STRUCTURAL ENGINEERING CONSIDERATIONS REGARDING
POTENTIAL MELLON ARENA REUSE CONCEPTS

Prepared for:
The Sports and Exhibition Authority of Pittsburgh and Allegheny County (SEA)
Oxford Development Company
Chronicle Consulting, LLC

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1.0 EXECUTIVE SUMMARY

Ammann & Whitney has conducted investigations to provide a preliminary assessment of the feasibility of removing existing concrete seating in the Mellon Arena. The primary focus of this assessment was to establish if the seating, as originally constructed, contributes to the structural integrity of the arena roof. These investigations have included: 1) Review of available drawings, construction records, photographs, etc.; 2) Interviews with present and/or former Ammann & Whitney employees who may have first-hand knowledge or experience with the design and/or construction of the Mellon Arena building; and 3) limited visual field observations of Mellon Arena conducted on June 29, 2010.

Based on these investigations, Ammann & Whitney has concluded the following:

1. The retractable roof, supported by the concrete ring girder and its foundations, appears to be designed and constructed as a self-supporting structure without interior vertical or lateral supports. Specifically, structures from Column Line G inward do not contribute to the lateral or vertical stability of the retractable roof and its supporting concrete ring girder, given a callout on the original drawings for an expansion joint between the roof support structure and the seating bowl. This expansion joint was verified at the site, and confirmed that the structures are fully independent.

2. It may be feasible to remove the seating elements and possibly some of the structural elements located inward from Column Line G. A complete stability analysis would be required to verify this conclusion and some building elements may need to remain in place to insure continued stability, specifically the exterior concrete slab at the entrance level. Snow load on the retractable roof in the open position may also need to be analyzed. The proposed demolition plan for the elements to be removed will also require analysis.
2.0 FINDINGS

2.1 Review of drawings, construction records, photographs, etc.

Ammann & Whitney reviewed information from several sources to gather background information for this report, including Ammann & Whitney original structural drawings and Mitchell and Ritchey original architectural drawings. Several documents specifically indicate that the retractable roof and concrete ring girder have no interior vertical or lateral supports, specifically:

- A copy of “Pittsburgh Public Auditorium Retractable Roof”, by E. Cohen and R.H. Goldsmith, *Welding Journal*, page 501, May 1961 (Appendix B) was obtained and reviewed. This document makes specific mention of the fact that the supporting ring and roof structure were designed to be structurally independent from other structures.

- Available photographs from the original construction were reviewed and the ring was observed to have been constructed independently from the seating bowl.

- Original design drawings specifically call for a 1” expansion joint between column line G and the inner seating area. This confirms that the structural intent of the design was for the inner seating area to be structurally isolated from the exterior roof support ring.

- The original design drawings also indicate that the Level 2 (entrance level) concrete floor slab from column line G outward (towards the exterior of the Arena) was designed to be constructed integrally with the ring girder vertical columns.
2.2 Interviews with present and/or former Ammann & Whitney employees

Ammann & Whitney Project Manager John P. Menniti, PE spoke with a former employee on June 22, 2010 regarding the design and construction of the Pittsburgh Auditorium project. This person was a Project Engineer on the Auditorium, and stated the following regarding this study:

- He recalls the concrete ring girder and retractable dome as being self-supporting, and not tied to any interior feature.

- He noted that the seating bowl was not designed by Ammann & Whitney and, therefore, its structural capacity to brace the concrete ring girder would not have been assumed in the original design.

No other surviving Amman & Whitney employees with experience on the project could be located.
2.3 Summary of field investigation of June 29, 2010

Ammann & Whitney employees John P. Menniti, PE, Nicholas Roberts, PE, and Alex Ludinich performed a field investigation to visually verify as much as possible the results of the findings in paragraphs 2.1 and 2.2 above. Also participating in the investigation were Paul Simpson (Mellon Arena Building Superintendent).

There were two primary objectives of the field investigation:

1) Verify the presence or absence of any connections of the seating bowl (or other interior elements), either by design or addition after the building opened in 1961, to the concrete ring girder that may constrain or influence its structural response. This must be determined in regard to the extent of the potential demolition of the building’s interior for three scenarios: a) connections that are present by design and must remain in place for structural stability; b) connections not in the original design that are present and may be beneficial for structural stability; or c) connections not in the original design that are present and may NOT be beneficial for structural stability.

2) Verify the presence or absence of any internal radial connections of the concrete ring girder’s foundations or supporting columns to the seating bowl (or other interior elements), or between foundations. Scenarios a) thru c) described above are applicable for these elements also.

Regarding Item 1), struts were constructed from the Section E/F seating added in 1988 to the concrete ring girder, to provide lateral support for these new sections. The struts connect to the inside of the ring girder, and do not prohibit the opening of the dome. The presence and/or removal of these struts has no effect on the structural response of the ring girder. No other connections to the ring girder were found other than the columns/foundations as originally designed and constructed.

Regarding Item 2), radial connections from the concrete ring girder’s foundations or supporting columns to the seating bowl (or other interior elements), or between foundations were not shown in the original construction documents, and no evidence for construction of these connections was found. In some instances, seating bowl columns are adjacent to (and in contact with) vertical concrete ring girder columns, however no connection between the columns was evident. Evidence of possible new foundations and/or subsurface bracing was observed due to sawcuts in the original concrete floor slab(s), however in all instances the sawcut area was adjacent to the original ring girder columns and aligned with steel framing added after the original construction.
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This work did not include any evaluations, designs, or calculations for potential reuse alternatives, or a structural analysis of the existing structure using modern analysis tools or software for wind, snow, and other loading conditions stipulated in current building codes. The goal of this investigation was to determine the original design intent of the Pittsburgh Auditorium concrete ring girder and retractable dome structure as designed by Ammann & Whitney, and their applicability as self-supporting structures for possible reuse alternatives. A thorough structural engineering evaluation necessary to ensure a safe structure for use as a public facility will be required. The specifics of each reuse alternative, and their potential impacts to the concrete ring girder and retractable dome, must be evaluated.
APPENDIX A

Ammann & Whitney resumes for personnel who developed this report
Experience Summary

Mr. Menniti has over 30 years of engineering and construction experience, with 20 years in management. He has a proven record of providing high quality and responsive service to PennDOT as well as other public and private sector clients in Pennsylvania.

His experience includes operations management, project management, design, value engineering, procurement, construction oversight and project closeout. Mr. Menniti has worked as a consultant in both the public and private sector, and also has eight years experience with the U.S. Army Corps of Engineers in the United States and Europe.

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PROJECT EXPERIENCE

SR0065 Bridge over Lowries Run, Emsworth, Pennsylvania, PennDOT District 11-0. Project Manager for the total replacement of the deteriorated 80 ft span steel thru-girder bridge carrying Ohio River Boulevard over Lowries Run. Ohio River Boulevard is the major highway route on the north bank of the Ohio River from Rochester in Beaver County into the City of Pittsburgh. It carries four lanes of traffic over the stream. The bridge width is deficient (lanes are approximately 10 ft wide with no median). Underground and overhead utilities are in the immediate vicinity, including both a sanitary sewer line and a pipeline attached to the bridge. Responsibilities include preliminary engineering, structural and roadway design, Phase I ESA, public involvement, supplemental field survey, geotechnical engineering, H&H report, cultural resources investigation and construction support. Key to the project is developing a construction staging sequence that will require a temporary support system, or “carrier beam”, to support half of the existing superstructure that will remain open and maintain two lanes of traffic during construction.

Mon-Fayette Expressway, Section 51-B, Fayette County, Pennsylvania Turnpike Commission – Project Manager, construction phase services. This $46M section includes 8,366 feet of new limited access highway, a 720 foot five-span prestressed concrete bridge, and a 1,000 foot five-span steel girder.
bridge. Responsible for supervision of shop drawing review, response to RFI's, client invoicing, and interfacing with Constructware construction management system.

**Mon-Fayette Expressway, Section 53-F, Allegheny County, Pennsylvania Turnpike Commission.**
This $156M section includes 8,500 feet of new limited access highway and a 3,500’ viaduct, in a dense urban area in Allegheny County, Pennsylvania. Responsible for right-of-way and utilities relocation design in the Design Field View phase of the project. Project includes interaction with four municipalities, 87 total takes, 46 partial takes, relocation of 8,500 feet of a railroad line, and interaction with eight utility companies which will require $5M in utility relocations.

**Grove Chapel Climbing Lane on S.R. 119, Indiana, Pennsylvania, PennDOT District 10-0.** Project Manager for the widening of a 1.6-mile section for a truck climbing lane and center turn lanes. Six online alternatives were initially developed for the widening, taking into account sensitive areas such as a church and cemetery, a large commercial nursery, and the replacement of two culverts. Two additional offline alternatives were then developed to address safety issues and design exceptions at the request of the District. One public meeting was conducted. A WELCOM schedule was developed and submitted to the District.

**Nixon Road Bridge Replacement, Harmar, Pennsylvania.** Project Manager for the replacement of an existing four span steel girder bridge over the Bessemer & Lake Erie Railroad. This was a local project designed and constructed under the supervision of PennDOT District 11-0. Two online and two offline alternatives were developed to improve the vertical and horizontal alignment of the structure. An online, three span concrete box structure was selected. Close coordination with the operating railroad was required for the project.

**North Avenue and Sherman Street Bridges, Millvale, Pennsylvania.** Project Manager for the replacement of two existing single span bridges over Girtys Run. These were local projects designed and constructed under the supervision of PennDOT District 11-0. Both structures were located in the central business district of Millvale, PA. The North Avenue Bridge was constructed over a 30” sewage force main owned by the Girtys Run Sanitary Authority, and relocation of the line was required with no closures and accomplished by bypass pumping.

**Industrial Center of McKeesport, Pennsylvania.** Project Manager for the design and construction in several phases of approximately 3500’ of new roads in a reclaimed brownfield site at the former McKeesport USX works. This was a local project designed and constructed under the supervision of PennDOT District 11-0. Excavation through the former steel mill site was closely monitored for payment item quantities, historic artifacts, and environmental contamination. Two at-grade railroad crossing were included in the project.

**Little Calumet River Flood Protection Project, Hammond, Indiana.** Project Manager for the design of four miles of flood control levees, pump stations, and bridges for the U.S. Army Corps of Engineers Chicago District. Project is located in a residential neighborhood and private golf course, and community sensitive design issues and public involvement were critical.

**Saluda Dam Remediation Project, Columbia, South Carolina.** Performed final design, specification development, document review, bid package assembly, quality control, and resident engineer services for this $200M combined Roller Compacted Concrete (RCC) and rockfill berm. The new berm is being constructed immediately downstream of the existing dam under full hydraulic load, without interruption to the hydroelectric plant operations and traffic on South Carolina Route 6.
Experience Summary

Mr. Riley has 23 years of experience in the inspection and design of bridge and highway projects. Mr. Riley's design experience encompasses preliminary engineering through final design and the preparation of PS&E documents for a wide variety of transportation related project for PennDOT, PTC, MDDOT and SJTA. He has performed bridge inspections for PennDOT, MDDOT, ConnDOT and the Delaware River Port Authority. As an inspection Team Leader/Bridge Inspector, he was responsible for the inspection, evaluation and load rating of more than 2,000 bridges throughout the Mid-Atlantic region, ranging from steel trusses to covered timber bridges, concrete arches to steel and prestressed concrete multi-girder bridges. He also has prepared hydraulic and hydrologic reports.

Relevant Experience

Riegelsville Toll-Supported Bridge Rehabilitation, Delaware River Joint Toll Bridge Commission: Project Manager for the rehabilitation of this historic 577’ three span suspension bridge across the Delaware River from Bucks County, PA to Warren County, NJ. The bridge was constructed in 1904 by John A. Roebling's Sons Co. Project objectives are to replace the floor system as well as evaluate the need for any applicable superstructure and substructure repairs (above the water line). Work includes performing an in-depth inspection, load rating, paint system analysis, concept study report, structural evaluation, preliminary engineering, final design and construction support services.

SR0065 Bridge over Lowries Run, Emsworth, Pennsylvania, PennDOT District 11-0: Quality Control Engineer for the total replacement of the deteriorated 80 ft span steel thru-girder bridge carrying Ohio River Boulevard over Lowries Run. Project includes structural and roadway design, public involvement, supplemental field survey, geotechnical engineering, H&H report, cultural resources investigation and construction support. Key to the project is developing a construction staging sequence that will require a temporary support system, or “carrier beam”, to support half of the existing superstructure that will remain open and maintain two lanes of traffic during construction.

Trenton-Morrisville Toll Bridge Rehabilitation, Delaware River Joint Toll Bridge Commission, Trenton, NJ/Morrisville, PA: Quality control for preliminary engineering, final design and construction support for redecking the south Pennsylvania overpass, wall and ground mounted sound barriers, new retaining wall, rehabilitation/retrofit of all retaining walls on Pennsylvania approaches, and new sign structures. (2005-2009)

Tacony-Palmyra and Burlington-Bristol Bridges, Burlington County Bridge Commission, Burlington County, NJ: Quality Control for the load ratings of the Tacony-Palmyra and Burlington-Bristol Bridges over the Delaware River. This project involved the structural analysis of several
different and complex structures, including a tied arch, bascule span, through truss spans, tower spans and a lift span. (2006-2008)

**Mon/Fayette Expressway, Route 51 to Pittsburgh, Turtle Creek to Business Route 22 (Section 53F), PTC, Allegheny County, PA:** Project Manager for preliminary engineering and final design of a $150 million, two-mile limited access four-lane highway, a mainline toll plaza, a 3,600-ft dual high level signature viaduct, mainline bridge and one Union Railroad bridge over Thompson Run and nine retaining walls. Project includes alternative studies for horizontal and vertical alignments, alternative studies for the signature viaduct, development of design field view and conceptual TS&L submissions, storm water management, hydraulic and hydrologic analysis of the impacted creek, ROW acquisition, preparation of E&S plans, utility coordination, geotechnical investigation and final contract documents. Monthly meetings with local design advisory team are also included with this project. (2005-present)

**Mon/Fayette Expressway, Uniontown to Brownsville, Section 51B, PTC, Fayette County, PA:** Deputy Project Manager and Project Structural Engineer for work associated with the design of 8,366 ft of new four-lane roadway and two major multi-span bridges. The first bridge is a 720-ft long four-span curved steel multi-girder structure. The second bridge is a 1,000-ft long five-span steel multi-girder structure with 150 to 200 ft tall piers. The project was designed on an accelerated schedule.

**I-80 Clearance Resolution, SR 0209, Section 16B, and Feasibility Study for Intersection, SR 0209, Section 017, of SR 0209 and SR 0447, PennDOT, Monroe County, PA:** Project Manager for preliminary engineering and final design for the elimination of substandard vertical clearances of the bridge carrying SR 0209 over I-80 and the feasibility study for realigning of the intersection of SR 0209 and SR 0447. Project includes horizontal and vertical alternative studies, preparation of CE2 documents, preparation of contract documents and feasibility study report.

**Total Reconstruction Project, MP 214 to 227, PTC, Cumberland County, PA:** Project Manager for this rehabilitation/replacement project to widen the Turnpike from 82 ft to 126 ft. Ammann & Whitney is responsible for preliminary design of all overhead structures to be replaced (13 total), final civil & structural design for one overhead bridge replacement, preliminary & final structural design for widening or replacement of mainline structures over local waterways (five total), and redesign of the existing Carlisle interchange (including a new bridge over the mainline roadway). (1999-2006)

**SR 0219, Section C10, PennDOT, Elk County, PA:** Project Manager for final design and construction consultation for two new bridges (1,320 and 945-ft long structures) and retaining walls on new alignment. This is part of the relocation of an existing local arterial highway. Also known as the Johnsonburg Bypass, this project consists of 1.5 miles of roadway, three multi-span curved girder bridges over existing railroad tracks, local streets, and the Clarion River, three retaining walls, two signalized intersections and one noise barrier. Key project issues include curved steel beam analysis, bridge aesthetics, hydraulic design constructability and economy of design.

**Kratz Road Bridge Replacement, SR 4008, Section 93S, PennDOT, Montgomery County, PA:** Project Manager for inspection, preliminary engineering, alternatives analysis, final design and construction support services for the award winning design of a two-span, prestressed concrete box beam replacement structure for this historic, three-span, stone/masonry arch bridge. The bridge is located in a State park. Thus, the project also included cultural resource and environmental investigations for completion of a categorical exclusion document and Joint Permit Application. A complete hydrologic and hydraulic analysis, including HEC-RAS modeling, was also completed to determine the impact of the proposed structure on Skippack Creek.
Experience Summary

Mr. Stahmer has over 23 years of participation in the inspection, design, analyses and coordination of Ammann & Whitney's most complex projects. He has extensive experience in the application of in-house and commercial computer programs for the analysis of linear and non-linear structures with static and dynamic loads. He has gained specific expertise in the design, analysis and investigation of structures for seismic and other dynamic loadings. In addition to his tenure at Ammann & Whitney, Mr. Stahmer has held an assistant professorship at Pratt Institute teaching courses in advanced structural analysis and design.

Relevant Experience

Limited Area Processing and Storage Complex (LAPSC), NAVFAC EFANW, Naval Submarine Base Bangor-Silverdale, WA: Vice President in Charge, construction administration services for a high security military complex. LAPSC contains above ground structures and a multi-level below ground structure having a combined area of 16,000 square meters. The structures are reinforced concrete containment structures designed to withstand both interior and exterior explosions. All exterior penetrations and blast doors are gas-tight. There are a total of 24 blast doors; 12 are located in the above ground structures and 12 in the buried structure. All 12 doors in the above ground structures are subject to exterior blast loads while seven are subject to additional internal gas loads. Of the 12 doors in the buried structure, six doors are subject to blast loads from both sides, the remaining six are subject to blast loads from one side only but three must be gas-tight. Penetrations from the buried structure are blast resistant and gas-tight. Special features include: power-operated physical security and blast-resistant doors; TEMPEST shielded rooms; a new 1000 kW emergency generator enclosed within a new hardened, reinforced concrete structure; and security guard towers. The construction cost is $160 million.

Armament Integration Facility (AIF) USACOE, Picatinny Arsenal: Vice President in Charge of Design. This project includes the construction of a new, independent 12,000 square foot laboratory building and a new, 8,300 square foot, 100 meter indoor range appended to an existing facility. The range is a hardened structure and was subject to review by the DDES. Design of the range will accommodate indoor firing of up to 40mm weaponry and considers both the pressures generated by the weaponry and provisions to control ricochet effects of projectiles. The construction cost is $10 million.

New Terminal, JetBlue Airways, JFK International Airport, NY: Project Manager and Engineer of Record responsible for all structural engineering for a new $465 million, 26-gate terminal, including an elevated roadway and provisions for future expansion to 30 gates. The new terminal is being built behind JFK’s landmark TWA international terminal. It will connect to this terminal as well as to the new AirTrain and a new parking garage. The
location of the new terminal contributes to two key design challenges. First, the JetBlue terminal will be built on an existing apron that encompasses a myriad of utilities underneath. These required careful planning to enable maintenance of all ongoing airport operations during construction. Second, the connection to the TWA terminal required consideration of the existing building’s landmark status. An additional project complexity was the very aggressive schedule. Design was completed in 14 months to allow for a fall 2008 opening. Multiple construction packages were prepared, including demolition, foundations, structural steel and fit-out. When complete, the JetBlue terminal will accommodate 46,000 passengers per day on 500 inbound and outbound flights. At peak hour, 8-9:00 a.m., it will handle 4,600 passengers on 45 flights. The construction cost is $465 million.

**Chennai International Airport Terminal Expansion, Airports Authority of India, Chennai, India:** Structural Project Manager and Structural Engineer for the design of a new 140,000 m² terminal. As a member of the design team with the winning concept design, Mr. Stahmer led the structural design effort for the new domestic and international terminals, a kilometer long elevated departures level roadway and four 1500 car parking garages. The terminal buildings are enclosed with large, arching trusses that provide a column free space of over 210 by 39 meters in the ticketing hall and hold room areas. The open quality of these trusses and the glazed curtain walls allows the lush gardens, situated between the landside and airside terminal elements, to be experience by passengers moving through the building. In addition to his role as the structural project manager Mr. Stahmer was personally responsible for the structural design of the precast concrete roadway structure and elevated pedestrian walkway.

**Regional Commuter Terminal, US Airways, Philadelphia International Airport, PA:** Project Manager and Engineer of Record for design of a new $26 million, 38-gate regional commuter terminal. Engineering services included developing schematic through final design construction documents and construction support for a new terminal and a new elevated, enclosed pedestrian bridge. The pedestrian bridge connects the new terminal to existing terminals. The terminal was designed and will be constructed on a fast-track basis (28 months from notice-to-proceed to estimated date for useful occupancy). The 165,000 sf, single-level hub-airline terminal will include a passenger ticket lobby, passenger movement areas, security check-points, holdrooms, baggage claim area, baggage handling facilities, concessions and other passenger amenities, airline club, airside shuttle bus lounge, airline operations area and ticket office space. During construction of the terminal, the project contractor encountered quality control problems with the alternate design-build foundation system that he substituted for the Ammann & Whitney design. Mr. Stahmer worked closely with the contractor, construction management team and owner to solve problems with in-place piles. The construction cost was $65 Million

**Journal Square Transportation Center, Master-Planning Study, Journal Square, NJ:** Structural consultant for a major masterplanning effort investigating options for transforming a large urban transportation hub into a commercial center. Goals of the project included review of feasible overbuild options to allow the revitalized facility to function as an economic hub and engine for the surrounding area.

**Facility Conversions, MWAA, R. Reagan Washington National Airport, VA:** Project Coordinator and Structural Engineer for design of two fast-track adaptive re-use projects. The work includes conversion of a six-gate US Airways terminal to a hangar and conversion of a three-gate Delta terminal to an office complex. The US Airways facility conversion includes reinstallation of aircraft hangar doors, rehabilitation of the building’s west facade (including preservation of historic windows) and inspection and roof repairs. Adapting the Delta Terminal was a complex task requiring conversion of a temporary elongated L-shaped terminal into a functional office building. The extensive modifications will result in a new office building with a recognizable image connoting its function as the agency’s administrative headquarters facility. The Construction cost was $8 million
Experience Summary

Mr. Roberts has been involved in the industry of bridge design and construction for eight years. During that time, Nicholas has been involved in a wide range of industry services including design, inspection, research, construction services, and construction inspection which has allowed him to develop a broad knowledge of the industry and its practices.

He began his engineering career in New York City where he played an integral role in the construction services and design of complex moveable bridge structures such as the Woodrow Wilson Bascule Bridge in Washington, D.C. and the Third Avenue Swing Bridge in New York, NY.

Since that time, Nicholas has had experience with several different clients including state and city DOT's, turnpike authorities, and other public agencies. He has had significant design-build experience and has developed into a talented project engineer responsible for the planning and design of new and rehabilitated bridge projects. As well as being a technically competent engineer, Nicholas has more recently taken on a greater role in marketing and business development activities.

Mr. Roberts, is an active member of the American Society of Civil Engineers (ASCE) and his other professional memberships include the American Society of Highway Engineers (ASHE) and the Association for Bridge Construction & Design (ABCD). In addition to being an active member of professional societies, Nicholas is a published author in the Journal of Bridge Engineering.

Relevant Experience

**DRJTBC, Riegelsville Bridge Rehabilitation, Riegelsville NJ/PA:** Structural Engineer for the 100 year old suspension bridge rehabilitation. The project includes assessment of the existing structure, design of the rehabilitation, and construction support services. Responsibilities include inspection and load rating of the existing bridge. In addition, responsibilities include preliminary engineering and final design of the floor system replacement.

**District DOT, Frederick Douglass Memorial Bridge, South Capitol Street over Anacostia River, Washington, DC:** Structural engineer for the fracture critical flanking spans of the bridge originally constructed in 1950. Each of the two flanking spans consists of two cantilever spans and a pin-and-hanger suspended span. Responsible for the load ratings and fatigue evaluation performed on the bridge superstructure, including the pin-and-hanger assemblies.
PennDOT, S.R. 0676, Vine Street Expressway Overhead Bridge Replacements, Philadelphia, PA: Structural Engineer for the development of Type, Size and Location Reports for the superstructure replacement of existing bridge structures over the Vine Street Expressway. This project includes replacement of the two-span bridge superstructures with single span structures under staged construction. Technical responsibilities included the evaluation of existing abutments and substructure units to be salvaged and reused in conjunction with the newly designed superstructures.

PennDOT, S.R. 0676, Vine Street Expressway Overhead Bridge Replacements, Philadelphia, PA: Structural Engineer for the development of Type, Size and Location Reports for the superstructure replacement of existing bridge structures over the Vine Street Expressway. This project includes replacement of the two-span bridge superstructures with single span structures under staged construction. Technical responsibilities included the evaluation of existing abutments and substructure units to be salvaged and reused in conjunction with the newly designed superstructures.

Virginia DOT, I-495 HOT Lanes, I-495 over Arlington Boulevard - Washington, DC: Structural Engineer responsible for the final design and oversight of plan development for four prestressed concrete bulb-tee girder (continuous for live load) grade separation structures on multi-column concrete bents and integral abutments.

PennDOT, S.R. 0065, Ohio River Blvd. over Lowries Run, Bridgeville, PA: Structural engineer for the design of a complex moment slab required as a result of District safety review meetings. Responsible for final design and detailing of the moment slab and oversight of plan development.

PennDOT, S.R. 3011, Keel Ridge Road over Hogback Run, Hermitage, PA: Structural engineer for the preliminary design of a precast reinforced concrete box culvert.

Virginia DOT, I-95/I-395 HOT Lanes, HOT Lanes Flyover Ramp over I-95 - Washington, DC: Structural Engineer responsible for design, quantity estimation, and plan sheet development for the ready-for-estimate phase of this design-build project. Responsible for design tasks including the design and quantity calculations for the 3-span curved continuous steel plate girder superstructure and integral steel pier cap beams. Substructure design tasks include the design of concrete hammerhead piers and a welded steel box girder straddle bent.

Virginia DOT, I-95/I-395 HOT Lanes, Telegraph over I-95 - Washington, DC: Structural Engineer responsible for design, quantity estimation, and plan sheet development for the ready-for-estimate phase of this design-build project. Responsible for design tasks including design calculations for the 2-span continuous steel plate girder superstructure.

NYCDOT, Terrace Bridge, Hill Drive over Prospect Park Lake – Brooklyn, NY: Structural Engineer responsible for final design and plan development for this open spandrel deck arch bridge built in 1890 and listed on the National Register of Historic Places.

NYCDOT, Brooklyn Bridge, Brooklyn Bridge over East River – New York, NY: Structural Engineer responsible for in-depth inspection tasks associated with the Manhattan approach ramps and the Manhattan approach structure of the Brooklyn Bridge in order to determine the need for seismic rehabilitation.
APPENDIX B

Pittsburgh Public Auditorium
Retractable Roof
for weatherproof enclosure can be quickly opened to provide spectacular open-air stadium to accommodate 13,600 persons

BY E. COHEN AND R. H. GOLDSMITH

SYNOPSIS. The design of the roof for the nearly completed public auditorium in Pittsburgh, Pa., is described with special attention to the extensive utilization of welded components and connections.

The unique feature of the auditorium is its vast retractable roof. The first such dome-like roof ever built and one of the largest clear-span roofs in the world, its eight 45 deg sections or leaves (six movable and two stationary) make possible a weatherproof auditorium that can be converted to a spectacular open-air stadium in 2½ min. The configuration when closed is that of a compound spheroid nearly circular in plan, approximately 417 ft in diam, and 109 ft high at the center.

The dome has no interior supports, the leaves being supported at their bases on motorized wheel carriages. These rest on rails mounted on a reinforced concrete ring girder which is 34 ft above the arena floor. At the crown the leaves rotate on the pins of a multiple clevis mounted at the end of a space frame that cantilevers from outside the dome and terminates at the center.

The cantilever space frame, which is the main support for the leaves, is a triangular truss in both plan and cross section. The bottom chord is a curved box girder approximately 8 ft wide and 17 ft 6 in. deep embedded in a concrete abutment at the base. The top chord tie-back members are 3- x 3½-ft box sections and extend from the anchorage points up to a point on the box girder near its top. The entire cantilever space frame truss weighs about 1400 tons.

The upper joints of the cantilever space frame are formed of stress-relieved shop weldments. These weldments, connecting as many as seven large box-section members in one shop welded joint and developing all four sides of each box, are believed to be the first of their kind. The box-section members between joints are fabricated separately and connected to the weldments in the field with high-strength bolts. The joints on the lower chord are also stress-relieved shop weldments which are fastened to the box girder by field welding. The segmental arch rib of the roof leaves, the wheel carriages, the pivot assemblies, the tie back anchors, the rail supports and numerous other elements of the structure were designed for and fabricated by welding.

All welds were inspected visually and either by radiographic or ultrasonic testing. Dye penetrant tests were made in some cases. Special procedures were developed for the main field welds in the space frame. Weldment plates were checked for lamination with ultrasonic apparatus. Computed deflections were verified by load tests on the actual structure.
General Description
The Pittsburgh public auditorium now near completion will be the key structure in Pittsburgh's Lower Hill Redevelopment. This $20,000,000 structure will be able to accommodate up to 13,600 persons. The auditorium, including mall and parking area, will occupy about 20 acres. Its diversity of equipment will permit a range of performances from grand opera to water shows, ice skating and rodeos—Fig. 1.

A unique feature of the auditorium is its vast, dome-like, retractable roof with a total surface area of 166,000 sq ft, sheathed in stainless steel. The first such roof ever built, its movable leaves make possible a weatherproof auditorium that can be converted to a spectacular open air stadium in 2½ min—Fig. 2. Further, it is one of the largest clear span roof structures in the world, the maximum span of roof being approximately 417 ft measured from base of rails and the average rise of roof leaves, measured from pin center to base of rail, being about 109 ft—Fig. 3. The entire dome is mounted on a reinforced concrete ring girder approximately 34 ft above the arena floor. It is divided radially into eight 45-deg sections or leaves, six movable and two stationary, as shown in Fig.
4. To retract the roof, the six movable roof sections roll on concentric circular rails, telescope one over the other, and finally nest in sets of three over the two fixed roof sections to open the arena to the skies.

The dome has no interior supports. Instead, it is supported at its zenith by an exterior curved steel space frame of triangular cross section that cantilevers from outside the dome and terminates at the top in multiple clevises for the leaf pivot pins. The bottoms of the roof leaves rest on a series of motorized carriages which roll on steel rails anchored to the concrete ring girder.

**Roof Leaves**

**Structural Framing**

In plan view, each roof leaf is a 45 deg sector of a circle. The top leading leaf has a radius of 207 ft and weighs 360 tons. Its curved base is about 162 ft long, and the distance from ring girder rail to pivot (measured along the curved axis of the roof arch ribs) is about 250 ft. Leaves vary slightly in size so they can nest together. Each leaf is about 3 ft thick, including roof and ceiling. Structural framing is shown in Figs. 5 and 6, the radial ribs, being 30-in. WF beams. Each rib is composed of a series of straight beam sections mitered and then spliced together by shop butt welds to form straight chords following the compound spherical surface of the dome, the shape approximating the dead-load string-polygon. The butt welds were formed by welding the webs first. The rib portion was then turned 90 deg, and the upper and lower flanges were welded in the flat from one side, using backing strips. Finally, it was turned 180 deg, the backing strip removed and the root passes gouged and rewelded. This welding was performed with semiautomatic submerged-arc equipment.
Prior to welding the flanges, the web was preheated in the weld zone. Field connections are made with high-strength bolts. These splices are located in the straight portions of the ribs between mitered, and the ends of beams are milled to bear. The ribs are spaced 27 ft apart at the base of the top leaf. As a leaf decreases in width toward the apex, alternate ribs are discontinued, their loads being transferred by cross framing to adjacent ribs. To keep the over-all thickness of the nested leaves as small as possible, the clear distance between them is reduced to a minimum. Consequently a high accuracy of fabrication and erection is required to prevent binding between leaves. That all the ribs of a leaf must concurrently bear against one common milled surface on the pivot weldment is further reason for accuracy. For these reasons each rib was shop assembled in its entirety, with the milled ends at all field splices tightly drawn up.

The leaf ribs were designed for two conditions of loading: (a) for the conventional stresses under dead load and uniform live load of 15 psf over the entire leaf plus a pattern load of 15 psf placed for maximum effect at the design sections, and (b) for yield point stresses under dead load plus a pattern load of 50 psf on one half placed for maximum effect plus a uniform load of 20 psf over the other half of the roof. Because of the nonlinear increase in the secondary (deflection) stresses which are augmented by the large transverse horizontal deflection of the cantilever frame at pivot points, the second condition usually controlled. A transverse deflection at the pivots amounting to about 1 ft is maximum for a 30 psf differential live (snow) load on half the roof.

Purlins consist of 8- and 10-in. WF beams spaced 7½ ft apart and end connected for full continuity span between the radial ribs to support the roofing. Diagonal bracing between the ribs and purlins stiffens the leaves against the traction, wind and bumper forces. Purlins and bracing are field connected to gusset plates by high-strength bolts. The shop welding of the hundreds of these gusset plates to the ribs affords a great simplification in details and savings in weight.

Leaf Roofing

The leaf roofing as shown in Fig. 7 consists of Robertson 2-20-Q decking and rigid insulation with an exterior covering of stainless steel of AISI
 designation Type 302, Finish 2-D and of 20 and 22 gage. Bolts with rubber and steel washers under their heads clamp down the decking. Circumferential decking joints with oversize holes permit sliding and minimize distortion of the decking from interaction with the deflecting leaf. Also to avoid interaction and to allow for differential expansion and contraction due to temperature, the radial joints in the stainless steel gage metal are perpendicular and covered with battens, as illustrated, and the circumferential joints are flexural lock seams to permit sliding.

At the top 50 ft of each leaf, which is at a flat slope, the joints are made by field seam welding. False welded caps are attached to continue the batten lines for the sake of appearance. The hung ceiling of each leaf, rigidly suspended from the structural framing, is of perforated steel sheet, zinc coated and with a baked enamel finish. Expansion joints are located in the ceiling between ribs.

Pivot Weldments on Leaves

As illustrated in Fig. 8, the converging ribs at the apex of each leaf join into a single pivot weldment which delivers the leaf thrust—a maximum of 335 tons—through the pivot to the cantilever frame. Each of these weldments is made up of a combination of one carbon steel casting and four short lengths of WP beams, all shop welded together and stress relieved. The short beams are butt welded to the casting and form stubs to which the ribs are field bolted. The casting has a 23-in. diam bore to fit a double flanged Lubrite lining. The inside of this lining is machined to fit a stainless steel spherical knuckle over which the leaf can rotate in any direction without end restraint to accommodate relative rotation between a leaf and supporting cantilever frame. The stainless steel knuckle, bored to accommodate a Lubrite bushing, is mounted on a vertical 12-in. diam pin fixed between two of the ears of the multiple clevises at the end of the cantilever frame. Opening and closing movements take place around this pin. Stainless steel and self-lubricating Lubrite bearing surfaces have been used at both the spherical and cylindrical surfaces so as to reduce maintenance, access being difficult.

Wheel Carriages

The wheel carriages at the base of each leaf rib are 27 ft apart for the top leaf, and 26½ ft for the two intermediate leaves. All carriages are double-wheeled, except for the end carriages of the intermediate leaves and the trailing end carriages of the top leaves, these being single-wheeled. Each double-wheeled carriage supports the leaf by means of a 7-in. diam forged steel equalizer pin passing through a Lubrite-lined casting bolted to the leaf. Each of the 30-in. diam rim-roughened, wrought-steel, double-flanged wheels is designed for a maximum reaction of 60 tons in the plane of the wheel accompanied by a transverse force of 4 tons delivered against the flange when the roof is closed and has full live load. Under motion, each wheel is designed for a reaction of 35 tons, a transverse 2 tons force on flange and a tractive force of about 13 tons. The wheels, weighing 1500 lb each, rotate on pairs of spherical roller thrust bearings which are mounted on 6-in. diam forged steel shafts fixed to the carriage side plates. The carriages, themselves, are stress-relieved weldments made up of stiffened plates of thicknesses up to 2½ in. The weight of each double-wheeled carriage, exclusive of motors and brakes, is four tons.

The 25-hp drive motors and brakes are mounted above the intermediate carriages and are connected to the wheels by means of sprockets and roller chain drives. Only one wheel of each carriage is driven, but both are braked through gear reducers. A self-contained hydraulic buffer is mounted at each edge of each movable leaf, should the leaves hit the fixed bumpers or the cantilever girder at too high a speed in the event of control failure. Under operating conditions, opposing pairs of leaves will operate symmetrically. The control system not only insures this operating condition but controls the differing rates of movement of each of the various pairs of leaves. The controls, the many safety features, and the comprehensive trouble indicator panel result in a highly complicated and unique electrical design.

Rails

The reinforced concrete ring girder serves to support some 3000 linear ft of rail, curved to radii averaging about 200 ft. Each rail is accurately bent about both its axes so that its web lies on a conical surface.
The radial closure at each of the six trailing edges of the two movable leaves is accomplished by a flexible, bulb-shaped neoprene seal suspended from the upper leaf and extending over the full radial length of the joint. Flexible neoprene curtains form a weatherproof connection between the bulb and the upper leaf. When a roof leaf reaches its closed position, a steel cable within the bulb is tensioned by a mechanical linkage, in turn actuated by a tripper plate on the ring girder. The resulting radial component of cable tension pulls the bulb downward, compressing it against a smooth pad on the adjacent leaf below and effecting a waterproof seal. The neoprene curtains have enough slack so that the maximum expected differential deflections between two adjacent leaves will not lift the bulb from the roof surface. When the roof opens, the tension of the cable within the bulb is released, and the bulb is lifted to a retracted position by a spring tensioned suspender cable.

The circumferential closure at the bottom of each leaf is achieved by a neoprene bulb attached to the edge of each leaf. As the roof closes, the bulb moves into a position adjacent to a curved steel curtain plate mounted on the ring girder. Leaf springs clamped to stem of the bulb hold it clear of the plate when the roof is open. As the leaf reaches its closed position, a tripper mechanism tensions a cable within each bulb and compresses the bulb against the curtain plate.

Closures at other points such as the apexes of the roof leaves are made by means of various interlocking annular rings, abutting closure plates and neoprene bulbs.

**Cantilever Frame**

**General**

The cantilever, which supports the pivot points of the roof leaves, is a structural steel space frame located at the rear outside face of the roof. It is basically an inclined, curved tripod—Fig. 10. The lower leg of the "tripod"—an 8- x 7'/2-ft box girder acting as the compression member—is curved in a series of straight chords between panel points to a shape approximately following that of the roof and encroaching only a minimum amount below the ceiling line. The two upper legs of the "tripod," also chords of a curve, are 3- x 3'/2-ft box-members acting as the tension tie backs. Positioned high above the rear entrance to the auditorium in order to meet the various access and architectural requirements, they terminate in anchorages about 110 ft to the rear of the ring girder. They are held away from the box girder by triangular frames consisting of struts and cross ties. Toward the top of the girder where such triangular frames would be shallow, they are replaced by cross girders as shown in Section D-D. The profile of each tie back is a string polygon so shaped as to place minimum moment in the box girder when the girder receives its maximum compressive thrust. This occurs when the leaves are nested, at which time
they apply a total horizontal load of over 1500 tons to the pivots. The concurrent vertical load is less than 3% of the horizontal.

The triangles formed by the strut and cross tie members are supplemented by, first, a “K” bracing between tie backs and, second, by diagonals between the plane of tie backs and the top of box girder to form a space frame capable of resisting vertical, horizontal and torsional loads. Members other than the tie backs and box girder are 2- x 2\(\frac{1}{2}\)-ft box members.

The cantilever frame design utilizes the computed participation of all members, since the box girder is rigidly attached to its abutment, the space frame is indeterminate to the fifth degree.

Interconnecting the space frame members at the panel points presented a serious design problem, since as many as seven space frame box members intersect at one point. Further, few of the member sides lie in common planes, and many of the intersection angles between the members are sharp. Various riveted, bolted, pin and weldment type connections were studied before selecting the shop-welded, field-bolted, “stub-weldment” design which was finally shown on the contract drawings. It consists of a group of intersecting rectangular stubs, each suitably reinforced at its interior with welded diaphragms or plates to permit the transfer of stress from the abutting stubs welded to its exterior surfaces. All stubs are field-spliced to the

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**Fig. 10—Plan and elevation of cantilever frame**

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space-frame members by high-strength bolts. Although the contract specifications left the contractor free to select any other appropriate manner of connection, he elected to use the stub weldments such as shown in Fig. 11.

The deflections of the loaded cantilever are of particular interest because of the interdependence of leaf and cantilever design. When the leaves nest, the tip of the cantilever moves outward 3 in. and downward 6 in. An unbalanced snow load on the closed roof causes a sideward movement of about a foot. After the cantilever frame was erected, and prior to installation of leaves, a load of 50 tons was placed on its free end, deflecting it $1\frac{1}{2}$ in. and verifying the design stiffness. In order to further verify the rigidities calculated for leaves and cantilever interacting together under various loadings such as unbalanced snow load, the leaves will be opened unsymmetrically after completion, and deflection and strain measurements will be made.

**Box Girder**

The 8- x 17½-ft box girder of the cantilever frame is a riveted box member divided into an upper and lower section by a continuous web at mid-height. Its lower section has a stairway for access from the auditorium to a winch platform suspended directly beneath the leaf pivot points. Its interior also provides a path for the many cables required for lighting, operation and control. The base of the box girder is embedded in a reinforced concrete abutment in a manner to transfer the thrust, moment and torsion reactions to the concrete—Figs. 12 and 13. As shown in Section E-E of Fig. 10, a “pickback” stub weldment is mounted on top of the box girder at each point where struts and diagonals frame in. Its manner of connection will be described later. Near the upper end of the
box girder the struts and cross ties are replaced with cross girders riveted to the box girder. Finally, the tension tie members converge toward the box girder and are connected to it by tee flanges as shown in Section C-C. The front end of the box girder terminates in the clevis weldment described below.

Since the concurrent thrusts of the roof leaves on the box girder vary in direction and sometimes in magnitude, the resultant force of these concurrent leaf thrusts upon the clevis weldment is usually eccentric to the centerline of the box girder. The resultant forces introduced by the "pickaback" stub weldments—Fig. 14—are also often eccentric. Hence, the entire box girder and its clevis weldment—Fig. 15—are designed to withstand the resulting bending and torsional moments.

Clevis Weldment on Box Girder

The clevis weldment at the front of box-girder, indicated in Section A-A of Fig. 10 is 20 ft high, 15 ft wide and 10 ft long. At each of its sides protrude hollow, tapered ears of a 36 x 18 in. rectangular cross section to form the multiple clevises supporting the 12-in. diam pivot pins for the leaves. The pins are held between the clevises by large 4-in. thick collars welded to the ears as shown in Fig. 8. In order to avoid distortion of the machined
forged-steel pivot pins and any misalignment of the pin centers from distortion of the weldment, the weldment was furnace stress relieved prior to attachment of pin collars. To prevent distortion and to partially relieve any residual stress that might build up in the clevis ears as the pin collars were welded, a welding procedure was established using a special nickel-base electrode with a skip sequence. The high ductility of this weld material permitted its peening without building up brittleness in the weld. All weld passes were peened except for the first and last ones. To insure proper amount of peening, pairs of temporary nubs were mounted about 10 in. apart on the main material and as close to the final toe of the weld as possible to form a peening gage. After each peening operation, the distance between nubs was measured to check that the metal remained undistorted.

**Pickaback Stub Weldments on Box Girder**

As shown in Section E-E of Fig. 10, some space frame web members terminate in "pickaback" stub weldments surmounting the box girder. These weldments are field connected by heavy fillet welds at their base to sole plates on top of the box girder. The sole plates are riveted to the girder, the rivets being countersunk in the vicinity of the weldment base. The weldments were furnace stress relieved after fabrication. After field welding, a procedure similar to that previously described for the clevis weldment was used.
Tie Backs and Tie-back Anchorage Weldments

The two tie backs are each subjected to a maximum force of about 2100 tons when the roof is open and the leaves nested. A stress of comparable magnitude occurs with unbalanced snow load when the roof is closed. These 3 x 3½-ft box members are connected to tee flanges flaring out from the front end of the box girder, as shown in Section C-C of Fig. 10. As the tie backs diverge, they are supported, in turn, by cross girders, such as shown in Section D-D and by triangular frames as shown in Section E-E. As indicated in Fig. 16, each tie back terminates at its base in a tie-back anchorage weldment which engages the reinforced concrete tie-back anchorage by means of high-strength anchor bolts, a shear key and dowels. The 3 x 2½-in. key is heavily butt welded to the underside of the 2½-in. thick base plate of the anchor weldment. It is machined to engage a pair of 2-in. high machined stop bars welded to the upper side of the tie-anchorage shear plate. This shear plate, furnished with shear clips welded to its underside, is embedded in the reinforced concrete anchorage together with the lower 16 ft of the 21 ft long 2½-in. diam high-strength anchor bolts. The bolts are coated heavily with bituminous material to prevent bonding to concrete. When the space frame was finally adjusted, the approximate 3-in. space between weldment and shear plate was grouted and the anchor bolts prestressed to 125 tons per bolt, an amount beyond the expected maximum tension. The prestressing minimizes tension stresses in the anchorage, prevents damage to the grouted surface from "working" of the joint under load, and prevents the long anchor bolts from participating in the cantilever frame deflection.

Features of Weldments and Welds for Cantilever Frame

General

In order to illustrate the approach used in the laying out and designing of stub weldments, details of the stub weldment at point S2 are shown in Figs. 17, 18 and 19. Seven space frame members converge at this point. Wherever possible, a diaphragm is so located that its edge backs up an abutting face of a stub as shown in Fig. 17. Where such a diaphragm cannot be introduced, a grid is built up within the stub to back up the abutting face as shown in Section E-E of Fig. 19, such a design taking into account the "hard-points" at the outer non-yielding faces. Where an abutting face meets a surface near a reinforced line in that surface, such as near to the corner of a box or near to a diaphragm edge, then the face is so bent as to meet that corner or edge, and the bend line is properly reinforced with a plate or diaphragm. Some of the areas of the stub elements are made approximately 10% greater than that of the areas of the connecting members because of stress considerations.

All material for the weldments is ASTM A373 steel. Except for the welding of the pin collars to the clevis-weldment and of "pickaback" stub weldments to the box girder as previously described, all welding was performed in the shop prior to stress relieving, either manually with low-hydrogen AWS ASTM A233 grade E7018 electrodes or with semiautomatic submerged-arc welding. Although
the dimensions for the groove welds were shaped to meet the requirements of manual welding, the great majority of welds were performed with semiautomatic submerged-arc equipment; only where position or access prevented the submerged-arc’s application were manual welds used.

The junction of one stub to another is made by a variety of groove weld types. Some typical welds used in weldment S2 are shown in Fig. 20. A stub's cross-sectional area is somewhat greater than that of the corresponding space frame member in order to meet the requirements of stress combination at the junction with another stub. Where such stress conditions are unusually severe, the stub is increased in thickness near the junction as shown in Fig. 20 (j).

Junctions at highly-acute angles between plates are made by groove welding to machined junction pieces as shown in Fig. 20 (c). Other junctions at less severe angles are welded in manners shown in Fig. 20 (d), (e) and (f). They utilize flat or machined backing strips where necessary. In some cases after the welds are tested, bulkhead plates are welded on so as to seal off the inaccessible portions of the converging surfaces from atmospheric corrosion. This bulkheading is applied to all pockets and corners in the weldments which are difficult to reach. However, as much surface is left accessible for inspection as possible.

The type of connection considered most critical was the double-tee joint which transmitted tension from the edge of one plate transversely through a second plate into the edge of a third. To insure that the second plate had no laminations which would prevent proper transmission of the stress, this plate was carefully examined ultrasonically before fabrication. Welds to each face are groove welds. In addition to having the outer face of weld flared about 1/4 in., these welds are provided with fillet weld reinforcement on each side to eliminate the deep stress raisers that exist at any sharp re-entrant corner. Those groove welds, which were accessible for welding at one side only but which require development of full thickness of material, utilize a grooved backing plate to effect this reinforcement such as shown in Fig. 20 (g), (h), and (i).

Accessibility to the interior of the weldments and box members for maintenance is afforded by manholes and ladders located throughout the space frame. Manholes have sealed covers to reduce to a minimum the exposure of interior surfaces to atmospheric corrosion.

Loadings

Each particular point of a weldment is designed for that set of concurrently acting forces in its stubs which yields the worst stress combination. The set of forces is obtained by combination of the loading conditions shown in Fig. 21. The combination of loading causing the maximum stress usually is one of the following three:
1. Dead load with the leaves nested plus live load on the two top leaves.
2. Dead load with leaves nested, plus live load on top leaf.
3. Dead load with roof closed plus snow load on all leaves on one side of the box girder.

In no case does the effect of wind exceed that of live load.

Permissible Unit Stresses

In general, stresses and details of design conform to the requirements of the American Institute of Steel Construction specifications for the design, fabrication and erection of structural steel for buildings.

A maximum value of 20,000 psi was taken as the permissible unit stress for the A373 material used in the weldments, the unit stresses being calculated by application of the octahedral shear stress theory as a criterion of initial yielding. This theory is used in place of other available ones, since the nature of the combination of stresses in the weldments is such that strain energy provides a more reliable criterion of yield strength. Where \( \sigma \) is the unit tensile or compressive stress on a face and \( T \) is the unit shear stress on the faces of an element, the combined effective unit stress, \( \sigma_e \), may be computed as follows:

\[
\sigma_e = \sqrt{\left(\sigma_x\right)^2 + \left(\sigma_y\right)^2 + \left(\sigma_z\right)^2 - \sigma_x \sigma_y - \sigma_y \sigma_z - \sigma_z \sigma_x - \frac{3(T_{xy})^2}{\sigma_x \sigma_y} - \frac{3(T_{yz})^2}{\sigma_y \sigma_z} - \frac{3(T_{zx})^2}{\sigma_z \sigma_x}}
\]

<table>
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<th>ROOF OPEN (LEAVES NESTED)</th>
<th>ROOF CLOSED</th>
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</thead>
<tbody>
<tr>
<td>DEAD LOADS</td>
<td></td>
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<tr>
<td>LIVE LOADS</td>
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<td>0.5 PSF on top leaves</td>
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<tr>
<td>30 PSF on top leaves</td>
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<tr>
<td>20 PSF</td>
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</tbody>
</table>

**Fig. 21**—Loading conditions on roof system.
For the majority of the loading conditions which control the design, the stresses are in two planes only, eliminating five of the nine terms in this formula with a concurrent pair of terms such as $\pm \sigma_x$ and $\pm \sigma_y$, the $-\sigma_x\sigma_y$ term becomes additive. The large contribution of shear to the resultant stress should also be noted in this formula. Had the principal stress theory been used, the calculated critical stresses would have been somewhat smaller.

**Stress Analysis**

In order to demonstrate the approach used in designing the stub weldments, an analysis for part of the weldment at panel point S2 is shown in Fig. 22. Element “K,” one critical point in a section, Section 4-4, is analyzed for the stress calculated from a set of concurrent loads. Referring to Fig. 22, using the area and the moment of inertia of the trial section, the transverse and normal components of the stress in members S;U; and S;L; are applied to the section to obtain the unit shear stress and the varying unit stress diagram of tension and compression. Various points in the trial sections are then analyzed for maximum stress. Such an analysis is outlined here, where the shear and compression contributed by S;U; and S;L; combine with the tension in S;S, to give a combined stress of 20,000 psi, based on the previously described octahedral stress formula.

**Welding Procedure**

**General**

The project specifications required that all provisions of the American Welding Society Specifications for Welded Highway and Railway Bridges be observed, including qualification of welding procedures and welders where the welding procedure departed from the standard. It was necessary to make special procedure tests for certain connections, such as the one utilizing a machined junction piece to join intersecting plates—Fig. 20 (e). As a prototype for the qualification test, a test weldment composed of a machined junction piece and short lengths of plate was used. Examination of the test weldment was made by both the ultrasonic method, as a control on this manner of testing, and visually on polished and macroetched transverse sections. The effectiveness of the one-sided buttl joint design shown in Fig. 20 (g), (h) and (j) was evaluated by the fabrication of a qualification procedure weld which was examined in the same manner.

Qualification tests were made for the unconventional groove joints such as the one sided welds shown in Fig. 20 (h), using both the manual and semiautomatic submerged-arc welding processes. Each weld was made in duplicate—one unpreheated, the other preheated to 250°F—and were cut to obtain 0.050-in. diam all-weld-metal specimens and Charpy V-notch specimens for testing of unaffected base metal, of base metal in the heat-affected zone and of weld metal. The Charpy specimens were tested at various levels between room temperature and $-10^\circ$ F.

A procedure qualification test was made to evaluate the suitability of the use of the special nickel-base electrodes intended for field connections since there is no specification for their use. The purpose of this test was to verify that the strength and ductility were not impaired by the peening operation. At the same time these tests provided an opportunity for qualifying welders and instructing in the proper use of the peening gage. This strain measurement gage was previously described in the description of the clevis weldment. Additional tests were made in accordance with AWS Specifications to prove the shear resistance of these connections.

**Stress Relief**

All weldments were furnace stress relieved after fabrication and ultrasonic testing of welds were completed. Except for the previously described peened high-nickel welds and for four repair welds, no other welding was performed after stress relieving.

A special test was made to determine the lowest temperature-time combination for A-573 steel at which the yield strength is depressed to not more than 5000 psi. This test showed that 1100°F for 3 hr per in. of thickness satisfied the above requirement. When the weldments were placed in the heat-treating furnace, all legs were suitably braced and propped to prevent distortion. Thermocouples were attached to the thinnest and thickest members to permit heat control and avoid the development of temperature differentials in excess of 150°F between pieces of different thickness; maximum heating and cooling rates were maintained at 200°F per hr unless reduced by those temperature differential requirements. The weldments were removed from the furnace at 600°F. Heating rate, holding time and cooling rate were controlled by use of multipoint control instruments.

After completion of stress relieving, the weldments were retested ultrasonically. Machining was then performed on any surface requiring this operation, such as on large flat bases of pickaback weldments.

**Inspection Prior to Fabrication**

Inspection prior to welding utilized ultrasonic testing to check the plate material for laminations at those areas where tension was to be transmitted across plate. Areas indicated by this test to be nonhomogeneous were outlined on the plates by colored markers. Unless these defects fell within areas of the plate which were to be scrapped, the entire plate was rejected.

Flame-cut plate edges were inspected visually for laminations. When warranted, dye-penetrant tests were made to verify visual findings.

Assemblies tack for welding were inspected for compliance in regard to the details of the joint design, over-all dimensions and the cleanliness of the joint faces.
Inspection During Fabrication

Welding currents for both semiautomatic and manual welding were set to the qualification test values, using portable clamp-on type ammeters. Tolerances for all operations involved in the fabrication of these weldments were standardized.

Inspection was concerned with insuring uniform and correct thicknesses of weld passes in manual welding and uniform rates of progress in semiautomatic welding, with avoiding undercutting, with the application of full thickness of backing passes and with attaining the proper contour and specified dimensions of all welds.

All heavy joints were preheated to at least hand warm prior to the deposition of the first root pass. In double-vee joints the root pass was gouged out and inspected visually for complete removal of the unfused metal. When effectiveness of the gouging operation was questioned, dye-penetrant tests were made to ascertain the soundness of the root.

Low-hydrogen electrodes were taken in small quantities from heating ovens where they were kept at 400°F. Similar precautions were taken with flux used in the semiautomatic welding.

Inspection of Completed Weldments

All welds were examined by either radiographic or ultrasonic tests. All the butt-welded 30 in. WF beams of the roof leaf ribs were examined radiographically. The welds of the space frame weldments, most of which are complex, do not lend themselves to radiographic inspection, however, since in most cases the radiographs would have to be interpreted for passing through two or more separate welds and through a material of varying thickness. On the other hand, the ultrasonic method, after appropriate calibration for use in complex welds, permits instant discovery of a local change in character of a weld, indicating a flaw in that area. The areas indicated to be defective were examined more closely by visual inspection, dye penetrant, magnetic particle or radiography. It is to be noted that in ultrasonic testing, only those discontinuities in the material lying in a plane perpendicular or near perpendicular to the path of the ultrasonic beam produce a signal on the viewing screen, regardless of whether the discontinuity is a crack less than a thousandth of an inch or a small slag inclusion only a few thousandths of an inch thick. Accordingly, the

\[ \sigma = \sqrt{\sigma_x^2 + \sigma_y^2 + \sigma_z^2} \]

\[ = 20 \text{ K.S.I.} \]

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Fig. 22—Stress analysis
effectiveness of the testing method depends primarily on the beam orientation. The frequency of the ultrasonic beam determines its ability to penetrate the desired thickness of material and to overcome the obstacles of large grain size.

During ultrasonic testing, the transducer head is moved in a back and forth sweeping motion. In preparation for testing, the surface on the main materials to be traversed by the head was smoothed by use of a sander and covered by a film of oil. The rate of progression of testing averaged 18 ipm. Isolated defects in the material were identified by small, momentary signals. They were either examined further or were considered as negligible in much the same manner as would have been indications of such defects appearing on radiographs.

In all the welding operations, only four welds had to be removed and rewelded because of cracks occurring in the main material adjacent to the weld. These cracks did not appear to have been caused by any geometric configuration of weldment framing. The four defects occurred in two completed and inspected weldments which had been moved to an unheated area awaiting transfer to the heat-treating plant. During a storage period of two days there was a sharp temperature drop. In one weldment two cracks—each over 12 in. long—developed in the base metal, and these were attributed to thermal stresses. As disclosed by a macrograph of a transverse section removed from the mid-length, the crack originated from a flaw consisting of a fold in the base metal which was sufficiently fused to remain undetected by ultrasonic inspection. Propagation of the cracks was of a brittle fracture nature.

The two defects in the other weldment were internal cracks over 12 in. long. These were detected during the reapplication of ultrasonic testing following stress relieving. However, it was not determined whether they developed in the period between the application of the first ultrasonic test and the transfer to the heat-treating plant or during the heat-treating operation.

In repairing the four cracks, the defective area was removed by arc-air gouging and grinding. The completeness of crack removal was checked by dye-penetrant testing. The repair welding utilized the usual methods of interpass temperature control. Block sequence welding was used to prevent cracking. The two stress-relieved weldments were not stress relieved, since the location of the repaired areas was not regarded as critical.

Other Tests

In addition to ultrasonic testing, several other tests not specified by the AWS Specifications were made. One dealt with the calibration of the ultrasonic apparatus for indicating flaws occurring in acute angle, full penetration tee welds.

A test of two 75-ft long rib sections verified that the residual stresses resulting from the welding did not affect the rib geometry under the design loading and that stress relief heat treatment could be omitted for the 30-in. WF arch ribs shop splices.

Acknowledgment

The owner is the Public Auditorium Authority of Pittsburgh and Allegheny County. S. A. Swensenrud was Chairman from Oct. 29, 1954 through December 1959, after which William B. McFall was elected chairman. Mr. Edward Fraker is the Executive Director of the Authority. H. R. Helvenston is Resident Engineer and General Superintendent of Construction for the Authority.

The architects of the project are Mitchell and Ritchey, AIA, of Pittsburgh, Pa., with James C. Armstrong acting as project manager during the design phase and Edward R. Gallagher during the construction phase.

The engineering of the Pittsburgh Public Auditorium retractable roof and supporting structure, including structural, mechanical and electrical features, was done by Ammann & Whitney, Consulting Engineers. Boyd G. Anderson was the partner in charge of design. Werner Ammann was the partner in charge during the construction phase. The authors were Project Engineer and Assistant Project Engineer, respectively. A. Anderson, L. Au, F. Chang, H. Dandliker, M. Klein, A. Menanteaux, H. Rothman, H. Samelson, F. Serim and other members of the staff participated in the final design of the retractable roof.

Bela Ronay acted as Welding Consultant during fabrication and was responsible for the testing and inspection procedures of all welding operations. Mr. Helvenston supervised all testing and inspection.

The roof structure was detailed, fabricated and erected by American Bridge Division, United States Steel Corp.

E. C. Ernst, Inc., holds the electrical contract including roof drives and installed the power and control systems for the roof. Leaf rolling equipment was furnished by Heyl and Patterson. The drive, control and indicator systems were furnished by Westinghouse Electric Corp.

Limbach Co. fabricated and installed the stainless steel roofing and the weather seal closure system.

Dick Corp. performed all the reinforced concrete work on the project.

Photographs used in this article are courtesy of U. S. Steel Co., Heyl & Patterson and Lincoln Electric Co.