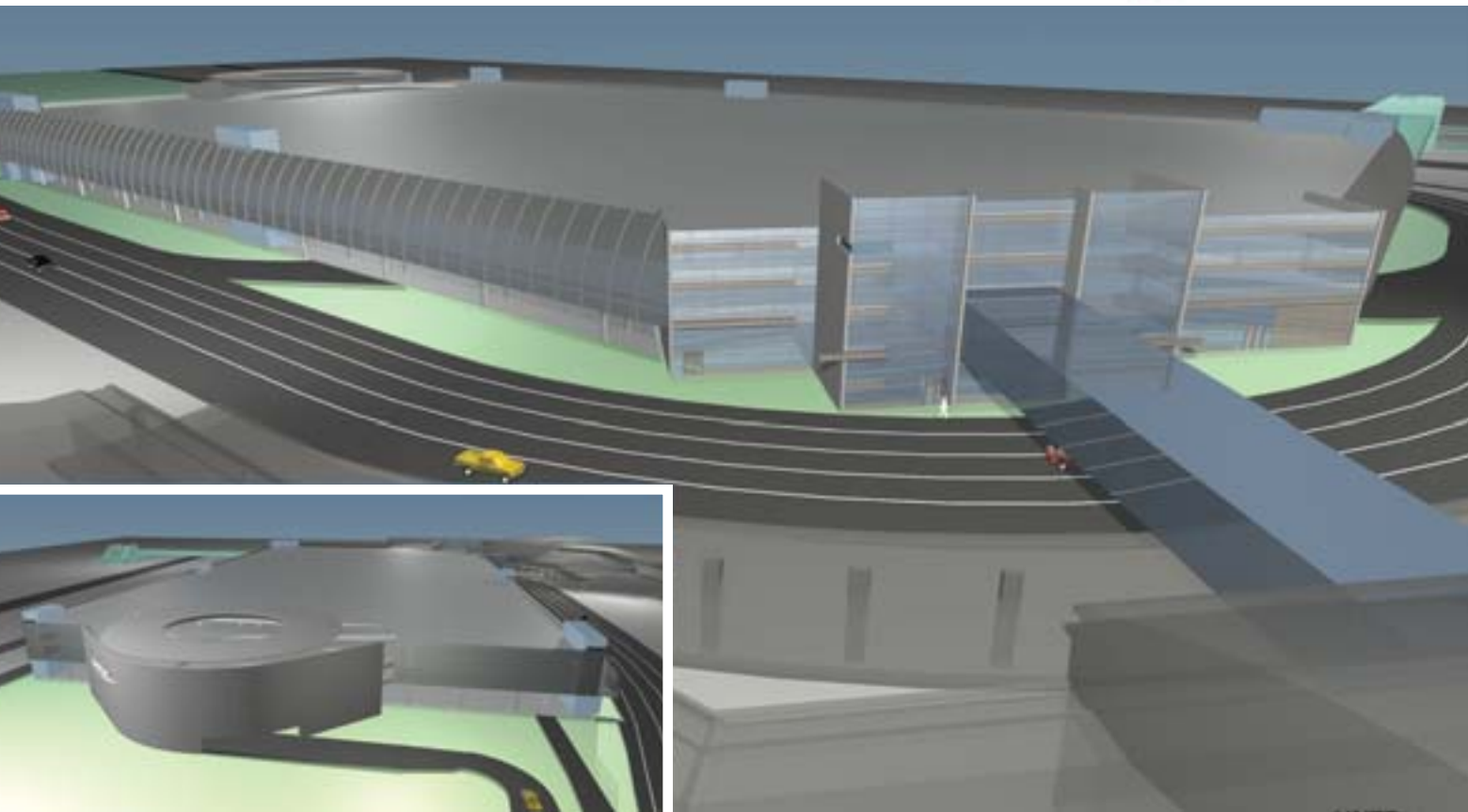# Park and Fly

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A 3,200-vehicle parking structure will anchor Harrisburg International Airport's new multi-modal transportation center in Middletown, PA.

Construction is underway on a $222-million facilities improvement program at Harrisburg International Airport (HIA) in Middletown, PA. By summer 2004, Susquehanna Area Regional Airport Authority (SARAA) expects to complete the first multi-modal transportation center in the eastern United States.

Twenty-five major contracts cover ramp and apron upgrades, a new 75' × 10,000' parallel taxiway with high-speed exits, a CAT-III Instrument Landing System and the elimination of some WW II-era structures on the airside of the 800-acre airport. Landside improvements include a 283,000-sq.-ft passenger terminal with eight multi-use gates, a double-platform Amtrak Station, roadway improvements, and a four-level parking garage and Ground Transportation Center (GTC) for 3,200 vehicles. The user-friendly design will connect the different structures with climate-controlled pedestrian bridges and moving sidewalks.

Balanced design and materials specifications should instill a 50-year service life in the 1.1-million-sq.-ft. garage facility. All design, materials and operational factors for the $50-million facility were subjected to cost-benefit, life-cycle and value-engineering analyses before choosing a a steel/concrete combination structure with pile foundations, precast double tee decks and structural steel framing. Vehicles will access and egress the public parking decks on the three upper floors from a double-threaded, 143'-0"-diameter helix designed as cast-in-place construction.

The ground level will hold rental car operations and serve bus traffic. Three traffic lanes will run at a 35-degree angle off the structure's southeast axis. The bus traffic through this level will establish an 18'-2" floor-to-floor height from top of slab on grade to the base of the first deck. The 12'-2" floor-to-floor height of the three upper

levels is set to accommodate large passenger vehicles like SUVs and vans.

DESIGN IN STEEL

Parking structures share many of the same design considerations as multi-story buildings and bridges. They are exposed to a variety of loads, seismic and environmental impacts, daily and seasonal temperature swings, and high-traffic use. But two factors are the most likely to erode the long-term integrity of a parking facility: moisture and chloride used to combat icy road conditions.

The HIA parking facility is designed with a number of safeguards to combat corrosion, including hot-dip galvanized steel framing, bolted instead of welded connections to preserve the uniformity of galvanized coating specified for the shop-fabricated members, and a $1\frac{1}{2}$-degree slope across the decks that ensures positive drainage.

STEEL FRAMING

Several considerations favor steel framing over structural precast or reinforced concrete construction.

First, the aesthetic goals for the 720' (E/W) by 396' (N/S) structure favored steel. A translucent skin was proposed for the structure, and through design development a stainless-steel woven fabric was selected to cover the moving walkways along the east and south face of level 2. The woven skin will curl up and over the conveyances from level 2 to level 4. The moving walkways will be carried by W27×217 (Grade 50) beams that cantilever through the W14×90 (Grade 50) perimeter columns.

Galvanized steel moment frames will complement the appearance of the stainless steel skin and deliver the desired skeletal look. From a functional standpoint, moment frames will offer efficient resistance to lateral loads and negate the need for shear walls or braced framing. This also delivers flexibility in managing the first floor rental car space and will help with the constructability of the curved façade enclosure for the moving walkways.

The framing will utilize approximately 2,475 tons of galvanized ASTM A992 (Grade 50) beams and columns, and 166 tons of galvanized ASTM A500 (Grade 46) tubes. Approximately 53,000 sq. ft of woven stainless steel fabric also applies to the structure.

The driving surface will consist of 12'-wide by 34"-deep precast double tees carried by W24×176 support beams. HNTB engineered the precast double tees for a 3,000-lb. wheel load versus the customary 2,000-lb. wheel load, to accept heavier SUVs now entering the marketplace. For corrosion protection, the precast double tees will incorporate a .38 water/cement ratio, corrosion inhibitors and a $1\frac{1}{2}$-degree slope for positive drainage. The plan translates into 34' by 60' bays, with steel framing spanning the 34' N/S bays and the double tees establishing the 60' E/W bays. The N/S moment frames will be spaced 120' apart at two intermediate grid lines for each segment. The E/W frames will be located near the north and south perimeter to preserve flexibility on the ground floor.

COLUMN CONNECTIONS

A column-tree configuration will apply to the 504 moment connections. Full penetration welds will attach 3'-long
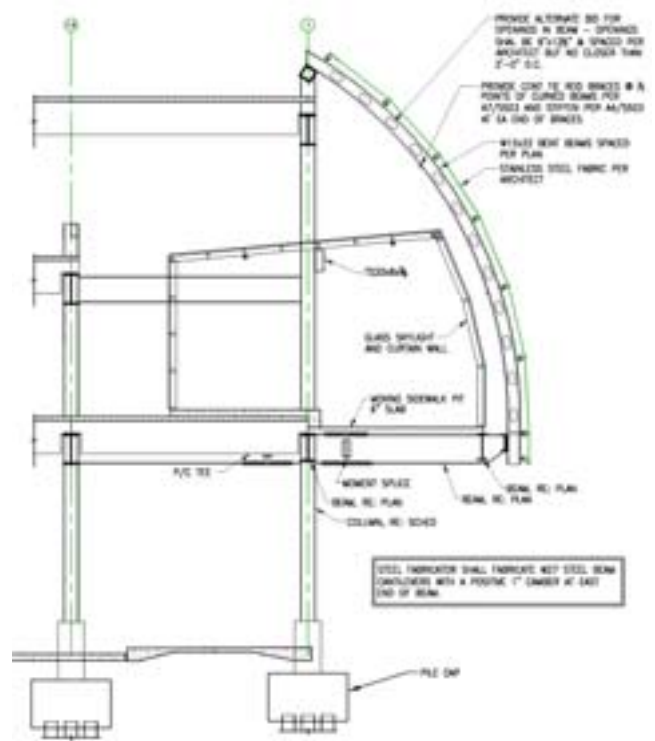
The moving sidewalk will be supported on cantilevered steel framing along the east face of the garage.
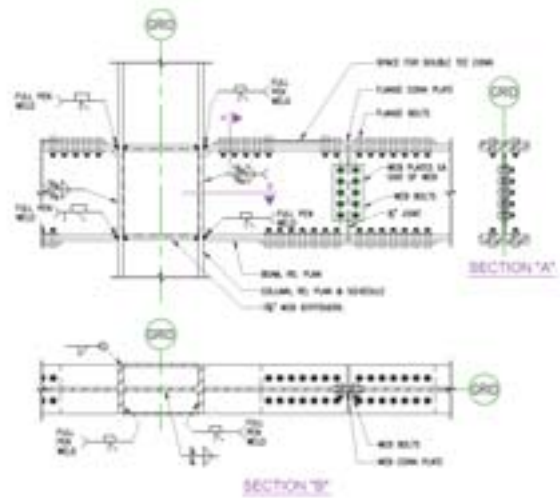
This detail shows the moment connection for one side of the typical column tree. The beam stub is shop-welded to the column to allow a fully field-bolted connection after the assembly has been hot-dip galvanized.

beam stubs to each side of the typical W21×166 columns. The column-tree branches, to be fabricated from the same steel beam section sizes, will be connected to the beams that span between each column tree with bolted connections. Heavily loaded simple shear connections will be created using bolted double angles and bolted shear tab connections. A 5.0 mil hot-dip galvanized coating has been specified to protect all steel from corrosion, using ASTM 123, ASTM A143-01 and ASTM 780 standards. Durability analysis favors bolted connections versus field welding both for speed of construction and to preserve a uniform hot-dip galvanized coating rather than using zinc-rich paint on welded field connections.

Going Galvanized

HNTB adopted a corrosion protection system that protected the client's first cost and should deliver a minimum design service life of 50 years (to 5 percent surface rust). The drainage design slope of 1.5 percent for the precast double tee floor system should minimize or eliminate any standing water. The steel coating system was equally essential to gaining an adequate service life. Three primary coating systems were evaluated for the structural steel: paint, galvanizing and a combination. A hot-dipped galvanized (HDG) coating system was selected because it should provide a maintenance-free service life of almost twice that of a paint system. For more on galvanizing, see "Taking a Dip," pages 55-59.

The American Galvanizers Association (AGA) provides a chart to use in estimating the service life of galvanized coatings based on the corrosion rate of zinc in various environments. The chart can be found on the AGA website at www.galvanizeit.org.

The documentation accompanying this chart listed Harrisburg, PA as having a "Suburban Environment," but HNTB rated it an "Industrial Environment" due to the likely exposure to high concentrations of road salts and carbon dioxide associated with a large garage. It was concluded by averaging the two environments that a 5-mil galvanizing thickness would provide the required service life of 50 years.

It is important to specify the following requirements so that a quality galvanizing product is provided:
- Steel is prepared in successive chemical baths of caustic cleaning, pickling and flux,
- A minimum mill thickness of 5.0 per ASTM A123,
- Inspection for brittleness per ASTM A 143-01, and
- Repairs made per ASTM A780 using the method of soldering.

The precast double tee stem-to-steel beam connections for the double tees will be stainless steel bearing plates welded to embedded stainless steel plates. The bearing plates that will bolt to the steel beams have pre-adhered bearing pads to protect the galvanized coating. The ¾"-diameter A325 bolts connecting the bearing plate to the supporting beam will be spread 15" apart to provide flexible connections that will allow the bearing ends of the precast double tees to rotate, and will reduce induced loads from creep, shrinkage and thermally-induced movement.

MOMENT FRAMES

The column-tree configuration offers the additional ability to dissipate energy in a full design seismic event. Engineers based the design of the column-tree moment frames on procedures in "Seismic Design of Steel Column-Tree Moment-Resisting Frames," published by the Structural Steel Educational Council (SSEC). An additional reference was AISC's *Seismic Provisions for Structural Steel Buildings*.

The bolted connections are designed as slip-critical under service loads and will slip under a design seismic event. The slip in the connections under a design seismic event provides added ductility in the moment frames. The added ductility categorizes the frames as special steel moment frames with $R = 8$ per the 1996 BOCA code and reduces the required lateral seismic load resistance.

The magnitude of the service moment at the bolted connection will be significantly less than the maximum at the column face, because the center of the splice is 4'-0" from the center of the column. The beam-column joints of the moment frames required significantly more engineering time. HNTB used spreadsheets to complete the repetitive calculations. The only additional detailing addressed the ½"-thick web double plates added to the beam-column panel zone for about 60 percent of the project's moment connections. However, the cost of these additional reinforcing plates has been offset by the reduced frame loading.

The following two special load cases are specified in *Seismic Provisions for Structural Steel Buildings* and apply to strengthening the panel zone:

$$U = 1.2\,DL + 1.0\,L + .2\,S + \Omega_0 Q_E \quad \text{(Eq. 4-1)}$$
$$U = .9\,DL + \Omega_0 Q_E \quad \text{(Eq. 4-2)}$$

A value of 3.0 was used for the parameter Ω_0 and is required for special steel moment frames, and the parameter QE is applied to the horizontal portion of the seismic load. These load cases were required for special circumstances noted in the publication, where the code's prescribed seismic loads were not applicable in the assumed "over-strength" of the structural system. The steel code requires these load cases for panel zones since the area of the structure does not demonstrate the over-strength capacity of the structure as a whole.

EXPANSION JOINT SPECS

Two N/S expansion joints will divide the structure into three segments. In the early stages of design development, there was concern that the flexible moment frames would increase the cost of these joints. After extensive analysis, the HNTB design team determined it would be more cost effective after a design earthquake to simply replace more conventional expansion joints than to incur the high cost of installing seismic expansion joints during initial construction. Therefore, the joints were designed only for combined service loads (wind + creep + shrinkage + thermal). Harrisburg, PA is in a moderate seismic zone where the A_v has a magnitude of 0.085, or a Category B Seismic Performance. The site classification was determined as S1 by the geotechnical engineer and C_s on the order of 0.013.

Because the BOCA code lacks specifics on how to calculate separation between two structures using amplified drift, HNTB calculated the project's required separation using the square root of the sum of the squares method from the International Building Code. To reduce the calculated drift, each section was modeled in RISA-3D and the period determined for each section of the structure in each orthogonal direction. BOCA, as well as other codes, does not require the structure period to be limited to $C_a \times T_a$ when calculating amplified drift. The higher period was calculated and used for this project to reduce the calculated seismic base shear and associated drift

and thereby arrive at an appropriate structure separation.

A review of published data established the maximum design width for the largest standard-winged, compression-type expansion joint. An engineer can determine the allowable opening width between structure segments for a particular expansion joint by subtracting from the maximum expansion-joint opening width the amount of drift associated with wind, creep, shrinkage and temperature forces. The allowable elastic drift for each of the building segments can be found using the following formula:

$$\text{Allowable Elastic Drift} = (\text{Building Separation} /1.41) / C_d$$

Frames were analyzed and frame member sizes adjusted to alter the stiffness until the actual calculated drift at each floor was less than the allowable elastic drift. Period and seismic loads were recalculated for the increased stiffness in an interactive process until achieving convergence. The process revealed that steel beam and column sections of the E/W frames of the center and west segments of the building could be slightly enlarged to gain the adequate stiffness. The asymmetrical layout of the east segment, however, made stiffening frames uneconomical. A seismic expansion joint was therefore specified for the top level of the garage between the center and east segments.

This project's steel/concrete combination design satisfied the owner's preference for a steel-framed facility and played to major aesthetic elements of the architectural design. In this particular instance, steel also compared favorably in price against precast and cast-in-place alternatives during design development. This project presents another example of how steel can offer a multitude of advantages without penalizing a project's budget. ★

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