inspection of the existing PIP ramp complex and other structures that may impact design of the PIP ramps. Verify existing field conditions including taking all appropriate field measurements. Specific security requirements concerning access and personnel identification at the site must be followed. These requirements also include daily notifications to the GWB Police when entering and exiting the site.

   c. Provide all equipment including rigging, scaffolds, ladders and other necessary equipment, including traffic control devices, as required to inspect the structure and to maintain traffic under and over the structure. Staging of inspection work and nighttime access may be required.

   d. Submit copies of completed field findings and meet with Authority staff to discuss those findings.

3. Task C. Design Criteria Summary

   Prepare a detailed summary of all criteria to be used in the design including but not limited to loads (dead, live, wind, snow, vehicular, wheel, impact, or other loads as appropriate), material grades, codes and assumptions. As a minimum, the project shall be designed to AASHTO LRFD criteria, NJDOT Design Criteria, International Building Code, and seismic criteria of applicable code standards.

   The Design criteria shall also include assumptions selected for 1) the existing traffic lane closures, 2) construction staging requirements and 3) construction sequence. These items shall be coordinated with the Port Authority for meeting the Authority's bridge operational requirements, construction cost effectiveness and minimum construction duration. The design team shall also review the design approach shown in the Stage 2 Report and shall discuss (and coordinate) with the Authority any changes considered to the assumptions made in the report.

4. Task D. Final Design and Contract Documents

   Prior to the performance of this task you shall submit your specific Quality Control/Quality Assurance Program for the professional services to be performed in connection with the final design and the preparation of Contract Drawings and Specifications specified herein.

Upon completion of the contract documents required hereunder, you shall submit a letter to the Engineer certifying that you have you have performed the Quality Control/Quality Assurance Program as defined by you at the start of this Task.

a. Contract Drawings: Prepare a final design and Contract Drawings for work to be performed by a Contractor based on the approved Preliminary Design which shall include, but not be limited to, the appropriate work items contained in the foregoing tasks. The consultant shall include all applicable NJDOT requirements such as curing requirements for concrete, straightness and ride-ability of roadway surfaces on the Contract Drawings, if not addressed in the Technical Specifications. The Contract Drawings shall be prepared in conformation with the Port Authority CADD Standards and by using “Auto-CAD” computer program. For contract drawings, observe the following signature procedures:

   1) Sign and seal all drawings prepared by you.

   2) Any subconsultant shall sign and seal its own drawings. The Consultant's logo shall appear on each drawing prepared by a subconsultant.

   3) All drawings prepared for New Jersey Contracts shall be signed and sealed by a Principal of the firm with a New Jersey Professional Engineer's or New Jersey Registered Architect's License. The original tracing shall be backshaded with carbon paper in the area of the embossed seal so that the seal will print. In addition, following shall be placed below the seal:

Original Sealed and Signed By:
____
N.J.P.E. # or N.J.R.A. #

b. Design Calculations and Diagrams: The Consultant shall submit complete design computations and design drawings covering all structural framing and supports, such as primary framing members, bracing, foundations, siding, girts, roofing and architectural finishes. Calculations shall be submitted with 50%, 90%, and 100% submission of the contract drawings. Final submission of calculations shall be bound.

   Calculations shall clearly distinguish between new and existing construction. Distinct, independent, and complete calculations are required for each structure. Documents from which existing dimensions and existing member properties were obtained shall be referenced in the calculations.

   All engineering calculation sheets shall be numbered, dated, and indexed. The designer and checker shall initial all engineering calculations. The index sheets shall define the total number of the sheets submitted and shall bear the seal and signature of an experienced engineer holding a Professional Engineer's license in the State of New Jersey and who is familiar with and responsible for the design.

   If computations are submitted in computer print-out form, furnish the following:

   1) Description and proof of adequacy of the program. The description of each program shall include:

      ❒ The type of problems solved by the program.

- The nature and extent of the analysis.
- The assumptions made in the program.
- Instructions for interpreting the computer output format.

2) Indicate the design criteria used and the diagrams showing the loading conditions and loading combinations.
3) The design constants and equations used, including all references.
4) Submit indexed and clearly identified input and output sheets for the entire structure or for those portions of the structure that will be sufficient to enable the Authority to evaluate the structure. Clearly label all data on output sheets used for calculations Cross-reference to output sheets in calculations.
5) Submit a clear diagram of all member forces (axial, shear, bending or other forces, as appropriate) for each loading condition controlling the design.

c. Specifications: Prepare Specifications to include the work specified under subparagraph 2 above in accordance with the following:

1) Division 1 - Provide the following information for the Authority Standard Division 1 Specifications which will be prepared by Authority staff:
   - Information specifically related to Conditions and Precautions, Staging, Available Property, Temporary Structures, and other General Provision Requirements of the subject contract.
   - A list of the Contract Drawings.
   - A list of unit price items, where appropriate, with description and estimated quantities for each item.
2) Technical Specifications
   - The Authority has prepared certain standard technical specifications, which will be made available in hard copy as requested by the Consultant. These standard technical specifications must be used by the Consultant and may not be altered or revised in any way by the Consultant. Since these standard Technical Specifications may contain materials and related procedures that are not appropriate to the specific Contract being proposed, the contract drawings must clearly define the materials and scope of work. Division 1 of the Authority's specifications dealing with general provisions, includes the following language:

     "In case of a conflict between a requirement of the Contract Drawings and a requirement in Division 1 of the Specifications, the requirement of Division 1 shall control. In case of a conflict between a requirement contained in other Divisions of the Specifications and a requirement of the Contract Drawings, the requirement of the Contract Drawings shall control."
   - The Consultant shall prepare any technical specifications that are not available from the Authority. Any technical specifications prepared by the Consultant shall be in the same format as the Authority standard technical specifications and the Consultant shall make any changes therein requested by the Authority throughout its various reviews.

- The Consultant shall comply with the "Instructions to Consultants for Preparation of Port Authority Construction Contract Documents" which will be made available upon commencement of work by the Consultant.
- Other than hard copies of specifications prepared by the Consultant that are to be submitted to the Authority as noted herein, the Consultant shall submit 3 1/2" floppy disk copies of said specifications. One disk copy shall contain the specifications in the format of the original word processing program used by the Consultant, and shall labeled to clearly indicate the contract title, the name of the word processing program used and the revision number of said program. Another disk copy of the software program shall be submitted in ASCII format.

5. Task E. Request for Qualifications Development

   The Consultant shall assist Authority staff as required to develop a Request for Qualifications (RFQ). The RFQ shall be developed for the purpose of identifying qualified contractors for performance of the contract documents prepared hereunder.

   The Consultant shall estimate 40 staff hours for performance of this task.

6. Task F. Cost Estimate and Construction Schedule

   Prepare a Construction Cost Estimate based on the final Contract Drawings and Specifications and in accordance with the Authority's "Construction Estimating Guide", a copy of which is available from the Project Manager. Provide an estimate of the time required to complete construction, as well as an estimate of delivery time for all long lead-time items. Present the Construction Schedule in bar chart form using days, weeks or months as appropriate for the unit of time.

7. Task G. Post Award Duties

   a. Submit your specific Quality Control/Quality Assurance Program for the professional services to be performed in connection with the performance of your Post Award Duties specified hereunder.
   b. Review and respond to Requests for Information (RFI) from the Contractor, as requested by the Authority.
   c. Review, and approve or disapprove all working drawings, construction procedures, jacking procedures, catalog cuts and samples for conformance with the Specifications and Contract Drawings within 10 working days after receipt of said articles from the Contractor, for those articles for which you are Engineer-of-Record. Indicate any corrections and additions as required. Advise the Authority thereof giving the reasons for your decisions. Make all required distributions through final approval. Six copies of each working drawing will be required.
   d. In addition to any on-site observations you may require as the Engineer-of-Record, attend, at the request of the Authority, two pre-construction meetings and one field meeting for each month of the construction period.
   e. When requested, assist the Resident Engineer in the field to ensure that construction procedures such as jacking, temporary bracing and support of the Tolls Building, and caisson socketing into rock are properly implemented.
   f. Prepare and submit, at the first pre-construction meeting, an outline list of required contractor's submittals to include but not be limited to, working drawings, catalog cuts, samples, certificates and test reports.

g. Upon completion of construction, modify the Contract Drawings to "as-built" conditions and certify the same. The Authority will furnish the "as-built" information to be verified and incorporated.

h. Evaluate alternative construction details and materials, as requested by the Authority.

i. Make post-award contract changes with detailed estimates and make site inspections as required for the changes.

D. Schedule and Submissions

1. Submit the work identified above for review by the Chief Engineer within the number of calendar days stipulated below after receipt by you of one copy of the Agreement executed by the Authority.

2. Submit a design criteria summary within 30 calendar days. Authority comments will be forwarded to you within 7 calendar days after receipt of said submission.

3. Submit major deviations from the design approach shown in the preliminary design of the project (See Part V., Sections B.1. and B.2. below.) for Authority approval before proceeding with the design.

4. Submit your specific Quality Control/Quality Assurance Program for the professional services to be performed in connection with the final design and the preparation of Contract Drawings and Specifications specified herein within 14 calendar days.

5. Submit 30 sets of collated and stapled ozalid prints and one sepia set of 50% complete Contract Drawings, 30 printed sets of Technical Specifications, design calculations and information required for Division 1 of the Specifications within 120 calendar days. Authority comments will be forwarded to you within 14 calendar days after receipt of said submission. The Consultant will annotate with a response to each comment within 7 calendar days after receipt of comments.

6. Submit 15 sets of collated and stapled ozalid prints of 90% complete Contract Drawings, 10 sets of Technical Specifications, design calculations and information required for Division 1 of the Specifications within 220 calendar days. Authority comments will be forwarded to you within 14 calendar days after receipt of said submission.

7. Submit thirty sets of collated and stapled ozalid prints and one sepia set of 100% complete Contract drawings, thirty printed sets of Technical Specifications, 100% complete design calculations signed and sealed by a licensed Professional Engineer, and a final construction cost estimate within 240 calendar days. Authority comments will be forwarded to you within 21 calendar days after receipt of said submission.

8. Submit a letter to the Engineer certifying your compliance with the Quality Control/Quality Assurance Program established by you for the preparation of Contract Documents as required herein, along with one set of original mylar tracings of the complete Contract Drawings, reproducible masters of the Technical Specifications and a final construction cost estimate within 14 calendar days.

9. Submit your specific Quality Control/Quality Assurance Program for the professional services to be performed in connection with the performance of your Post Award Duties specified hereunder within 14 calendar days.

10. The completed Contract Drawings and Technical Specifications submitted above will be reviewed by the Authority's Law Department within 21 calendar days after receipt thereof. The Consultant shall make any changes to the Contract Drawings and Technical Specifications resulting from this legal review and submit the revised Contract Drawings and Technical Specifications, which will be used for bidding purposes, within 7 calendar days after receipt of Law Department comments.

E. Information and materials Provided by the Authority

The Authority will make available for the Consultant's information certain documents specified below. The documents specified under "A" below were not prepared for the purpose of providing information for the Consultant under the present work but they were prepared for other purposes, and do not form a part of this Agreement. The Authority makes no representation or guarantee as to, and shall not be responsible for, their accuracy, completeness or pertinence, and, in addition, shall not be responsible for the conclusions to be drawn therefrom. They are made available to the Consultant merely for the purpose of providing him with such information as is in the possession of the Authority, whether or not such information may be accurate, complete or pertinent, or of any value to the Consultant.

The documents specified under B below were prepared for the subject work and form a part of this Agreement. The Stage 2 Report (see Section B.1.) and the GWB 244.165 Contract Drawings (see Section B.2.) were prepared as the preliminary design of the project and do not provide all the details required to complete the work as stipulated in Part I. The consultant shall prepare final contract drawings and documents to complete all the work required by this RFP document. The scope of work shall also include all the items listed in the "Preliminary Design Estimate" in the Stage 2 Report (see Section B. 1. below) such as overhead sign bridges, miscellaneous roadway signs, light pole foundations, framing for Tolls Building soffit and fascias, utility support framing at bridge structures, armored expansion joints, drainage troughs at bridge expansion joints, bearings, chain link fence along Service Street parapet wall and along the north and south walls of the cut section, temporary bridges at the Tolls Building, specialized drainage structures, architectural finishes and the demolition of two residential homes, taxi garage, and salt hopper building with attached steel bridge.

Said document are as follows:

1. Existing Contracts
2. Documents
    a. Stage 2 Report for the GWB-PIP Connection Ramp to Lower Level Roadway
    b. Drawings developed as Contract GWB-244.165 numbered G1, G2, G3, G5, C1, C3 through C25, S01 through S05, S08, S10, S11, S15, S16A, S16B, S18 through S22, S25 through S49, E4 through E12, T1 through T9
    c. Existing Survey data
    d. Port Authority CADD Standards

    All documents, as well as Authority standards, Authority mylar tracings and examples of Authority specifications will be made available to the recipients of the Request for Proposals and the Consultant from the Project Manager listed herein at the Port Authority offices in Port Authority Technical Center located at 241 Erie Street, Jersey City, New Jersey 073108 during regular business hours.

F. Additional Information for the Preparation of Contract Documents and Construction Cost Estimates

1. Except as otherwise noted herein, the Preparation of Contract Documents shall conform to Port Authority standards and codes which would be applicable if the Authority were a private corporation and, in case of a conflict, the more stringent requirement shall apply.
2. Prepare all Contract Drawings on standard size Port Authority mylar tracings. Contract Drawings may be reduced to one-half size prints before distribution to contractors for bidding. Tracings shall, therefore, be prepared in such manner as to produce clearly legible drawings after reduction. Scales shall be graphical rather than numerical.
3. Meet with the Authority and incorporate Authority comments after submittals.

4. The following additional services shall be provided as part of this Agreement:
   a. Answer questions asked of the Authority staff by bidders during the bid period.
   b. Prepare Contract addenda including Contract Drawing revisions and engineering calculations, as necessary or as requested by the Chief Engineer, for Authority approval and issuance by the Authority. Furnish originals for final printing.
   c. Conform Technical Specifications and Contract Drawings to addenda when directed by the Chief Engineer after award of the Contract.
   d. Upon request, assist Authority staff in Items E1, E3 and E4 specified below.
5. Authority staff will:
   a. Prepare Information for Bidders, Form of Contract, Division 1 of the Specifications and the Analysis of Bid and Contract Progress Schedule.
   b. Meet with Consultant from time to time to review all Specifications, Contract Drawings, construction cost estimates and schedules prepared by him.
   c. Review with, and transmit comments from, various Authority Departments to the consultant for incorporation by him into the Contract Documents.
   d. Review addenda with and obtain approval of various Authority Departments.
   e. Solicit, receive, open bids, and award Contract or reject bids.

G. Conditions and Precautions

1. General

The Consultant shall immediately inform the Authority of any unsafe condition discovered at any time during the course of this work.

Vehicular traffic on George Washington Bridge shall always have priority over any and all of the Consultant's operations

2. Work Areas

The Consultant shall limit his inspection work to the areas necessary for the performance of such inspection and shall not interfere with the operation of the facility without first obtaining specific approval from the Chief Engineer.

During all periods of time when he is not performing operations at the work site, the Consultant shall store all equipment being used for the inspection in areas designated by the Chief Engineer and shall provide all security required for such equipment.

The Consultant shall not permit any objects or pieces of equipment to lie unattended on sidewalks, roadways or structures at any time.

3. Work Hours

The Consultant shall perform his work at the site between the hours of 8:00 A.M. and 4:00 P.M., Monday through Friday, where traffic operations are not affected and during off-peak periods (usually between 10:00 A.M. and 3:00 P.M. or 10:00 P.M. to 5:00 A.M.) unless otherwise directed by the Chief Engineer.

In any case, no work shall be performed at the site on a legal holiday or day before a legal holiday of either the State of New York or the State of New Jersey.

3.2.9 TEB

3.2.9.1 TETERBORO AIRCRAFT LOADS

Model DC-9					
	-15	-21	-32	-41	-51
Maximum Ramp Weight	91,500 lb (41,504 kg)	101,000 lb (45,813 kg)	109,000 lb (49,442 kg)	115,000 lb (52,163 kg)	122,000 lb (55,338 kg)
Percent of Weight on Main Gear	See Figures 3-N1 through 3-N5				
Nose Tire Size	26 x 6.6 Type VII				
Nose Tire Pressure	118 psi (8.3 kg/cm^2)	131 psi (9.2 kg/cm^2)	140 psi (9.9 kg/cm^2)	148 psi (10.4 kg/cm^2)	157 psi (11.0 kg/cm^2)
Main Gear Tire Size	40 x 14-46 20 pr	40 x 14-16 22 pr	40 x 14-16 24 pr	41 x 15-18 24 pr	41 x 15-18 24 pr
Main Gear Tire Pressure	130 psi (9.1 kg/cm^2)	143 psi (10.1 kg/cm^2)	155 psi (10.9 kg/cm^2)	160 psi (11.3 kg/cm^2)	172 [so (12.1 kg/cm^2)

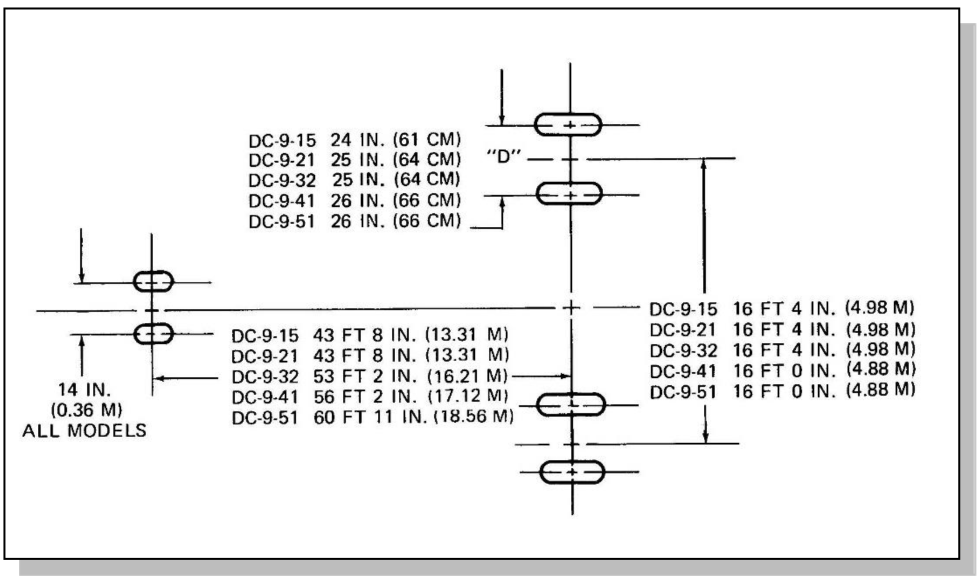
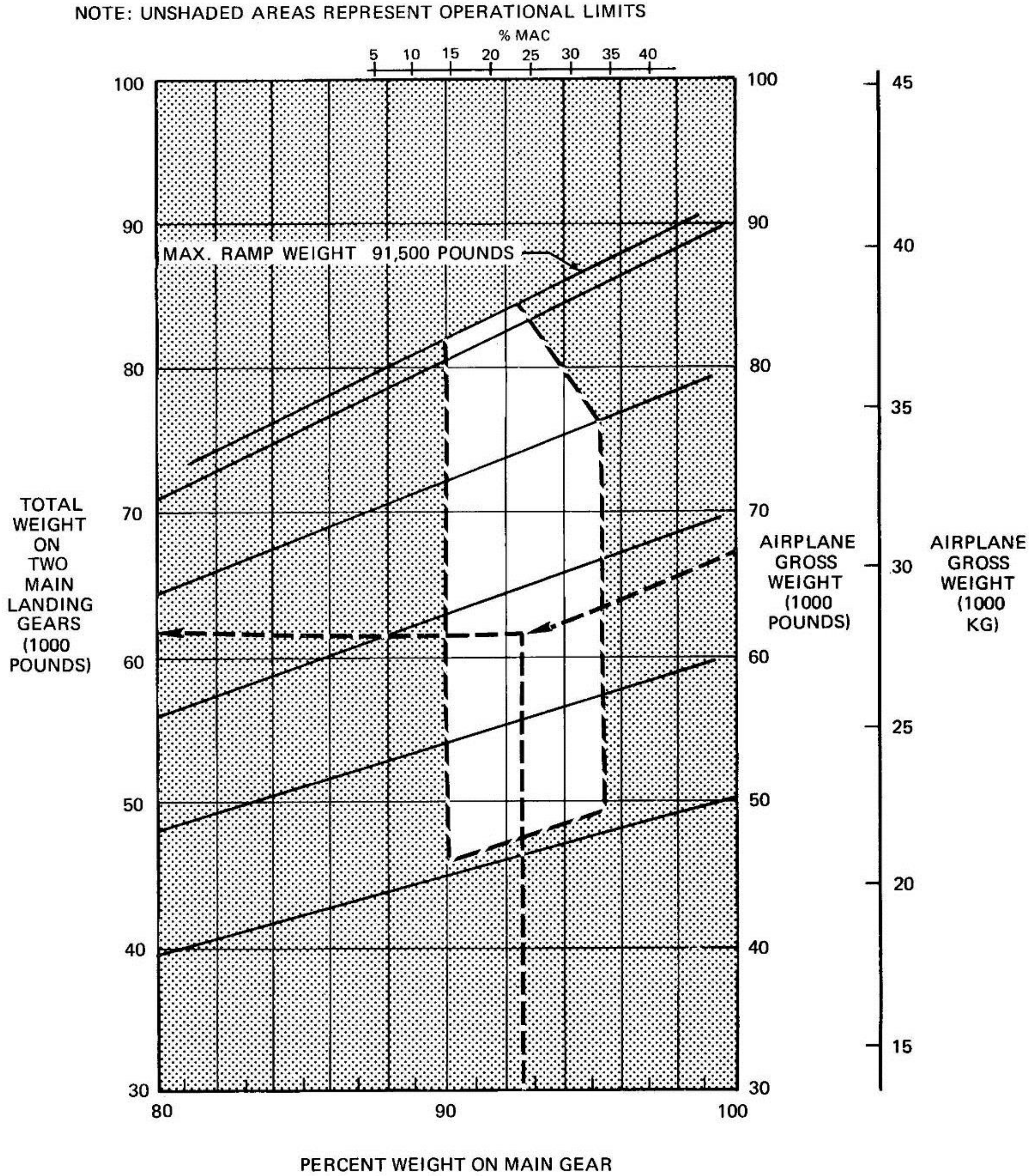


Figure 3-N
TEB Aircraft Loads



7.4 LANDING GEAR LOADING ON PAVEMENT
MODEL DC-9-15

Figure 3-N1

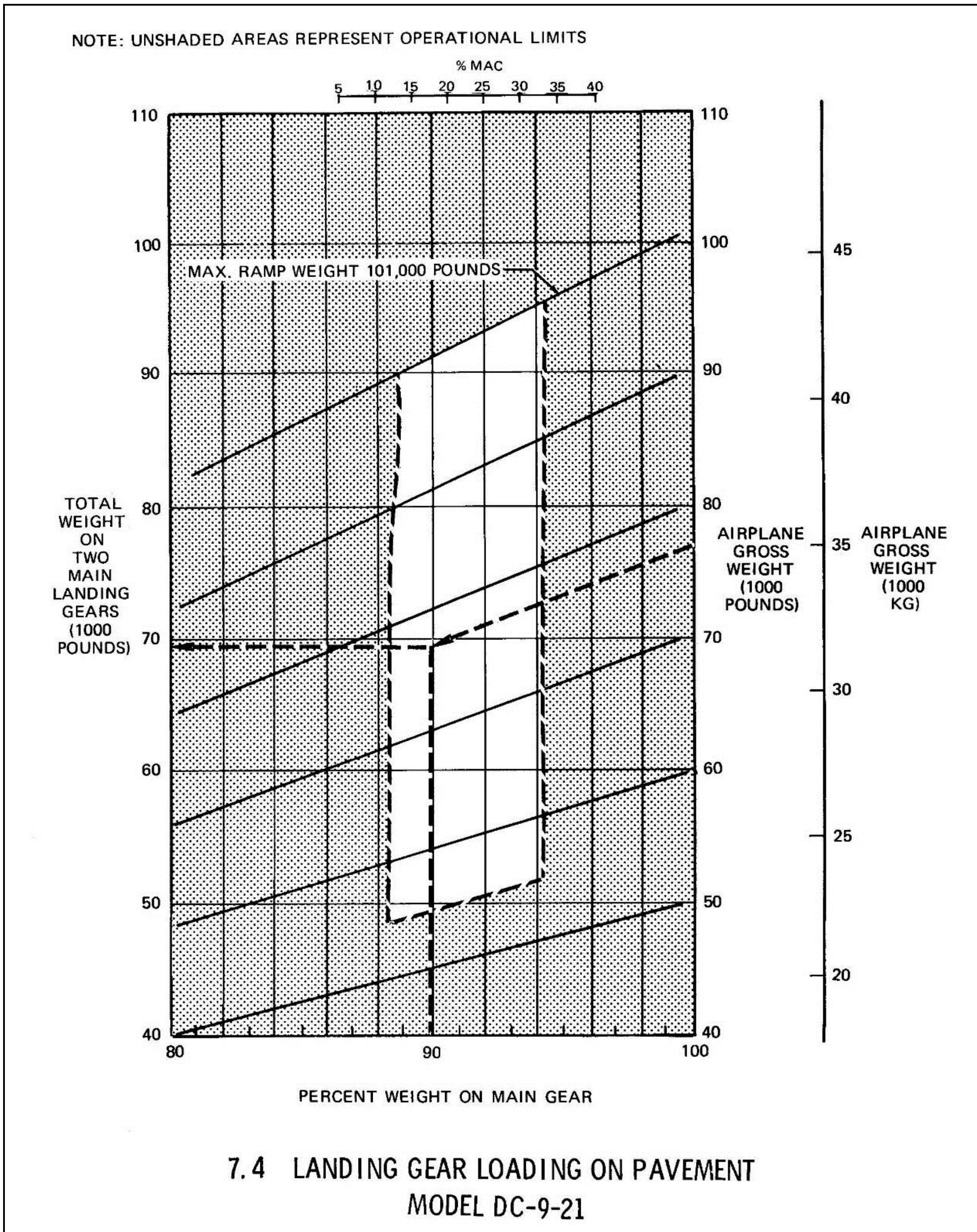

NOTE: UNSHADED AREAS REPRESENT OPERATIONAL LIMITS
% MAC
5 10 15 20 25 30 35 40
110
100
90
80
70
60
50
40
MAX. RAMP WEIGHT 101,000 POUNDS
TOTAL WEIGHT ON TWO MAIN LANDING GEARS (1000 POUNDS)
AIRPLANE GROSS WEIGHT (1000 POUNDS)
AIRPLANE GROSS WEIGHT (1000 KG)
45
40
35
30
25
20
80
90
100
PERCENT WEIGHT ON MAIN GEAR
7.4 LANDING GEAR LOADING ON PAVEMENT
MODEL DC-9-21

Figure 3-N2

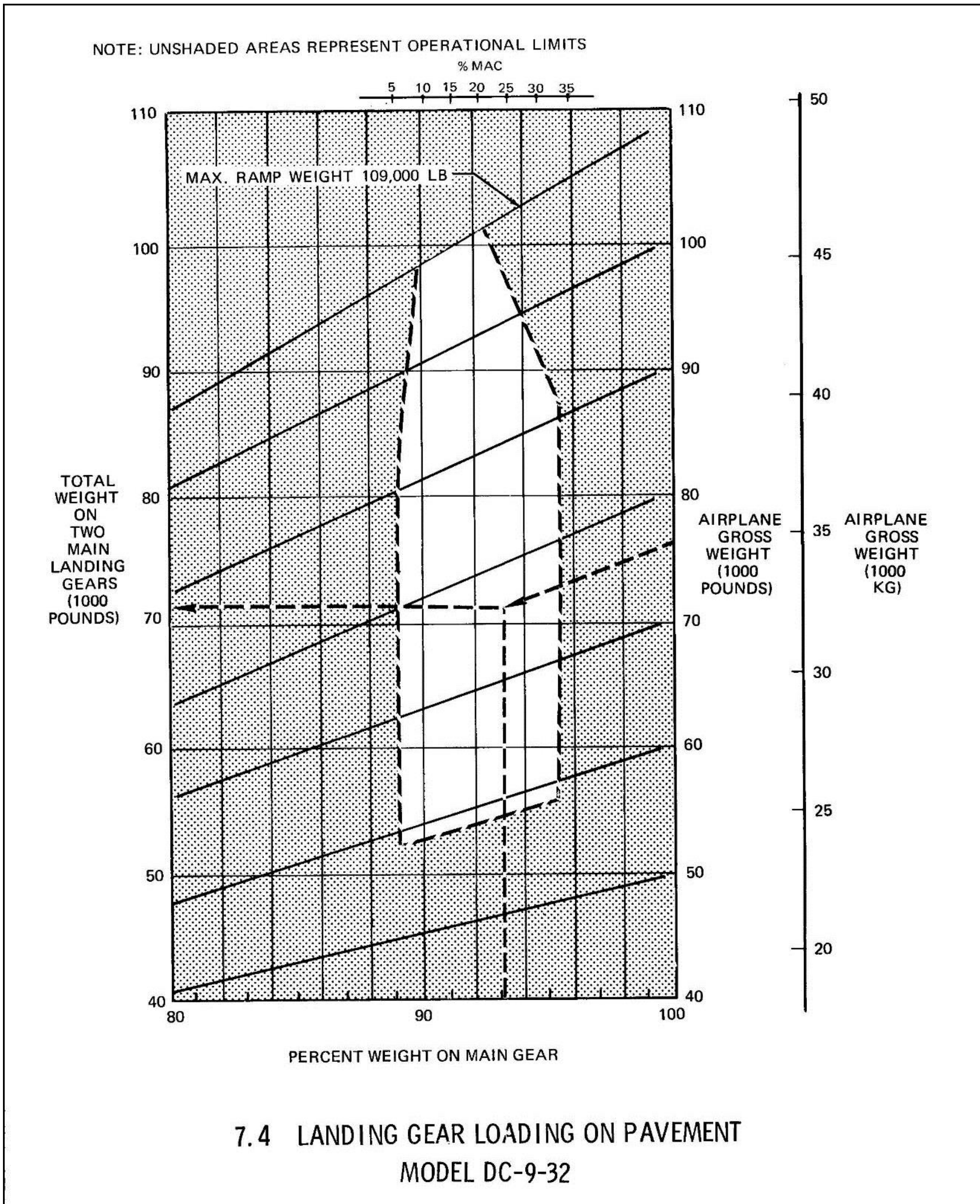
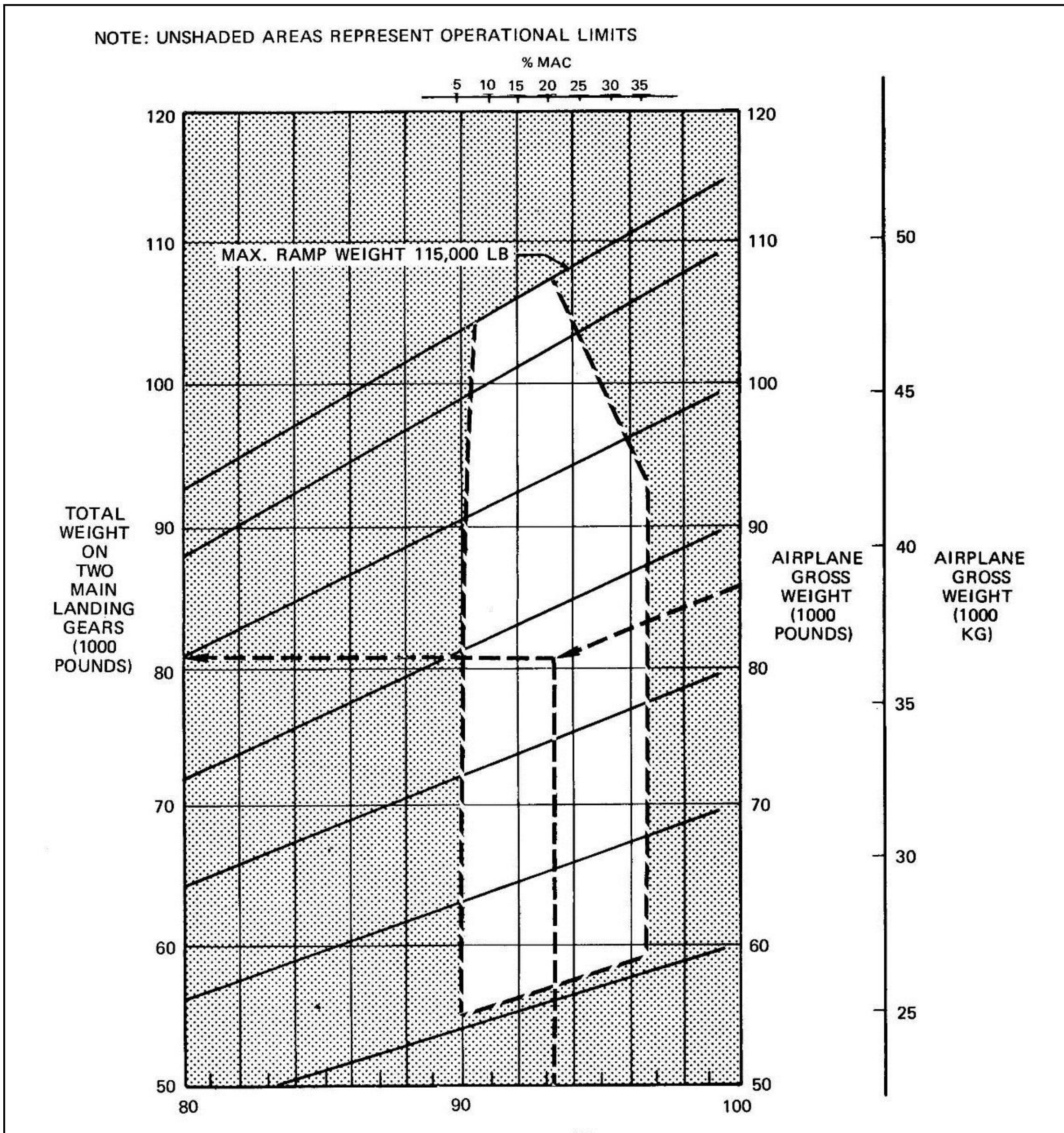

NOTE: UNSHADED AREAS REPRESENT OPERATIONAL LIMITS
% MAC
5 10 15 20 25 30 35
MAX. RAMP WEIGHT 109,000 LB
TOTAL WEIGHT ON TWO MAIN LANDING GEARS (1000 POUNDS)
AIRPLANE GROSS WEIGHT (1000 POUNDS)
AIRPLANE GROSS WEIGHT (1000 KG)
PERCENT WEIGHT ON MAIN GEAR
40
50
60
70
80
90
100
110
20
25
30
35
40
45
50
80
90
100

Figure 3-N3



7.4 LANDING GEAR LOADING ON PAVEMENT
MODEL DC-9-41

Figure 3-N4

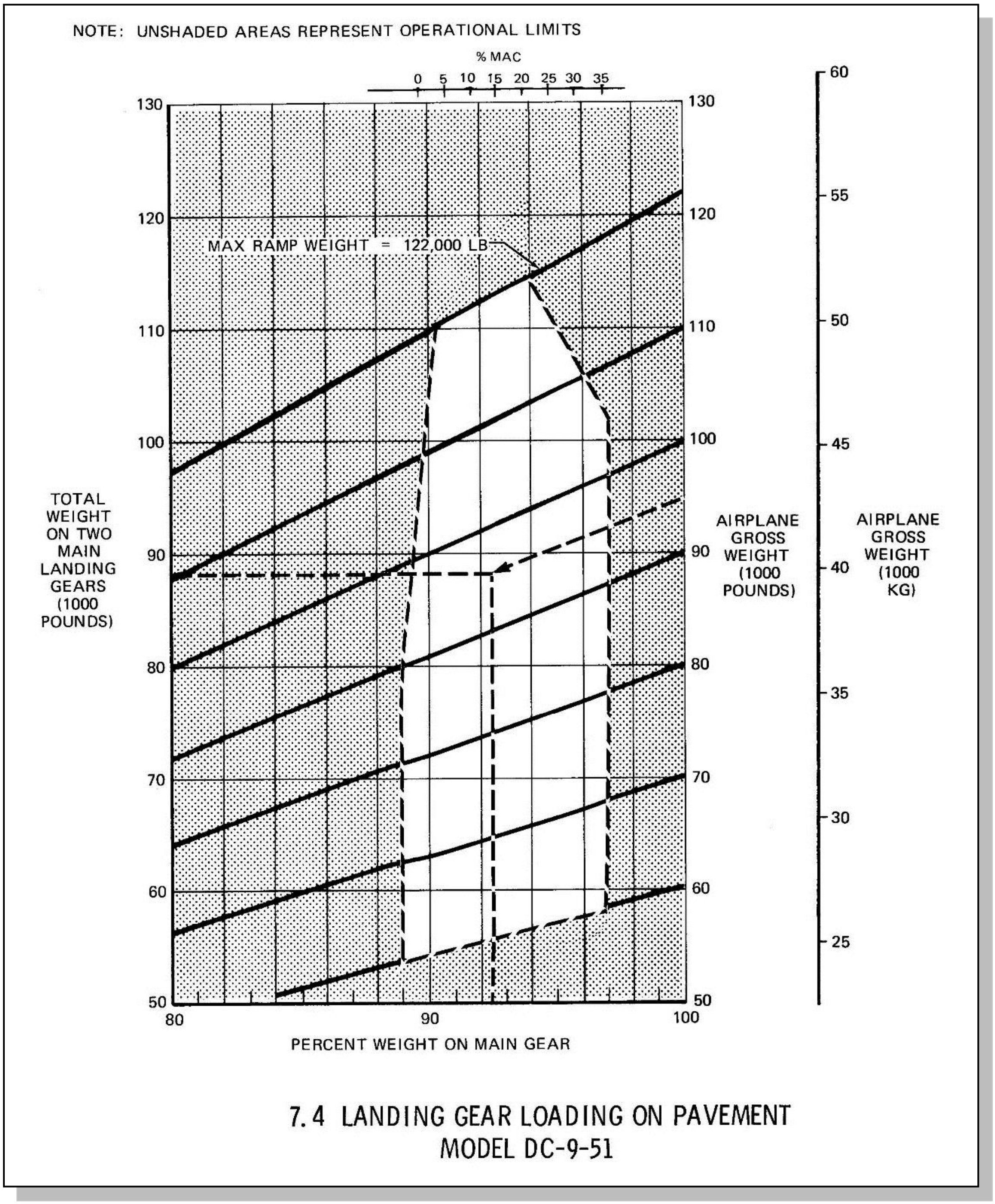

NOTE: UNSHADED AREAS REPRESENT OPERATIONAL LIMITS
% MAC
0 5 10 15 20 25 30 35
MAX RAMP WEIGHT = 122,000 LB
TOTAL WEIGHT ON TWO MAIN LANDING GEARS (1000 POUNDS)
AIRPLANE GROSS WEIGHT (1000 POUNDS)
AIRPLANE GROSS WEIGHT (1000 KG)
130
120
110
100
90
80
70
60
50
60
55
50
45
40
35
30
25
80
90
100
PERCENT WEIGHT ON MAIN GEAR
7.4 LANDING GEAR LOADING ON PAVEMENT
MODEL DC-9-51

Figure 3-N5

3.3 PREFERRED PRACTICES

3.3.1 CONSTRUCTION METHODS

3.3.1.1 CURVED GIRDER ERECTION PROCEDURE (GWB PIP RAMP - 2008)

Erection Notes for Curved I-Girders (GWB PIP Ramp, Three-span bridge over Hudson Terrace)

A. The entity performing the steel fabrication shall have a minimum of ten years' experience in fabricating built up welded steel curved girders, and shall be certified under the American Institute of Steel Construction Quality Certification Program in categories for Major Steel Bridges (CBR).

B. The steel fabricator shall be certified under the American Institute of Steel Construction quality certification program in Categories for Advance Steel Erection (CASE).

C. The entity performing steel erection shall have a minimum of ten years' experience in erecting built-up welded steel curved girders.

D. The erector shall have completed two projects of the type of construction shown in contract drawings with in last ten years, each in excess of 1000 tons.

E. The contractor shall submit a detail erection procedure signed and sealed by a Professional Engineer licensed in State of New Jersey for review which shall include but shall not limited to the traffic control plans, detail step by step erection method, construction equipment to be used, erection schedule, drawing showing crane locations and delivery truck locations and design calculations.

F. Revised drawing shall be resubmitted each time the contractor proposes to deviate from the sequence or schedule of erection in the previously approved drawings.

G. The contractor shall be responsible for the design, supply, installation and removal of work bridges, erection bracing, temporary wind bracing and lateral stability bracing for girders as required during all the handling operations, including loading, transporting, staging and erection of girders.

H. The actual erection methods and sequences employed by the contractor may have a substantial effect on the final steel profile. The contractor shall be responsible for taking all necessary compensatory action to ensure the required alignment and profile of the erected steel.

I. For each Girder, a table of theoretical elevations and alignment of the geometry control points shall provide with the erection procedure.

J. Thermal expansion/contraction shall be considered at the time of erection.

K. Horizontal and vertical alignment of the bearings and anchor bolts should be accomplished before erection begins.

L. It is recommended that the full curved girder span from abutment to pier shall be installed as a trail installation in the shop with diaphragms and splices to ensure proper fit up in the field. Assembly in the shop shall be properly blocked, pinned and bolted.

M. It is recommended elevations at the bridge seats in the shop shall be set to match surveyed top of bearing and bearing spacing of field installed bearings.

N. The erector should review the shop assembly blocking records to determine the effect of chamber fabrication tolerances on the final shape of the structure.

O. Elevations should be set to account for tolerance and to match shop blocking dimensions. Additional cranes may be used as a substitute for false work.

P. False work shall be provided near splices and stiffeners. Jacking reaction capability shall be provided in the false work to adjust and necessary to maintain required elevations.

Q. During staging and hauling, girders shall be maintained in up right position and shall be supported within 2 feet of the inside of the bearings area, properly blocked at intermediate locations and laterally braced as required.

R. Before erection of the girders, the contractor shall verify the length of the girders, the layout of the substructure units, the elevations of bearings seats and location of the anchor bolts.

S. The girders shall be picked only by the lifting devices and should have the lifting capacity of two times the lifting weight. The point of lifting should be checked for buckling stress of flange due to steel dead load only during erection procedure or specify other means of temporary bracing.

T. The girder should be handled carefully at all times during any movement with particular care being taken to ensure that bearings are not damaged.

U. Bolt should not be tightened until horizontal and vertical alignment of the members has been achieved.

V. Falls work shall only be removed after complete installation of I-girders this includes achieving vertical alignment and tightening of anchor bolts at bearings.

W. Contractor shall provide necessary supports during concrete pour to the deck to avoid lateral movement and rotation in the I-girders.

X. Following minimum information shall placed on the erection drawings:

1. Plan of the work area showing support structures, roads, utility or any other information pertaining to erection.
2. Erecting sequence for units, noting use of holding cranes or temporary supports, false work etc.
3. Delivery location of each unit and storage location if required.
4. Location and range of each pick.
5. A capacity chart of each cranes and boom length used in the work.
6. Pick point locations on each member.
7. Lifting weight of each member.
8. Lift and setting radius of each pick (or maximum lift radius).
9. Description of lifting devices or other connecting equipment, including capacity.
10. Beam tie down details or other method of stabilizing erected beam units, if required.
11. Blocking details for stabilizing members supported on expansion bearings and on bearings that do not limit movement in the transverse direction.

3.3.1.2 AISC Construction Methods [35]

3.3.1.3 ACI Construction Methods [36]

3.3.1.4 AREMA Electronic Document Library [37]

3.3.1.5 Modern Steel Construction [38]

3.3.2 Design Methods

3.3.2.1 AISC Design Methods [39]

3.3.2.2 ACI Design Methods [40]

3.3.2.3 AREMA Electronic Document Library [41]

3.3.2.4 Modern Steel Construction [42]

3.4 Design Guides

3.4.1 Guides for Steel Designs

3.4.1.1 Are You Properly Specifying Materials? [43]

3.4.1.2 AISC Downloadable Tools for Steel Design [44]

3.4.1.3 AISC Technical Library [45]

3.4.1.4 Not Used

3.4.1.5 AISC Frequently Asked Questions [46]

3.4.2 Guides for Concrete Design

3.4.2.1 ACI Technical Questions [47]

3.4.2.2 ACI E702 Design Examples [48]

3.4.2.3 ACI 318 Design Case Studies (Membership Required) [49]

3.4.2.4 ACI 318 Questions and Answers (Membership Required) [50]

3.4.2.5 ACI 318 References (Membership Required) [51]

3.4.2.6 Notes on ACI 318 Building Code Requirements for Structural Concrete with Design Applications (52)

3.5 OSHA Requirements for Structural Engineers

3.5.1 OSHA Steel Erection Standard — Summary

The OSHA Steel Erection Standard is provided in OSHA Instruction, Directive Number: CPL 1-1.34, dated March 22, 2002. The requirements in the instruction that apply to design and detailing of steel for structural engineers is summarized in the **November 2002 issue of the "STRUCTURE"** engineering periodical.

3.5.2 OSHA Instruction Directive CLP 2-1.34 (Steel Erection) (53)

3.5.3 OSHA Subpart R Steel Erection (54)

3.5.4 New OSHA Erection Rules, How They Affect Engineers (55)

3.6 Construction Standards - Inspection, Testing and Maintenance Requirements for Fire Protection and Life Safety Systems (56)

3.7 Condition Surveys Guidelines

3.7.1 Condition Surveys of Buildings (57)

3.7.2 Condition Surveys of Waterfront Structures (58)

3.7.3 Condition Surveys of Tunnels (59)

3.7.4 Condition Surveys of Sign Lighting Structures (60)

3.8 Products

3.8.1 Anchor Bolts

3.8.1.1 General Requirements

Nuts for anchor bolts and anchor rods shall include a lock washer.

Do not load adhesive anchor bolts, rods, or rebars until the adhesive or grout obtains full strength in accordance with the manufacturer's requirements.

Prior to drilling holes for any anchor bolts into existing concrete, use rebar locator system to locate existing reinforcement. Where required to clear reinforcement, shift locations of hole to the tolerance shown on the contract drawings. Misaligned, out of plumb, and out of level bolts shall not be accepted.

If necessary and approved by the engineer, supplement the use of the rebar locator system with 1/8 inch drilled pilot holes to ensure clearing the existing reinforcement.

Submit catalog cut and data on rebar locator system for review and approval.

The rebar locator system shall be capable of accurately locating and recording ferrous objects using a calibrated scanner and monitor that is specifically designed for detecting rebars in concrete. The system shall be capable of recording and saving scoped images and shall accurately locate rebars dimensionally utilizing a built-in grid system. The system shall be capable of monitoring functions and displaying on screen dimensional information.

All holes shall be drilled level and/or plumb. Use jig where required to ensure level and/or plumb holes.

3.8.1.2 ADHESIVE ANCHOR BOLTS IN OVERHEAD APPLICATIONS

The use of adhesive anchors in overhead applications is not allowed.

3.8.1.3 ADHESIVE ANCHORS IN BUILDINGS

The use of exposed adhesive anchors (not encased in concrete or fireproofing) is not allowed in buildings.

3.8.1.4 ANCHOR BOLT PULL TEST REQUIREMENTS

Upon installation and prior to load application of overhead anchor bolts, the authority shall pull test 5% of all anchor bolts installed or a minimum of 5 anchor bolts, whichever is greater. The pull test load shall be 2 (two) times the working load of the anchor bolt. The contractor shall coordinate this effort with the Authority and provide access and the necessary support for the Authority to perform the testing. Engineer shall specify anchor bolt working load on drawings.

3.8.1.5 GUIDELINES FOR THE DESIGN OF DRILLED-IN CONCRETE ANCHORS FOR OVERHEAD APPLICATIONS (61)

3.8.1.6 ANCHOR BOLTS IN PATH TUNNELS

In PATH tunnels, only expansion anchors should be used for hanging pipe and conduit supports and equipment, adhesive anchors should not be used.

In concrete, for loads less than 500 pounds use Hilti HSL heavy duty anchors or an approved equal. In concrete for loads in excess of 500 pounds use Hilti HDA undercut anchors.

In the brick sections of the tunnel use Hilti HLC sleeve anchors or an approved equal.

Provide the note below on drawings for testing of the anchors:

The Authority shall field test the installed anchors prior to loading to verify proper installation. No load shall be placed on any anchor until approved by the engineer. Notify the engineer when anchors are ready for testing. Anchors shall be loaded to two times the allowable working load. Anchors not meeting the above criteria shall not be loaded and additional anchors shall be installed at the manufacturer's recommended minimum distance from the original anchor. The contractor shall provide the necessary equipment and/or scaffolding to allow access to the installed anchors in order for the engineer to perform the stated testing.

3.8.1.7 ANCHORING TO CONCRETE, ACI 318-05, APPENDIX D (62)

3.8.1.8 FHWA TECH ADVISORY – USE AND INSPECTION OF ADHESIVE ANCHORS (63)

3.8.2 SILICONE SEALER

Install silicone sealer in accordance to the manufacturer's requirements and Standard Specification Section 07920. Sealant for caulking shall be ES-2 silicone sealant. Joint sealant backing shall be elastomeric tubing.

3.8.3 GROUT BELOW BASE PLATES

Grout used below column base plates shall be non-shrinking, non-metallic grout in conformance with Standard Specification Section 03602.

3.8.4 PROHIBITED PRODUCT LIST

3.8.4.1 CREOSOTE USE BAN

In 2007 the governors of both New York and New Jersey signed amendments to existing legislation that prohibits the use of creosote for any purpose except railroad operations and utility poles. This became effective in New Jersey on July 1, 2007 and in New York on Jan 1, 2008. Therefore, any standard specification that allows the use creosote treated wood should be revised in accordance with these amendments.

3.8.5 SWEET'S CATALOG (PRODUCTS) (64)

3.8.6 THOMAS REGISTER (INDUSTRIAL PRODUCTS/SERVICES) (65)

3.8.7 AMERICAN SOCIETY FOR TESTING MATERIALS (ASTM) (66)

4.0 DETAILS, NOTES, AND CUSTOM SPECIFICATIONS

4.1 COMMON DETAILS

4.1.1 GENERAL

4.1.1.1 TITLE SHEET (67)

4.1.1.2 INDEX OF DRAWINGS, ABBREVIATIONS, & LEGEND (68)

4.1.1.3 PRECAST CONCRETE BOX CULVERS, NYSDOT (69)

4.1.1.4 PRESTRESSED CONCRETE, NYSDOT (70)

4.1.1.5 CONCRETE TRAFFIC BARRIERS, NYSDOT (71)

4.1.1.6 EXCAVATION AND EMBANKMENT, NYSDOT (72)

4.1.1.7 PROTECTIVE STRUCTURE FOR MANHOLES (73)

4.1.1.8 PROTECTIVE STRUCTURE FOR CULVERT - JFK (74)

4.1.1.9 DUCT HANGERS AND SUPPORTS (75)

4.1.1.10 OVERHEAD DETECTION SYSTEM - PABT (76)

4.1.1.11 CAMERA AND ANTENNA SUPPORTS (77)

4.1.2 AVIATION

4.1.2.1 JFK GUIDE SIGN FOUNDATION (78)

4.1.2.2 JFK STRUCTURAL NOTES (79)

4.1.2.3 LGA LIGHT POLE FOUNDATIONS (80)

4.1.2.4 NY AIRPORT TRANSFER PAD (81)

4.1.2.5 NY AIRPORT BLAST FENCE DETAILS (82)

4.1.3 HIGHWAY/ROADWAY

4.1.3.1 TEMPORARY ROADWAY DECK PLATING (83)

4.1.3.2 CHAIN LINK FENCE, NJDOT (84)

4.1.3.3 BEAM GUIDE RAIL, NJDOT (85)

4.1.3.4 BARRIER CURB, NJDOT (86)

4.1.4 BRIDGES

4.1.4.1 EXPANSION JOINT REPLACEMENT (87)

4.1.4.2 TEMPORARY ROADWAY DECK PLATING (88)

4.1.4.3 OBX TRUSS BEARING REPLACEMENT (89)

4.1.4.4 DECK REPAIRS AND CONCRETE REPAIRS - GOETHALS BRIDGE (90)

4.1.4.5 NOT USED

4.1.4.6 SLAB SPALL REPAIRS (91)

4.1.4.7 CONCRETE SPALL AND CRACK REPAIRS (92)

4.1.4.8 TEMPORARY BEAM SUPPORTS (93)

4.1.4.9 NOT USED

4.1.4.10 SEISMIC COLUMN JACKETING AND STRINGER RESTRAINERS (94)

4.1.4.11 NOT USE

4.1.4.12 NEOPRENE DRAINAGE TROUGHS (95)

4.1.4.13 NOT USED

4.1.4.14 SPALL AND FULL DEPTH SLAB REPAIRS (96)

4.1.4.15 BEARING REPLACEMENT (97)

4.1.4.16 BEARING STOOL REPLACEMENT (98)

4.1.4.17 BRIDGE SEAT REPAIRS WITH JACKING (99)

4.1.4.18 ABUTMENT DETAILS, NYSDOT (100)

4.1.4.19 INTEGRAL ABUTMENTS, NYSDOT (101)

4.1.4.20 SAFETY NETTING (102)

4.1.5 PORTS

4.1.5.1 BROOKLYN PIERS, PILE REPAIRS, FENDER SYSTEMS

4.1.5.1.1 Pile Extension, As-Builts (103)

4.1.6 RAIL

4.1.6.1 CONCRETE RAILROAD BRIDGES (104)

4.1.6.2 PATH Station Platform Stair Replacement (105)

4.1.6.3 Miter Rail Replacement (106)

4.1.7 Modular Building Foundation (107)

4.1.8 Ceilings (108)

4.1.9 Tunnels

4.1.9.1 Supplemental Deck and Underdeck Mesh Details (109)

4.1.9.2 Cracked CI Ring Repair (110)

4.1.9.3 CI Ring Bolt Replacement (111)

4.1.9.4 CI Ring Flange Seepage Repair (112)

4.1.9.5 Ring Bolt and Flange Lead Removal Notes (113)

4.1.10 Signs

4.1.10.1 Overhead Sign Structures

4.1.10.1.1 JFK Overhead Sign Structures

- Overhead Sign, Plan, El Section, Sheet 1 (114)
- Overhead Sign, Plan, El Section, Sheet 2 (115)
- Overhead Signs, Columns and Foundations (116)

4.1.10.1.2 Not Used

4.1.10.2 Not Used

4.1.10.3 Not Used

4.1.10.4 Breakaway Signs Supports (Ground Mounted), NJDOT [117]

4.1.10.5 Steel U-Post Sign Supports, NJDOT [118]

4.1.11 Concrete Repairs

4.1.11.1 Bridge Deck Partial Depth Spall Repairs [119]

4.1.11.2 Concrete Crack and Rebar Repairs [120]

4.1.11.3 Bridge Seat Repairs with Jacking [121]

4.1.12 Steel Repairs

4.1.12.1 Rivet Replacement [122]

4.1.12.2 Girder and Stringer Repairs [123]

4.1.12.3 Column Repairs [124]

4.1.12.4 Platform Framing for Storage Tanks

4.1.12.4.1 Platform Framing, Sheet 1-6 [125]

4.1.12.5 PATH Station Platform Stair Replacement [126]

4.1.12.6 Safety Netting at GWB [127]

4.2 Structural Notes

4.2.1 General

4.2.1.1 GWB Tolls Building Underpinning (Sample Structural Notes – 2008)

General

The dimensions shown on the contract drawings are derived from drawings of the original construction. They may vary from the actual conditions as exist in the field. Verify by field measurements all dimensions, curvatures, elevations of the existing structure that may be required to locate, align, dimension, detail any construction required to perform the work. Field measurements shall be performed prior to and coordinated with the preparation of shop drawings and the manufacturing, fabrication, and installation of all items of construction. All variations from the contract drawings shall be brought to the attention of the engineer prior to the preparation of the shop drawings. Dimensions based on field measurements taken by the contractor shall be clearly marked on the shop drawings.

The contractor shall exercise caution so as not to damage the existing structures to remain.

The contractor shall locate all existing utilities prior to excavation. The contractor shall exercise caution when removing concrete and excavating so as not to damage existing utilities.

Design, detailing, fabrication, and execution of construction shall conform to the requirements of the specifications and of current issues, including interims, of the following publications:

- ❒ AASHTO Standard Specifications for Highway Bridges.
- ❒ ACI Building Code Requirements for Structural Concrete and Commentary, ACI 318.
- ❒ Manual of Standard Practice for Detailing Reinforced Concrete Structures, ACI 315.
- ❒ American Welding Society Bridge Welding Code, D1.5.
- ❒ Occupational Safety and Health Administration (OSHA) Safety and Health Standards – 29 CFR 1926.

All calculations submitted to the engineer shall be signed and sealed by a Professional Engineer licensed in the State of New Jersey.

See electrical and civil contract drawings for existing utilities, installation of utilities, and relocation and reinstallation of existing utilities.

See geotechnical contract drawings for earthwork, foundation subgrade, temporary sheeting, rock excavation, condition survey/vibration monitoring. See geotechnical contract drawings for rock anchor/bolt material, rock anchor/bolt details, and rock anchor/bolt installation.

See Contract Drawing No. S003 for membrane and geocomposite drain requirements.

See civil contract drawings for tie in of the abutment wall drainage system into the lower level roadway drainage system and replacement of asphalt pavement at service street.

Within 90 days after receipt by him of the acceptance of his proposal, the contractor shall submit for review and approval a detailed construction procedure and schedule for modifying the tolls building following the steps shown on Contract Drawing No. S252.

Within 90 days after receipt by him of the acceptance of his proposal, the contractor shall submit for review and approval a detailed installation procedure with drawings and calculations for installing the support beams and needle beams. Procedure shall include temporary structures and supports, heavy duty scaffolding, lifting equipment (cranes, forklifts, etc.), and accessories such as rollers, winches, come alongs. The drawings and calculation shall be signed and sealed by a professional engineer licensed in the state of New Jersey.

The contractor shall inspect the existing upper level tolls building underdeck, both fascias, and the exposed sides of building bent walls at the eastbound lower level roadway to identify deteriorated concrete areas.

90 days prior to the demolition of the south bent wall of the tolls building, the contractor shall submit a sketches signed and sealed by a New Jersey Professional Engineer showing deteriorated areas and repair details. The contractor shall repair all deteriorated concrete. The contractor shall be compensated for such work at the net cost thereof. Work compensated for at the net cost shall include inspection of the concrete, repair sketch preparation, and repair of deteriorated concrete areas. For compensation of net cost, see Contract Drawing No. G006.

If required to clear the needle beam installed in step 10 of the “construction procedure” on Contract Drawing No. S252, re-support microwave detectors located on the east fascia as directed by the engineer. Contractor shall be compensated for such work at the net cost thereof. For compensation of net cost, see Contract Drawing No. G006.

The following items A, B, and C (see Contract Drawing No. S256) shall be as directed by the engineer and shall be compensated for at the net cost thereof. Work compensated for at a net cost shall include the design, fabrication, transport, installation, and removal of the temporary bridges. For compensation of net cost, see Contract Drawing No. G006. Drawings and calculations for the temporary bridges shall be signed and sealed by a Professional Engineer licensed in the State of New Jersey.

A. Temporary pedestrian bridge at excavation pit to the loading platform of the building.

B. Temporary pedestrian bridge at excavation pit to the rear door of the building.

C. Temporary vehicular bridge over the excavation pit at the east end of the building to the upper level access ramp. The vehicular bridge shall be designed for HS20 loading and shall be approximately 25 feet long with a roadway travel width of 11 feet. The bridge shall have a 3-foot-wide sidewalk with a handrail. Bridge shall be provided with fascia parapets. Ramps at the end of the vehicular bridge shall not exceed a 6% grade. The overall roadway superstructure height shall not exceed 24 inches to ensure that there is enough room for placement of the ramp atop the toll building before encountering the upper level toll plaza.

Steel and High-Strength Bolts

All structural steel shapes and plates shall be AASHTO M270 Grade 50 (ASTM A709 Grade 50) unless otherwise noted.

All steel shall meet the Charpy V-Notch Impact Requirements for Temperature Zone 2. Submit results of Charpy V-Notch testing to the engineer for approval prior to steel fabrication.

The structural steel fabricator shall be certified under the AISC Quality Certification Program in Categories for Major Steel Bridges (CFR).

The structural steel erector shall be certified under the AISC Quality Certification Program in the category for advanced certified steel erector (CASE).

All needle beams shall be assembled in the shop with the splice plates bolted in place to ensure proper fit-up. Contractor shall submit documentation indicating proper fit-up of splice plates. Bolts used in shop shall not be reused for field splicing.

The 82'-4" long support beams shall be shipped and installed as one unit. Field splicing shall not be allowed. The plate material required for the support beams and needle beams at the lengths required are long lead items. Contractor shall coordinate with fabricator to ensure beams are delivered on time and meet the schedule requirements.

All steel shall be fabricated straight. The maximum variation in straightness (camber) shall be 1-1/2 inch for the support beam and 1-1/2 inch for the full length erected needle beam.

Support beams and needle beams shall be installed with camber up.

Stainless steel for expansion bearings and building brace beams shall conform to ASTM A240 Grade 30, Type 304 and shall be polished to a No. 8 bright mirror finish.

High strength bolts for needle beams:

- ❒ The use of oversized or slotted holes is not permitted unless otherwise noted.
- ❒ Bolt threads shall be excluded from all shear planes.
- ❒ Splices shall require bolts that are tightened from the nut end of the bolt at the outside of the box beam.
- ❒ Bolts for splices shall be 1-1/8 inch diameter ASTM A325, ASTM F1852 Type 1, Tru-Tension fasteners as manufactured by Nucor or an approved equal. Install bolts with manufacturer's recommended installation tool and installation procedure.

Welding shall conform to AWS D1.5 Bridge Welding Code. Electrodes shall be E70 for carbon steel to carbon steel. Electrodes shall be E309-16 for carbon steel to stainless steel.

Use installed anchors as template for field drilling holes in base plates, masonry plates, and support beam flanges. As an alternate, contractor may shop drill holes using survey of installed anchors, unless otherwise noted.

Concrete and Grout Pads

Compressive strength (f'c) for all concrete including encasements and fillers shall be 4,000 psi.

All concrete, except encasements shall be Category IV in accordance with Contracts Unit Standard Specifications. https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx# (EOL hyperlink is an internal website)

All concrete encasements shall be Category V in accordance with Contracts Unit Standard Specifications. https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx# (EOL hyperlink is an internal website)

All concrete encasements shall be formed concrete, shotcreting shall not be allowed.

Submit formwork details (including pattern of reinforcement ties) and method of installing and supporting all formwork for review.

All formwork shall provide a clearance of no less than 16 feet to the lower level roadway below.

Reinforcement shall be AASHTO M31 (ASTM A 615) Grade 60.

All reinforcing bars shall be epoxy coated except at the north wall corbels. Epoxy-coated reinforcing bars shall conform to AASHTO M284 (ASTM A775).

Mechanical splices be shall develop 125% of the yield strength of the spliced rebar.

Concrete cover for reinforcement shall be 3 inches, unless otherwise noted.

Bar supports, spacer bars, and other devices and supports for securing reinforcement shall be plastic or plastic coated.

Tie wire for reinforcement shall be 16 gage or heavier conforming to AASHTO M32 (ASTM A82). Use Monel, stainless steel, plastic, or nylon coated wires.

Provide ½-inch chamfers at all edges and corners, unless larger chamfers are indicated on the contract drawings.

Color and finish of all exposed concrete and concrete encasements to match color and textured finish of exposed existing concrete.

V-groove exposed abutment walls, all fascia concrete including encasements and curtain walls at both fascias, and both sides of the parapets at service street.

V-grooves to match the existing, see Reference Drawing Nos. 39 and 41 in Contract GWB-190.028.

Submit two finished and v-grooved 12-inch by 12-inch by 2-1/2-inch panels with mix design that simulates actual conditions for review and approval.

Submit for review and approval method for achieving desired color, texture, and v-grooving of concrete.

Prepare existing concrete surfaces for grouting and base plate installation as follows:

- ❒ Scarify surface with chipping hammers weighing no more than 15 pounds to the depth indicated on these contract drawings.
- ❒ Use only sharp chisels on chipping hammers.
- ❒ Vigorously wire brush and air blast clean scarified surface immediately prior to grouting of base plates.

- ❒ Load base plate only after grout has achieved full strength in accordance with the grout manufacturer's requirements.

Grout shall be non-shrinking, non-metallic grout in conformance with Contracts Unit Standard Specifications. https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx# (EOL hyperlink is an internal website)

All grout pads shall be formed. Pump grout at support beam top flanges and pour grout at all other locations. Install pads in accordance to the manufacturer's requirements.

Drilled in Anchor Installation Procedure

Prior to drilling holes for any anchor bolts into existing concrete, use rebar locator system to locate existing reinforcement. Where required to clear reinforcement, shift location of hole to the tolerance shown on the contract drawings. Misaligned, out of plumb, and out of level bolts shall not be accepted.

If necessary and approved by the engineer, supplement the use of the rebar locator system with 1/8-inch drilled pilot holes to ensure clearing the existing reinforcement.

Rebar locator system shall be Ferroscan FS 10 as manufactured by Hilti or an approved equal. Submit catalog cut and data on rebar locator system for review and approval.

The rebar locator system shall be capable of accurately locating and recording ferrous objects using a calibrated scanner and monitor that is specifically designed for detecting rebars in concrete. The system shall be capable of recording and saving scoped images and shall accurately locate rebars dimensionally utilizing a built-in grid system. The system shall be capable of monitoring functions and displaying on screen dimensional information.

The rebar locator system shall be capable of accurately locating the rebars indicated below, see Reference Drawing No. 42 in Contract GWB-190.028:

A. Top mat of rebar on south face of south bent wall at building brace beams.

B. Bottom mat of rebar at building underdeck above the support beam.

Use only solid steel drill bits. Hollow drill bits are strictly prohibited.

All holes shall be drilled level and/or plumb. Use jig where required to ensure level and/or plumb holes.

Prior to the installation of beams and braces, the authority shall pull test 5% of the anchors at the underdeck for the support beam and 5% of the anchors at the building wall for the building braces. The pull test load shall be two times the working load of the anchor bolts. The contractor shall coordinate this effort with the authority and provide access and the necessary support for the authority to perform the testing.

Painting

All cleaning and painting shall be in accordance to Contracts Unit Standard Specifications. https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx# (EOL hyperlink is an internal website)

Power tool clean to SSPC SP-3 and field paint exposed steel in bearings including exposed section of bearing anchor bolts with paint system S-1s. Install primer in shop.

Bollards shall be power tool cleaned to SSPC SP-3 and painted with paint system S-1s. Color of top coat shall be yellow.

Reinforcement exposed by cutting and concrete removal in the existing north and south bent walls shall be power tool cleaned to SSPC SP-3 and painted with 2 coats of the primer (epoxy mastic) for paint system S-3s. Extend paint 1 inch onto adjacent concrete surfaces.

Apply all paint coats to the dry film thickness (dft) recommended by the manufacturer.

Expansion Joints and Silicone Sealer

Preformed joint seal system at abutment slab expansion joint shall be Evazote 380® E.S.P. as manufactured by Epoxy Engineered Materials, LLC or an approved equal. Install in accordance with manufacturer's requirements.

Expansion joint shall be installed by a licensed installer. Submit copy of licensing contract for review and approval.

Submit shop drawings and product catalog cuts of the expansion joint to be used, for review and approval by the authority. Shop drawings shall include complete details and the installation method. Submit one sample, of 6" length for the type of joint to be used.

Armoring (including angles, end welded studs, and square bars) for the expansion joint at the access ramp shall be hot dip galvanized after fabrication in accordance with ASTM A123, G230 (2.3 ounces of zinc galvanizing per square foot).

Headed end welded studs for joint armoring shall be type H4L headed concrete anchors (HCA) as manufactured by Nelson Stud Welding or an approved equal.

Asphalt joint filler at abutment slab expansion joints shall be hot applied, rubberized asphalt sealant as manufactured by W.R. Meadow or an approved equal, meeting federal specification SS-S-Q1401 C. Asphalt joint filler shall also be in accordance with Contracts Unit Standard Specifications. https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (128) (EOL hyperlink is an internal website)

Install silicone sealer in accordance to the manufacturer's requirements and Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx# (EOL hyperlink is an internal website). Sealant for caulking shall be ES-2 silicone sealant. Joint sealant backing shall be elastomeric tubing.

Preformed joint filler shall be Flexcell expansion joint filler as manufactured by Knight-Celotex Fireboard or an approved equal.

Permanent Netting

Permanent netting shall be heavy duty nylon debris netting with a ¼-inch mesh.

Netting shall be attached to existing concrete and concrete encasement with stainless steel anchors spaced no more than 3 feet on centers.

When installing anchors, use rebar locator system to ensure clearing of existing reinforcement.

Submit design of protective netting along with calculations and catalog cuts for review and approval. Drawings and calculations for protective netting shall be signed and sealed by a New Jersey Professional Engineer.

Timber Cribbing for Brace Beams and Blocking for Needle Beams

All timber shall be Douglas Fir-Larch No. 1 or No. 2.

All timber shall be fire retardant and shall conform to the building laws of the City of New York.

Timber shall be sound, square-edged and free from shakes, loose knots, and decay.

Jacking for GWB Tolls Building Underpinning

Contractor shall submit jacking procedure and method for monitoring load and movement of the needle beams for review to the authority 90 days before commencing with any jacking. Jacking and monitoring procedure shall be signed and sealed by a Professional Engineer licensed in the State of New Jersey.

Contractor shall assign a Professional Engineer licensed in the State of New Jersey to supervise the jacking and monitoring operation. The contractor’s engineer shall have a minimum of 3 years of experience with jacking of buildings. Submit documentation on Professional Engineer for authority review.

Instrumentation to monitor up or down movement shall be provided at the bottom of all needle beams directly below the support beam. Instrumentation shall be capable of measuring movements of .01 inches or less. Monitoring shall be continuous during all jacking and bent wall removal.

Jacks shall be clp-4002, pancake lock nut cylinders as manufactured by Enerpac Hydraulic Technology or an approved equal. Jacks shall have a minimum capacity of 400 tons with a maximum collapsed height of 8 inches, a minimum stroke of 1.75 inches, a maximum outside diameter of 17 inches. Lock rings shall be capable of adjustment to a tolerance of 1/64 inch. Manufacturer shall demonstrate method for achieving and tracking 1/64 inch adjustment tolerances and shall ensure proper pitch on lock ring to achieve desired tolerances.

Jacking shall be conducted with a pc-controlled synchronous lift system as manufactured by Enerpac Hydraulic Technology or an approved equal. The synchronous system shall be installed with a 6-point layout. Contiguous abutment jacks shall be paired to a single point. The system shall be capable of jacking to a tolerance of 1,000 pounds and lifting to a tolerance of .01 inch. The synchronous system shall have data acquisition features to record and monitor force, displacement, and pressure at all jacks at any time. Synchronous system shall be capable of tracking loads to the nearest 1,000 pounds and displacements to the nearest .04 inch. The synchronous system shall include automatic stop at pre-set stroke or load limit and shall have multiple lift point capabilities.

Install 6 stroke sensors with one sensor located at each abutment jacks. PC-controlled synchronous lift system shall be capable of tracking displacements to the nearest .04 inch at all sensors simultaneously.

Jacking equipment, load monitoring devices, and instrumentation for measuring movement shall be suitable for the purpose intended, and shall be submitted for review by the authority. Each jack shall have the rated capacity clearly shown on the manufacturer's name plate attached to each jack. Jacks shall be equipped with pressure gages or other load measuring devices that will enable the applied lifting force to be monitored at all times.

Catalog cuts for the synchronous lift system, jacking equipment, load monitoring devices, and instrumentation for monitoring movement shall be submitted for review by the authority.

The contractor shall coordinate all jacking operations with the authority. The contractor shall notify the authority 14 days before commencing with any jacking operations.

The contractor shall report in writing all field measurements from the monitoring of the load on the needle beam, and the movements of the needle beam at the support beam and abutment to the authority for review.

At least 7 days prior to scheduled jacking operations, the contractor shall perform a “dry run” in the presence of the authority to verify that all jacking equipment, pumps, load monitoring devices, movement monitoring devices, and appurtenances are functioning properly.

4.2.5 CLEANING AND PAINTING

4.2.5.1 CLEANING AND PAINTING NOTES

The existing paint contains lead, chromium, or other heavy metals. Conform to Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx# (EOL hyperlink is an internal website), for containment, paint debris collection and disposal, worker protection, and environmental requirements.

Cleaning and painting shall conform to Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx# (EOL hyperlink is an internal website), and Items A and B below.

A. Cleaning and Painting Procedure

1. Installed Steel: Shop clean all surfaces to SSPC SP10 (Near-White Metal Blast Cleaning). Shop coat with primer of paint system S-1S (see note 4).
2. Existing Steel: At contact surfaces and surfaces within a 2-inch radius of replacement fasteners, clean to SSPC SP11 (Power Tool Cleaning to Bare Metal) using HEPA vacuum attachments to a 1- to 2-mil profile. Extend cleaning to 2 inches beyond limit of contact surfaces. Coat with primer of paint system S-1S (see note 4). Minimize overlap onto existing paint. If existing steel and existing fasteners cannot be painted within 8 hours or before rerusting occurs, reclean surfaces to SSPC SP11 immediately prior to painting steel and fasteners.
3. Installed mechanically galvanized A325 fasteners, installed steel, and surrounding existing steel—Hand wire brush exposed portion of galvanized fasteners and field paint fasteners and installed steel with second and third coat of paint system S-1S. Minimize overlap onto existing paint.
4. The prime coat shall provide a Class A or B slip-critical surface with a minimum slip coefficient of 0.33.

B. Paint Application

1. Apply primer and subsequent paint coats to the dry film thickness (DFT) recommended by the manufacturer.
2. Do not stop or resume painting at edges of inside or outside corners. Always extend paint a minimum of 1-1/2 inches beyond corners before stopping and when resuming painting.
3. Paint coat colors shall be distinctly different from one another. Primer and second coat color shall be different than the existing paint color. Color of the finish coat of paint shall match existing paint. Submit a 4 inch by 8 inch color sample of finish coat to the engineer for approval, prior to ordering material(s).

Painted surfaces of existing and installed steel damaged during construction shall be repaired in accordance with Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website).

4.2.5.2 PAINTING AND TEMPORARY PLATFORM NOTES

Typical Structural Painting Notes

A. All existing paint contains lead.

B. The contractor shall complete repairs shown on Drawing Nos. 250 to 292 in each span prior to cleaning and painting the span in accordance with Drawing Nos. 202 to 222, and 3 to 9.

C. Sizes, quantities, details, and locations of all items identified on the contract drawings are given only as an indication of the work involved in this contract. The exact sizes, quantities, details, and locations shall be determined by the contractor

D. The dimensions shown on the contract drawings for each structure are derived from drawings of the original construction and from drawings of subsequent rehabilitation/repairs. They may vary from the actual conditions as exist in the field. The contractor shall verify by field measurements all dimensions that may be required to locate, align, dimension, and detail any construction required to perform the work. Field measurements shall be performed prior to the preparation of shop drawings and the manufacturing, fabrication, and installation of all items of construction. All variations from the contract drawings shall be brought to the attention of the engineer prior to the preparation of shop drawings.

E. The contractor shall note that due to the complexity of the structures and the extent of repairs over the years, not all details and items that require cleaning and painting are shown. The contractor is responsible for cleaning and painting all surfaces within the limits of the work shown in this contract on Drawing Nos. 202 through 222, regardless of completeness of details shown on these contract drawings.

F. The full extent of conduits and utilities on the existing structures is not shown on the contract drawings. The contractor shall clean and repaint all painted conduits, utilities, and the firestand pipe on the sidewalk.

G. Furnish and install temporary supports where required and where directed by the engineer, to support elements that are disconnected from their existing supports to make repairs or clean and paint the supporting members. Elements shall include, but are not limited to drainage scuppers and catch basins, drainage troughs, conduits, utilities, cables, junction boxes, grating, wiring, lighting, and signs. Reinstall all elements after repair or painting is complete and dry. Compensation for this work shall be at the net cost thereof. For computation of net cost, see Note No. 17.

H. All existing structural shapes, existing plates, and other existing elements are carbon steel or silicon steel, unless otherwise noted.

I. All heavy equipment and items such as dust collectors, abrasive blasting equipment, compressors, piping, and conduits shall be secured properly to account for vibration and movement of the existing structure.

J. The existing metal catwalks and the metal catwalk constructed under span 28w in this contract (see Drawing Nos. 250 to 292) shall not be used by the contractor to transport and or store construction loads of any kind, or to support temporary structures (such as containments, platforms, scaffolding, bracing, etc.). The contractor shall protect the catwalk during all repair operations, and abrasive cleaning and painting operations.

K. As directed by the engineer, the contractor shall repair existing support connections of the catwalks. Compensation for this work shall be at the net cost thereof. For computation of net cost, see Note No. 17.

L. Remove existing platforms that may be in place during this contract (see Drawing Nos. 204 and 205) as required to perform work and reinstall upon completion of work in the panel. Compensation for this work shall be at the net cost thereof. For computation of net cost, see Note No. 17.

M. Prior to the start of the steel and concrete repair work shown on Drawing Nos. 250 to 292, the contractor shall submit for review detailed schedule, construction sequence, and erection procedures for all repairs. The contractor shall coordinate all repairs, especially where multiple repairs to the same member occur simultaneously, and detail the repairs so that they do not interfere with one another and have sufficient clearances.

N. Detailing, fabrication, and execution of construction shall conform to the requirements of the specifications and of current issues of the following publications:

1. Standard Specifications for Highway Bridges adopted by the American Association of State Highway and Transportation Officials (AASHTO), including interim specifications.
2. The American Welding Society Bridge Welding Code AWS D1.5.

O. The bridge will be open to vehicular traffic during the work of this contract, except as noted for bearing replacement on Drawing No. 290. For hours of work and lane availability, see "Conditions and Precautions" in the contract specifications.

P. For legend and abbreviations, see Drawing No. 2.

Q. Net cost shall be computed in the same manner as is compensation for extra work, including any percentage addition to cost, as set forth in the clause of the contract providing compensation for extra work. Performance of such net cost work shall be subject to all provisions of the contract relating to performance of extra work. Compensation for said net cost work shall not be charged against the total amount of compensation authorized for extra work.

Temporary Structures (Containments, Platforms, Scaffolding, and Other Structures)

A. For cleaning and painting shown on Drawing Nos. 202 to 222, contractor shall use enclosure (containment) types I to III as specified on Drawing Nos. 3 to 9, 221 & 222.

B. For enclosure (containment) system requirements for cleaning and painting shown on Drawing Nos. 202 to 222, see these structural notes (Drawing Nos. 200 & 201) and Drawing Nos. 3 to 9, 221 & 222, and Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website).

C. The contractor shall provide protective shielding for all temporary structures above public places, roadways, and railroad tracks to protect pedestrians, vehicular traffic, rail traffic, and waterway traffic. The contractor may use platforms as protective shielding. Maintain vertical clearances for all roadway, waterway, and railroad traffic as shown on the contract drawings.

D. The contractor shall submit for review structural design calculations, design drawings, layout drawings, erection drawings, procedure for moving platforms, and shop details for the containments and all other temporary structures, including but not limited to protective shielding, platforms, scaffolding, bracing, and jacking/load transfer details. Shop drawings shall include the maximum allowable loading at the load cells, strain gages, or dynamometers. Shop drawings shall also include the maximum allowable depth of blast medium on the platforms. If more than one engineer of record is utilized, all work shall be fully coordinated with each engineer of record. Submittal of shop drawings and calculations shall be complete and shall be done at the same time.

E. Verify the structural integrity of the existing structures for imposed construction loads for each typical enclosure layout (including all associated temporary structures) at each varying location (see Drawing Nos. 3 to 9, 221 & 222) in accordance with the specifications and contract drawings.

F. All calculations, drawings, and shop drawings shall be signed and sealed by Professional Engineer(s) licensed in the States of New Jersey and New York. All drawings, details, and procedures shall include the following note, “for performing work shown on Drawing No(s). [add applicable drawing number(s).] and/or specification section no(s). [add applicable section no(s).].” All drawings, details, and procedures shall also include the following:

1. Contract title, contract no., date, and name of contractor and/or subcontractor.
2. Drawing or page number, as applicable, and signatures.

G. Detailing, fabrication, and installation of the containment and all temporary structures, including but not limited to platforms, shielding, scaffolding, and bracing, shall conform to the requirements of the specifications and to the more stringent sections of the current issues of the following publications, unless otherwise noted in the “construction loading criteria” given herein:

1. Standard specifications for highway bridges adopted by the American Association of State Highway and Transportation Officials (AASHTO), including interim specifications.
2. American Society of Civil Engineers, minimum design loads for buildings and other structures (ASCE 7-95).
3. Occupational Safety and Health Administration (OSHA), Safety and Health Standards - 29 CRF 1926.
4. The American Welding Society Bridge Welding Code AWS D1.5.

H. Submit evidence that the engineer of record responsible for the engineering of the containment and temporary structures has previously engineered at least three installations of the same type.

I. For all temporary structures and temporary construction, specifically not defined in the current edition of the OSHA standards, the contractor shall submit OSHA documentation and interpretation that indicates acceptance by OSHA.

J. The engineer of record shall inspect and supervise the construction of the containment and other temporary structures to ensure conformance to his design. Prior to the commencement of the specified repair work or surface preparation work at each temporary structure, the contractor’s engineer of record shall inspect and verify that the construction of the temporary structure conforms to his design drawings, approved shop drawings, and the calculations. The contractor’s engineer of record shall submit his certification to the engineer. Certification shall indicate that the work is safe and is in accordance with approved documents. The contractor shall not use the temporary structures in any way until such certificate is submitted to and reviewed by the engineer.

K. Actual failure or close to failure condition of any component of the temporary structure, including containment, shall be immediately reported to the engineer by the contractor, and repairs shall be made as directed by the engineer of record to the satisfaction of the engineer. In addition, the contractor shall submit a written report, prepared by the engineer of record, on reasons, actions taken, and the extent of damage and repairs performed, within 2 weeks of any such incident.

L. Movement of the containment and work platform shall be made only when traffic lanes below enclosures and/or work platforms are closed to traffic, as specified for this contract.

M. Contractor shall ensure that the enclosure (containment) system’s working/walking surface is safe for the intended work and is sound and stable. If working/walking surface does not meet this criterion, the contractor shall supplement the enclosure with independent work platforms.

N. Enclosure (containment) tarps shall be fire retardant, and resistant to ripping and tearing.

O. Construction of containments and temporary structures shall be coordinated with traffic lane closures.

P. The contractor shall at all times control the accumulation of debris and blast medium, by continuously removing it, to ensure that the total construction load on the temporary work platforms, containments, and protective shielding does not exceed the specified load limits in the “construction loading criteria” given herein and the maximum depth of accumulated blast medium specified in the shop drawings.

Q. All connections to the existing structures shall be made by clamping with retaining devices to prevent any slippage due to bridge vibration. Field welding and/or drilling for bolting to the existing steel shall not be permitted.

R. The use of welding or burning equipment will not be permitted in the containment at any time without approval from the engineer.

S. Drilling into or attaching to the existing concrete shall not be permitted, unless approved by the engineer.

Construction Loading Criteria

A. Construction load is defined as any temporary load imposed by the contractor on the existing bridge structure and its component members for the performance of the contract and includes, but is not limited to, construction live load as described below, temporary supports, bracing, personnel, equipment, materials, containments, work platforms, protective shielding, and accumulated debris. Construction load shall also include wind loading, any snow, ice, and precipitation that accumulate on the enclosure (containment) and temporary structures. The contractor shall take necessary measures to eliminate ponding of water or accumulation of snow or ice on the enclosure (containment) and work platforms. Work platforms, shielding, and other items that may accumulate water, snow, or ice with the containment lowered, removed, or at any stage of development shall be designed for such loading. The contractor shall remove snow and ice from temporary structures during the duration of the contract, to prevent the roadway or areas underneath the temporary structures from developing slippery or unsafe conditions from melting or falling snow and ice.

B. Forces from the expansion and contraction of the existing structures shall not be transferred to the temporary structures in any way. Bridging over existing expansion joints shall not be allowed.

C. Temporary work platforms, containments, scaffolding, protective shielding, and other temporary structures shall be braced to prevent swaying and shall account for forces from wind, bridge movements, and bridge vibrations. Appropriate impact loads due to wind generated vibrations and wind generated sudden movements of the temporary structures shall be included in the designs and details.

D. For design purposes, the loading criteria for the individual components of the containments, work platforms, protective shielding, and the existing structure shall be as follows:

1. Decking material (platforms and/or shielding): Capable of supporting 150 psf construction live load and sustaining a dynamic load caused by a single 100 pound weight (block of concrete) falling 6 feet and shall be tested for said loading in the presence of the engineer prior to its installation.
2. Framing, hanger supports, and catenary cables of decking and their connections to the structural components of the existing steel framing: Capable of supporting their tributary load from decking, based on 70 psf construction live load.

3. For hung platforms of containments used for Drawing Nos. 202 to 222, contractor shall furnish and install load cells, strain gages, or dynamometers on the vertical hangers supporting the suspended platform assemblies and shall monitor loading on the members to ensure that the total construction load does not exceed the specified load limits. Contractor shall add load cells or strain gages to every 3rd hanger, except where platform length is less than 20 feet, contractor shall add load cells or strain gages to every 2nd hanger. In case of specialized construction of the platforms and where deemed necessary by the engineer, load cells or strain gages shall be spaced closer than stipulated above.
4. For catenary (cable supported) platforms for containments used for Drawing Nos. 202 to 222, contractor shall install load cells, strain gages, or other approved equal device(s) on all horizontal catenary cables to monitor loading to ensure that the total construction load does not exceed the specified load limits. Install load cells or strain gages on vertical hangers/cables of catenary support system in accordance with item 3.4c above. Install deflectometers or other approved equal device(s) for monitoring deflections to ensure that underclearances are not less than the specified underclearances. catenary support system shall be designed so that deflections of the catenary platform due to dynamic loading and ultimate loading as specified in Items 4A and 4E or during erection or rigging shall not reduce underclearances below the specified underclearances.
5. Contractor shall monitor depth of accumulated blast medium on the platforms to ensure the depth limits indicated on the shop drawings are not exceeded.
6. Platforms and its components shall also be designed to support an ultimate load of at least 4 times the maximum load without failure and all wire cables shall be designed for ultimate loading in accordance with OSHA. Design shall account for wind in any direction, including up and down direction, and shall also include wind gusts from trucks passing underneath. Platform deflections due to any load combination shall be limited to ensure that vertical clearances are not less than required. All platforms shall be designed as redundant systems with multiple load paths, so that failure of any member does not result in stresses higher than yield stresses in any other component of the platform.
7. Existing structural components of the steel framing: Components that are used by the contractor to support any temporary structures shall be restricted to stringers and floor beams. It is the contractor's responsibility to identify the need for designing, furnishing, and installing temporary supports or bracing required to reinforce the existing structure (or structural components), for imposed construction loads that produce a total bending and shear stress higher than the allowable stresses. In lieu of an analysis combining all loadings to determine the total bending and shear stresses on the existing steel, the contractor may use the following load criteria: The additional stresses due to the construction load on the existing structural component shall be limited to 3,000 psi. However, the local web and flange buckling stresses on the existing structure due to the ultimate loading shall not exceed the allowable stress as determined per the "allowable stress design" method. Existing members shall also be analyzed for local forces due to catenary cables and if required existing members shall be strengthened or transfer mechanism shall be designed to reduce or eliminate loading on existing member. The calculations shall include verification of stresses on the existing structures due to all construction loads including jacking and load transfer forces.
8. Enclosures (containments) for Drawing Nos. 202 to 222: In accordance with the specifications, the containment system roof and wall elements shall be removed or lowered when sustained wind or gusts in excess of 40 mph are anticipated. However, the containment's supporting elements in the fully installed condition, as well as the lowered or removed condition, shall be designed in accordance with ASCE 7-95. All elements exposed after removal or lowering of the containment, such as shielding and

work platforms shall be properly secured and shall be designed for the full wind loading per ASCE 7-95.

9. No reduction of loading will be allowed for load combinations.

E. The contractor shall not use the north sidewalk, except as noted in the specifications.

F. Due to the poor condition and low remaining carrying capacity of the existing south sidewalk (including supporting steel and concrete), it is not recommended that the contractor utilize the sidewalk for storing items, supporting equipment, and suspended platforms. However, if the contractor opts to utilize the south sidewalk, the contractor shall field inspect all sidewalk bays to be utilized, to check for section losses to the existing steel and deterioration to the existing concrete slab. The contractor shall submit documentation of inspection, and where losses and/or deterioration are present, the contractor shall submit calculations for the carrying capacity of the existing sidewalk for the proposed loading. In any case, the maximum allowable load on the existing south sidewalk shall be limited to 50 psf.

Paint Removal and Repainting Notes for Drawing Nos. 3 to 9 and 202 to 222

A. All existing paint contains lead. For cleaning and painting indicated in these notes and shown on Drawing Nos. 202 to 222 and Drawing Nos. 250 to 292, refer to Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website),

B. Remove paint from, and repaint existing steel and other elements at the following structures as indicated in the notes and shown on Drawing Nos. 202 to 222, unless otherwise noted:

- The New York Viaduct, the New Jersey Viaduct, and the Eastbound Approach Ramp, Ramps 6 and 7.

The existing structures shall include, but are not limited to the following:

- Floorbeams, stringers, girders, diagonal bracing, all miscellaneous metal appurtenances supported by the structures, connection plates, bearing plates, stiffener plates, framing connections, rivets, bolts, bearing assemblies, elements of drainage scuppers and catch basins, utilities, conduits, firestand pipe, steel curbs, expansion joints, drainage troughs, embedded steel plates, and angles. Removal of paint and repainting shall also include the steel repairs (with interim 2 coat paint system) in this contract shown on Drawing Nos. 250 to 292. The interim 2 coat paint system applied to the steel repairs shall be removed and steel repairs shall be repainted.

C. As directed by the engineer, the contractor shall perform rehabilitation and/or replacement of the existing deck joints and their supporting structural steel. Compensation for such rehabilitation/replacement work shall be at the net cost thereof. For computation of net cost, see Note 17.

D. The contractor shall clean debris from each area of the structure before cleaning or recoating.

E. Steel shall be cleaned by abrasive blasting to SSPC-SP10 (near-white blast cleaning) and shall be repainted with paint system S-1, except as noted below and as shown on the contract drawings. Galvanized steel, stainless steel, and aluminum shall be cleaned and painted (where applicable) as noted below. All cleaning and painting shall be in accordance with Contracts Unit Standard Specifications, https://panynj.sharepoint.com

/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website). Also on all horizontal surfaces, prior to the application of the primer, all pits 3/16 inch or deeper shall be filled with an epoxy filler material in accordance with the specifications where directed by the engineer.

1. Use of SSPC-SP11 surface preparation, power tool cleaning to bare metal using tools equipped with HEPA-vacuum attachments, shall be limited to small areas and to where it is not practical to erect a temporary containment for SSPC-SP10 (near-white blast cleaning). Surfaces to receive SSPC-SP11 surface preparation shall be approved by the engineer.
2. Bearing surfaces and moving mechanisms of bearing assemblies (see Drawing Nos. 216 & 217):
    a. Contractor shall identify limits of bearing assemblies (bearing surfaces and moving mechanisms) that require special cleaning (indicated below) to the satisfaction of the engineer. Contractor shall proceed with cleaning only after approval by the engineer.
    b. Clean to SSPC SP12 (high pressure water jetting) without abrasives to remove all paint, corrosion, and foreign matter. The contractor shall not mix water blasting and abrasive blasting operations. Use high pressures of 10,000 psi to 25,000 psi and vary nozzles as required for thorough cleaning to condition WJ-1 of SSPC SP12.
    c. Allow to dry.
    d. Spray penetrating sealer1 into spaces until they are thoroughly wetted. Do not paint.
    e. Clean surrounding steel that is contaminated with penetrating sealer to SSPC-SP10 (near-white blast cleaning) prior to any painting.
    f. Protect bearing surfaces and moving mechanisms from contamination and clogging at all times.
3. Steel surfaces at edges of steel members in contact with concrete (see Drawing Nos. 217 & 220):
    a. Clean in accordance with SSPC-SP10 (near-white blast cleaning).
    b. Apply zinc-rich primer of paint system s-1.
    c. Stripe steel/concrete interface with the penetrating sealer of paint system S-2. Overlap concrete a minimum of one inch.
    d. Stripe edges of steel with epoxy (2nd coat of paint system S-1) by hand brushing or equivalent.
    e. Paint using the 2nd and 3rd coats of paint system S-1.
4. Expansion joints, rocker joints, and fixed joints:

    Note that joint openings are difficult cleaning and painting areas. Joint openings are as small as 1/2 inch and are approximately 7 to 10 inches deep and may require the use of specialized equipment for cleaning and painting.

[1] *Penetrating sealer shall be Devoe® Pre-Prime 167 as manufactured by Devoe Coatings Company of Rahway, New Jersey or an approved equal. Application shall be in accordance with the manufacturer's requirements and recommendations. Apply sealer until the surface is thoroughly wetted.

a. Cover joint assemblies with rigid material to protect the traffic above, and to protect joint filler medium such as preformed compression seals and strip seals. Protective covering shall provide adequate protection from abrasive blast cleaning and painting operations. Contractor shall submit details for protecting expansion joints, and shall not proceed with cleaning and painting described above, until installation of protective covering is approved by the engineer. Any damage to the joint filler medium shall be repaired, or replaced by the contractor to the satisfaction of the engineer, at no additional cost to the authority.

b. Clean and paint (paint system S-2) difficult to access vertical surfaces in joint openings in accordance with the requirements for "limited access area - condition 1" in Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website), as directed by the engineer. Clean remaining sections of expansion joint in accordance with SSPC-SP10 (near-white blast cleaning) and paint with paint system s-1.

5. Cleaning and painting of girders, floor beams, stringers, and diaphragms at joints, see Drawing No. 220 for limits:

    a. Clean in accordance with SSPC-SP10 (near-white blast cleaning).

    b. Note that this area of the bridge has a heavy concentration of pitting. All horizontal surfaces with pits 3/16 inch or deeper shall be filled with an epoxy filler material in accordance with the specifications.

    c. Paint with primer of paint system S-1.

    d. Paint with 2 coats of 2nd coat of paint system S-1.

6. Areas of impacted rust (see Drawing No. 219):

    a. Clean in accordance with SSPC-SP10 (near-white blast cleaning).

    b. Remove rust using grinding tools, heavy duty vibrating, impact, and power tools as approved by the engineer. Chipping hammers shall weigh no more than 15 pounds.

    c. Remove loose materials by blowing out with clean and dry compressed air.

    d. Apply zinc rich primer of paint system S-1.

    e. Stripe impacted rust with epoxy (2nd coat of paint system S-1) by hand brushing.

    f. Paint using the 2nd and 3rd coats of paint system S-1.

    g. Apply sealant (see Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx, EOL hyperlink is an internal website) in accordance with the manufacturer's recommendation. Extend sealant 3 inches in both directions beyond impacted rust.

7. Through girder G16 at top of concrete sidewalk (see Drawing Nos. 204 & 276):

    a. Clean in accordance with SSPC-SP10 (near-white blast cleaning).

    b. Apply zinc-rich primer of paint system S-1.

    c. Stripe steel concrete interface with the penetrating sealer of paint system S-2. Overlap concrete a minimum of one inch.

d. Stripe steel at interface with top of concrete sidewalk with epoxy (2nd coat of paint system S-1) by hand brushing or equivalent. Striping to extend around the perimeter of the through girder.

e. Paint using the 2nd and 3rd coats of paint system S-1.

f. After painting, apply sealant atop concrete sidewalk along protruding through girder. Sealant to extend around the perimeter of the through girder.

8. Conduits and utilities:

   a. Painted galvanized steel, painted stainless steel, and painted aluminum:

      1) Clean to SSPC SP-7 (brush-off blast cleaning).

      2) Paint with paint system S-4.

      3) See special provisions for cleaning galvanized steel and aluminum in Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx, (EOL hyperlink is an internal website).

   b. Steel painted and unpainted: Clean in accordance with SSPC SP-10 (near-white blast cleaning) and paint with paint system S-1.

   c. Protect unpainted conduits (galvanized steel, stainless steel, and aluminum) during all cleaning and painting operations.

   d. The existing structures contain extensive PVC-coated conduits that were only recently installed. Protect all PVC-coated conduits during all cleaning and painting operations.

9. Aluminum drainage troughs (painted and unpainted) and painted aluminum drainage trough support beams: clean to SSPC SP-7 (brush-off blast cleaning) and paint with paint system S-4. See special provisions for cleaning aluminum in Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website). Do not clean and paint unpainted aluminum trough support beams, and protect during all cleaning and painting operations.

10. Painted elements of the scuppers and catch basins: aluminum, galvanized steel, and stainless steel elements shall be cleaned to SSPC SP-7 (brush-off blast cleaning) and painted with paint system S4. See special provisions for cleaning galvanized steel and aluminum in Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website). Protect all unpainted elements during cleaning and painting.

11. Other miscellaneous items shall be cleaned and painted as directed by the engineer.

F. Where the paint manufacturer requires a surface profile range of the cleaned steel more restrictive than specified in the specifications, the contractor shall prepare surfaces to the profile range required by the paint manufacturer at no additional cost to the authority. All unique requirements stipulated by the paint manufacturer for the more restrictive profile range such as mist coating prior to paint application shall be at no additional cost to the authority.

G. The cleaning and painting of steel in the space between floorbeams at joints may be done from the inside (worker bodily enters this space) at the contractor's discretion and risk. Cleaning and painting from the inside shall be done in accordance with the confined space requirements of the Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx, (EOL hyperlink is an internal website).

H. Do not clean or paint “pin and hanger redundant support system” shown on Drawing No. 218. Protect support system at all times during cleaning and painting, and all other work in this contract.

I. Protect all concrete surfaces and other surfaces not requiring cleaning during blast cleaning.

J. Overlap all cleaning and each coat of painting at breaklines a minimum of 3 inches as shown on Drawing No. 219. Only overlap compatible coats of paint in accordance with the manufacturer’s recommendations.

K. Extend cleaning and painting a minimum of 1-1/2" (one and one half inches) past all edges and corners as shown on Drawing No. 219. Do not stop cleaning or painting at edges of any inside or outside corners, always extend operation 1-1/2” before stopping.

L. Paint each small item in its entirety without any breaklines of primer or paint coats.

M. For information on the condition of the existing coatings and underlying steel substrate on the structure, refer to the reference document (KTA-Tator report) in the specifications. The document provides information on the condition of the existing coatings and underlying steel substrate in a few select locations on the structure at the time of testing. The data should not be relied upon as being representative of the condition of the coatings across the entire structure. The contractor is responsible to perform work as shown and specified, regardless of variations in the conditions of existing paint, number of coats, paint thickness, coating adhesion, and condition of underlying substrate.

4.2.5.3 STRUCTURAL, PAINTING, BEARINGS, & JACKING NOTES (138)

4.2.5.4 PAINTING IN WET CONDITIONS

Where painting or repainting is required in damp and wet conditions, strongly consider using a paint system that can be applied and cured in wet conditions. The Materials Engineering Division recommends the use of Alocit 28.15 as manufactured by Warfield Company or Permox 9043 as manufactured by Permite Corporation, or approved equal, for painting or repainting of steel in wet, damp conditions particularly in underground applications and where there is high humidity. Work with the Materials Engineering Division as appropriate in specifying materials in contract documents.

4.2.6 OSHA REQUIRED

4.2.6.1 THE OSHA STEEL ERECTION STANDARD - SUMMARY

The OSHA Steel Erection Standard is provided in OSHA Instruction, Directive Number: CPL 1-1.34, dated March 22, 2002. The requirements in the Instruction that apply to design and detailing of steel for structural engineers is summarized in the November 2002 issue of the *“Structure”* engineering periodical.

4.2.6.2 OSHA INSTRUCTION DIRECTIVE CLP 2-1.34 (STEEL ERECTION) (139)

4.2.6.3 OSHA SUBPART R STEEL ERECTION (140)

4.2.6.4 NEW OSHA ERECTION RULES, HOW THEY AFFECT ENGINEERS (141)

4.2.7 NEW JERSEY SPECIAL INSPECTIONS (142)

4.2.8 CEILINGS

4.2.8.1 RESTRICTIONS ON ACCESSING CEILINGS (143)

4.2.8.2 CEILINGS - HEAVYWEIGHT & LIGHTWEIGHT CRITERIA (144)

4.2.9 DRILLING, CORING, AND JACK HAMMERING

4.2.9.1 DRILLING HOLES FOR ANCHOR BOLTS

Prior to drilling holes for any anchor bolts into existing concrete, use rebar locator system to locate existing reinforcement. Where required to clear reinforcement, shift location of hole to the tolerance shown on the contract drawings. Misaligned, out of plumb, and out of level bolts will not be accepted.

If necessary and approved by the engineer, supplement the use of the rebar locator system with 1/8” drilled pilot holes to ensure clearing the existing reinforcement.

Rebar locator system shall be Ferroscan FS10 as manufactured by Hilti or an approved equal. Submit catalog cut and data on rebar locator system for review and approval.

The rebar locator system shall be capable of accurately locating and recording ferrous objects using a calibrated scanner and monitor that is specifically designed for detecting rebars in concrete. The system shall be capable of recording and saving scoped images and shall accurately locate rebars dimensionally utilizing a built-in grid system. The system shall be capable of monitoring functions and displaying on screen dimensional information.

The rebar locator system shall be capable of accurately locating the rebars indicated below, see Reference Drawing No. 42 in Contract GWB-190.028:

- Top mat of rebar on south face of south bent wall at building brace beam.
- Bottom mat of rebar at building underdeck above the support beam.

Use only solid steel drill bits; hollow drill bits are strictly prohibited.

All holes shall be drilled level and/or plumb; use jig where required to ensure level and/or plumb holes.

Prior to the installation of ceiling anchor bolts, the authority shall pull test 5% of all anchor bolts to be installed or a minimum of 5 anchor bolts, whichever is greater. The pull test load shall be 2 (two) times the working load of the anchor bolt. The contractor shall coordinate this effort with the Authority and provide access and the necessary support for the Authority to perform the testing.

The contractor shall provide necessary equipment and/or scaffolding to allow access to the installed anchor bolts in order for the PA to perform the stated testing.

4.2.9.2 JACK HAMMERING RESTRICTIONS

Prepare existing concrete surfaces for grouting and base plate installation as follows:

- ❒ Scarify surface with chipping hammers weighing no more than 15 pounds to the depth indicated on these contract drawings.
- ❒ Use only sharp chisels on chipping hammers.
- ❒ The impact angle between the hammer and surface of concrete shall not exceed 45°.
- ❒ Vigorously wire brush and air blast clean scarified surface immediately prior to grouting of base plates.
- ❒ Load base plate only after grout has achieved full strength in accordance with the grout manufacturer's requirements.

4.2.10 JACKING

4.2.10.1 JACKING OF BUILDINGS (SAMPLE NOTES)

Contractor shall submit jacking procedure and method for monitoring load and movement of the needle beams for review to the authority 90 days before commencing with any jacking. Jacking and monitoring procedure shall be signed and sealed by a Professional Engineer licensed in the State of New Jersey.

Contractor shall assign a Professional Engineer licensed in the State of New Jersey to supervise the jacking and monitoring operation. The contractor's engineer shall have a minimum of 3 years of experience with jacking of buildings. Submit documentation on engineer for Authority review.

Instrumentation to monitor up or down movement shall be provided at the bottom of all needle beams directly below the support beam. Instrumentation shall be capable of measuring movements of .01 inches or less. Monitoring shall be continuous during all jacking and bent wall removal.

Jacks shall be CLP-4002, pancake lock nut cylinders as manufactured by Enerpac Hydraulic Technology or an approved equal. Jacks shall have a minimum capacity of 400 tons with a maximum collapsed height of 8 inches, a minimum stroke of 1.75 inches, a maximum outside diameter of 17 inches. Lock rings shall be capable of adjustment to a tolerance of 1/64 inch. Manufacturer shall demonstrate method for achieving and tracking 1/64 inch adjustment tolerances and shall ensure proper pitch on lock ring to achieve desired tolerances.

Jacking shall be conducted with a PC-controlled synchronous lift system as manufactured by Enerpac Hydraulic Technology or an approved equal. The synchronous system shall be installed with a 6-point layout. Contiguous abutment jacks shall be paired to a single point. The system shall be capable of jacking to a tolerance of 1,000 pounds and lifting to a tolerance of .01 inch. The synchronous system shall have data acquisition features to record and monitor force, displacement, and pressure at all jacks at any time. Synchronous system shall be capable of tracking loads to the nearest 1,000 pounds and displacements to the nearest .04 inch. The synchronous system shall include automatic stop at pre-set stroke or load limit and shall have multiple lift point capabilities.

Install 6 stroke sensors with one sensor located at each abutment jacks. PC-controlled synchronous lift system shall be capable of tracking displacements to the nearest .04 inch at all sensors simultaneously.

Jacking equipment, load monitoring devices, and instrumentation for measuring movement shall be suitable for the purpose intended, and shall be submitted for review by the Authority. Each jack shall have the rated capacity clearly shown on the manufacturer's name plate attached to each jack. Jacks shall be equipped with pressure gages or other load measuring devices that will enable the applied lifting force to be monitored at all times.

Catalog cuts for the synchronous lift system and pressure gauges shall be submitted for review by the Authority.

The contractor shall coordinate all jacking operations with the Authority. The contractor shall notify the Authority 14 days before commencing with any jacking operations.

The contractor shall report in writing all field measurements from the monitoring of the load on the needle beam, and the movements of the needle beam at the support beam and abutment to the Authority for review.

At least 7 days prior to scheduled jacking operations, the contractor shall perform a "dry run" in the presence of the Authority to verify that all jacking equipment, pumps, load monitoring devices, movement monitoring devices, and appurtenances are functioning properly.

4.2.10.2 STRUCTURAL, PAINTING, BEARINGS, & JACKING NOTES (145)

4.2.11 CAUTIONARY

4.2.11.1 CAUTIONARY LEAD PAINT NOTE

Caution: Coordinate writing of this note with PA Environmental.

The existing paint on the steel structure contains lead. At all impact locations, where the work may cause this paint to become airborne, remove the lead paint in accordance with Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website).

4.2.12 WELDING

4.2.12.1 ARTICLES ON WELDING (LINCOLN ARC)

- Welding Innovation (146)
- Consider Direction of Loading When Sizing Fillet Welds
  VOL XV, NO 2, 1998
- Consider Penetration When Determining Fillet Weld Size
  VOL XV, NO 1, 1998
- Designing Fillet Welds for Skewed T-joints – Part 1
  VOL XIX, NO 1, 2002
- Designing Welded Lap Joints
  VOL XVIII, NO 2, 2001
- Fillet Welds That Are Too Long
  VOL XVIII, NO 1, 2001
- Fundamentals of Preheat
  VOL XIV, NO 2, 1997
- Mixing Welds and Bolts, Part 1
  VOL XVIII, NO 2, 2001
- Mixing Welds and Bolts, Part 2
  VOL XIX, NO 2, 2002
- Pay Attention To Tack and Temporary Welds
  VOL XX, NO 1, 2003
- Postweld Heat Treatment
  VOL XV, NO 2, 1998
- Selecting Filler Metals: Electrodes for Stress Relieved Applications
  VOL XVIII, NO 2, 2001
- Selecting Filler Metals: Low Hydrogen

VOL XVII, NO 1, 2000

- Selecting Filler Metals: Matching Strength Criteria
  VOL XVI, NO 2, 1998
- The Importance of Interpass Temperature
  VOL XV, NO 1. 1998
- Thicker Steel Permits the Use of Opposing Arcs
  VOL XIV, NO 2, 1997
- Use Caution When Specifying Seal Welds
  VOL XVI, NO 2, 1998
- Use Undermatching Weld Metal Where Advantageous
  VOL. XIV, NO 1, 1997

4.2.13 BOLTING

4.2.13.1 ARE YOU PROPERLY SPECIFYING MATERIALS (FASTENERS) (147)

4.2.13.2 RCSC SPECIFICATIONS FOR STRUCTURAL JOINTS USING HIGH-STRENGTH BOLTS (148)

4.2.14 LEAD PAINT

4.2.14.1 CAUTIONARY LEAD PAINT NOTE

Caution: Coordinate writing of this note with PA Environmental.

The existing paint on the steel structure contains lead. At all impact locations, where the work may cause this paint to become airborne, remove the lead paint in accordance with Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website).

4.2.15 LAW DEPARTMENT LANGUAGE GUIDE

4.2.15.1 CONTRACTOR'S COMPENSATION AND CONTRACT TERMINOLOGY (149)

4.2.15.2 CONTRACTS DRAWING LANGUAGE

See: Contracts Unit Standards (150)

4.2.16 PEREGRINE FALCONE NOTES

Special Precautions for Peregrine Falcons

Note: Peregrine Falcon nesting boxes are located on Piers B and C at the Goethals Bridge, see Drawing No. 202.

A. Avoid disturbance to Peregrine Falcons

1. The Peregrine Falcon is an endangered species and shall not be disturbed. No work is to be conducted past Piers A and D of the viaducts in the main span area from early march through late august. This is the critical nesting period when brooding, hatching, and fledgling occur.
2. Should a need arise to go beyond the viaducts into the main span area, the contractor will obtain approval from the engineer before proceeding.

3. No work shall be permitted in the vicinity of the nesting boxes.
4. Other safeguards required are clear identification markers (flags, etc.) for wires or cables that span great distances over water surfaces or onto new structures. Falcons may fly into unmarked or obscure cables or wires.

B. Avoid attack by Peregrine Falcon

1. Install safeguards (fencing, netting, etc.) to discourage falcon nesting/roosting on cranes and construction equipment.
2. Access to a ledge where a peregrine falcon has landed is prohibited to protect workers as well as the falcons. Peregrines are known to dive at nest territory intruders at high speeds, attacking with sharp talons. They usually hit the high points on a human (head or hands in the air).

4.2.17 Fire Retardant Wood Notes

All timber shall be fire retardant and shall conform to the building laws of the City of New York.

4.2.18 Milling Machines

4.2.18.1 Milling Machine Restrictions at Goethals Bridge (151)

4.3 Custom Specifications

4.3.1 Structural C-Specifications

4.3.1.1 Control Joint Requirements for Concrete Specification

(Dated 07/27/07) The concrete long form specification in the Contracts Unit Standard Specifications, https://panynj.sharepoint.com/sites/Engineering/SitePages/Contracts.aspx (EOL hyperlink is an internal website), due to an oversight, does not have provisions for cutting control joints in slabs. Until the specification is revised, please include the following notes on the drawings, if control joints are required to be cut on projects where concrete long form specifications are used. The location of the control joints should be shown on the contract drawings.

The Contractor shall attempt to saw cut each control joint at the point when a thumb print cannot be made on the surface and as soon as the concrete can support the weight of the saw and the operator without marring the surface or disturbing the final finish. At a minimum, these two checks shall be performed by the Contractor every hour until the control joints can be cut. Unless otherwise shown on the Contract Drawings, saw cut depth shall be the greater of 10% of the slab thickness or one inch. Saw shall produce a cut that does not ravel or damage the concrete. If approved for use, a liquid membrane forming curing compound must be applied prior to cutting. In general, control joints for standard concrete should be cut within 6 to 8 hours of concrete placement, and within 2 to 4 hours for very high early strength concrete. However, the timing of cuts will ultimately depend on the mix proportion and the ambient temperature.

4.3.1.2 05055 Rivet Replacement (5-10-04) (152)

4.3.1.3 05130 Bearings (10-28-16) (153)

5.0 REFERENCE MATERIALS

5.1 CHANNEL CLEARANCES

5.1.1 BAYONNE BRIDGE CHANNEL CLEARANCE (154)

5.1.2 GEORGE WASHINGTON BRIDGE CHANNEL CLEARANCE (155)

5.1.3 GOETHALS BRIDGE CHANNEL CLEARANCE (156)

5.1.4 OUTERBRIDGE CROSSING CHANNEL CLEARANCE (157)

5.2 PABT (DESIGN FEATURES & GUIDELINES, STANDARDS, PRECAUTIONARY, FIELD WELDING, ETC. - AL BOCH REPORT) (158)

5.3 BASE MAPS, PA FACILITIES (159)

5.4 NY FACILITY BOUNDARIES REQUIRING PA REVIEW FOR CRANE PERMITS (160)

5.5 DATUMS – PA FACILITIES

Effective September 2005, as directed by Engineering Management Services, all PA Facility surveying/mapping projects are to be reference to NAD83NYE/NAVD88.

Traditional Horizontal & Vertical Datums

Facility	Horizontal	Vertical
Aviation		
JFK	Borough of Queens	Jamaica Bay Low Water 2.223' Below NGVD29
LGA	Borough of Queens	Flushing Bay Low Water 2.103' Below NGVD29
EWR TEB	NAD 27 NJ	PA (NGVD +297.347)
Ports		
NJMT	NAD27 NJ and Local Stationing Grids	PA (NGVD +297.347)
BP	NYE 27	PA (NGVD +297.347)
HH	Bogart/NAD83NJ	NGVD29
TB&T		
GWB	Memorial Church	0.70 Above NGVD29
LT	Memorial Church	PA (NGVD +297.347)
HT	HT (Arbitrary)	PA (NGVD +297.347)
SIBs	Bogart	NGVD29
PATH Tunnels PATH Open Area	Hudson & Manhattan NAD 27	300.00 = NGVD29

Facility	Horizontal	Vertical
WTC (original) WTC	NAD83LI	PA (NGVD +297.347) PA (NGVD +297.347)
Teleport	Bogart	Richmond County

5.6 Red Book (Specs for Design of Bridges Carrying Highway and Electric Rail Pass Traffic) (161)

5.7 PATH Track Maintenance Standards Manual, 1980 (162)

5.8 PATH – Glossary and Definitions (163)

5.9 Not used

5.10 QAD Inspection Reports (164)

5.11 Quality Control and Quality Assurance

5.11.1 Quality Control Plan (165) (Internal EOL Website)

5.11.2 Guidelines for Auditing Work of Design Consultants (166) (Internal EOL Website)

5.11.3 QA Documentation (167) (Internal EOL Website)

5.11.4 QA/QC Instructions (168) (Internal EOL Website)

5.11.5 QC Structural Checklist (169)(Internal EOL Website)

5.12 Estimating

5.12.1 General Estimating Guidelines (170) (Internal EOL Website)

5.12.2 Construction Cost Estimates (Procedures) (171) (Internal EOL Website)

5.12.3 Hourly Wage Rate Table Instructions (172)(Internal EOL Website)

5.12.4 NJDOT Bid Prices (173)

5.12.5 NYSDOT Weighted Average Price Reports (174)

5.13 CAD Standards (175)

5.14 Engineering Estimating Tools (176) (Internal EOL Website)

5.15 Bridge Deck Evaluation Manual, NYSDOT (177)

5.16 OVERHEAD SIGN STRUCTURE INVENTORY & INSPECTION MANUAL, NYSDOT (178)

6.0 REFERENCE LINKS

1. [Online] https://panynj.sharepoint.com/sites/Engineering/_layouts/15/DocIdRedir.aspx?ID=ENGDEPT-387386333-90.

2. [Online] https://panynj.sharepoint.com/sites/Engineering/_layouts/15/DocIdRedir.aspx?ID=ENGDEPT-387386333-90.

3. [Online] https://panynj.sharepoint.com/sites/Engineering/_layouts/15/DocIdRedir.aspx?ID=ENGDEPT-387386333-90.

4. [Online] https://panynj.sharepoint.com/sites/Engineering/_layouts/15/DocIdRedir.aspx?ID=ENGDEPT-387386333-90.

5. [Online] https://panynj.sharepoint.com/sites/Engineering/_layouts/15/DocIdRedir.aspx?ID=ENGDEPT-387386333-90.

6. [Online] https://panynj.sharepoint.com/sites/Engineering/_layouts/15/DocIdRedir.aspx?ID=ENGDEPT-387386333-90.

7. [Online] http://www.astm.org/.

8. [Online] https://www.fmglobalcatalog.com/FMGlobalResourceCatalog/LogIn/tabid/4830/Default.aspx?returnurl=/Default.aspx.

9. [Online] http://www.concrete.org/.

10. [Online] http://www2.iccsafe.org/states/newyorkcity/Building/Building-Frameset.html.

11. [Online] http://codesonline.nfpa.org/index.php?task=checksession&sso=0.

12. [Online] https://www.osha.gov/.

13. [Online] https://panynj.sharepoint.com/sites/Engineering/_layouts/15/DocIdRedir.aspx?ID=ENGDEPT-654783833-1390.

14. [Online] http://www.state.nj.us/transportation/eng/specs/2007/Division.shtml.

15. [Online] http://www.state.nj.us/transportation/eng/documents/BSDM/.

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