CIVIL DESIGN

Design and Construction of the Weehawken Tunnel and Bergenline Avenue Station for the Hudson–Bergen Light Rail Transit System

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The existing 9.5 mi (15.2 km) long Hudson–Bergen Light Rail Transit System in Northern New Jersey is being expanded. This 6-mi (9.5-km) expansion includes 4,100 ft (1,250 m) through an existing railroad tunnel. The Weehawken Tunnel was built in 1881-1883 by the New York West Shore and Buffalo Railroad (NYWS&B) to provide a rail connection from west of the Palisades Ridge to the Hudson River Waterfront in Weehawken, N.J.

The engineering task was to provide an LRT alignment within the rock tunnel along with a mid-tunnel LRT station. The station is located approximately 160 ft (49 m) beneath the cities of Weehawken, West New York, Union City, and North Bergen Township. The existing tunnel needed to be enlarged by blasting methods to accommodate the two-track light rail trackway and station platform.

This paper discusses the building of the original tunnel; the engineering design relative to the new tunnel configuration and mid-tunnel station; and the construction work done to date.

INTRODUCTION

The Hudson–Bergen Light Rail Transit System (HBLR) Project is a 30-mi (48.1-km) and 37-station light rail system in the state of New Jersey (Figure 1). The project is located in the Hudson–Bergen transportation corridor, which is a vital artery to the economic and social well being of New Jersey and the adjacent New York metropolitan area. The project is being built in phases. The 9.5-mi (15.2-km) Minimum Operable Segment (MOS)-1 is in operation. The 6-mi (9.6-km) MOS-2 is currently under construction. The 13-mi (20.8-km) MOS-3 is being planned at this time. The light rail transit (LRT) alignment runs along the westside waterfront of the Hudson River, overlooking Manhattan, New York, from the southern tip of Bayonne to Bergen County. The LRT alignment serves surrounding communities, which include Bayonne, Jersey City, Hoboken, Weehawken, Union City, West New York, North Bergen Township, and the southern communities of Bergen County.

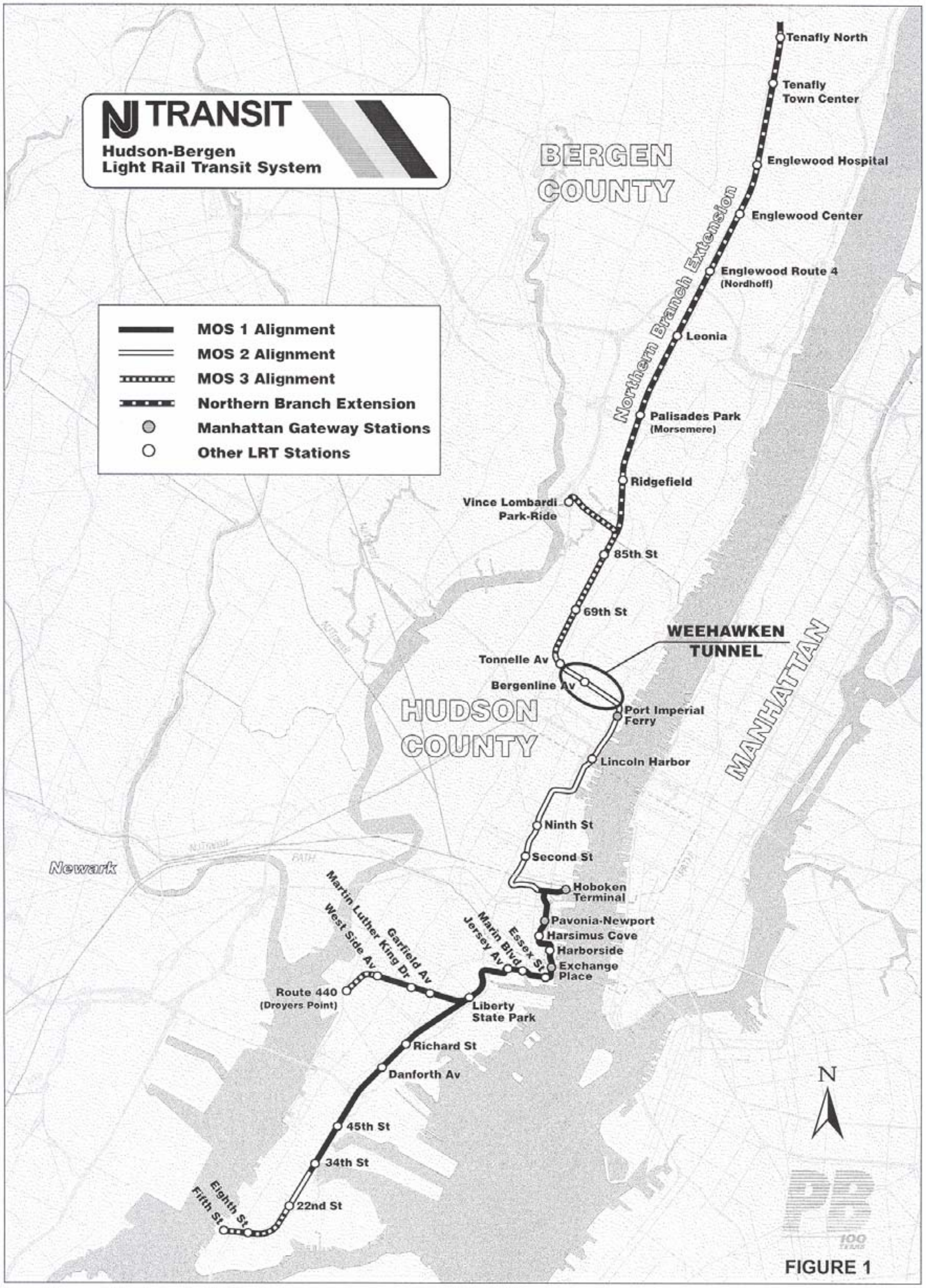

FIGURE 1 The Hudson–Bergen Light Rail Transit System.

A portion of the MOS-2 runs through an old 4,100 ft (1,250-m) long freight rail tunnel in Weehawken. The Weehawken Tunnel was originally built by the New York West Shore and Buffalo Railroad (NYWS&B) from 1881-1883 to provide a rail connection from west of the Palisades to the Hudson River waterfront where the NYWS&B had constructed the Weehawken Terminal, a freight and passenger terminal. It was designed and constructed under the supervision of NYWS&B Chief Engineer Walter Katte. The tunnel passes through the Palisades Sill, a steep prominent east-facing ridge along the west shore of the Hudson River.

Tunnel Construction

The original Weehawken Tunnel extended 4,014 ft (1,223 m) from its western portal in North Bergen Township, Hudson County, New Jersey, to its eastern portal near the Hudson River in the township of Weehawken, Hudson County, New Jersey (Figure 2). The tunnel extends beneath 48th Street in Weehawken, Union City, and West New York.

The original tunnel had a semi-elliptical arched roof with nearly vertical walls and was constructed for two tracks with a 13-ft (4-m) centerline distance. The majority of the tunnel was constructed in the rock of the Palisades Diabase. For approximately three-quarters of its length, the tunnel was unlined and the natural rock exposed. Eight sections of the tunnel, varying in length from 18 ft (55 m) to 357 ft (109 m), were lined with brick masonry. Excavation was conducted simultaneously from each end of the tunnel and in both directions from five construction shafts. Explosives were used to loosen the rock and workers loaded the debris onto rail cars, which were then hoisted to the surface through shafts, and removed by locomotive via a temporary rail line. An average of 450 men, working two shifts of 12 h, completed the tunnel in 2 years. The size of the existing tunnel was 27 ft (8.2 m) wide by 21 ft (6.4 m) high, or 19 ft (5.8 m) from the top of rail to the center of the roof arch.

Weehawken Terminal

The NYWS&B had grand designs for Weehawken Terminal, however, only the passenger terminal was completed, and one or two of the large piers originally planned were actually built. By 1884, the New York Ontario and Western Railway, partner of the NYWS&B, had declared bankruptcy, burdening the NYWS&B's already strained finances. In 1885, following the intervention of J. Pierpont Morgan, the NYWS&B was sold at foreclosure to the New York Central Railroad (NYC). It was immediately re-organized as the West Shore Railroad Company and became known as the West Shore or River Division. Within a few years, the NYC completed the Weehawken Freight Terminal complex according to original NYWS&B plans. This included: a dozen piers, two grain elevators, passenger, ferry, and freight terminals, locomotive roundhouse and turntable, railroad and marine repair shops, and an icehouse, all serving 12 freight piers that occupied over a mile of Hudson River waterfront. With its completion, the Weehawken Terminal became the NYC's major freight export facility in New York harbor.

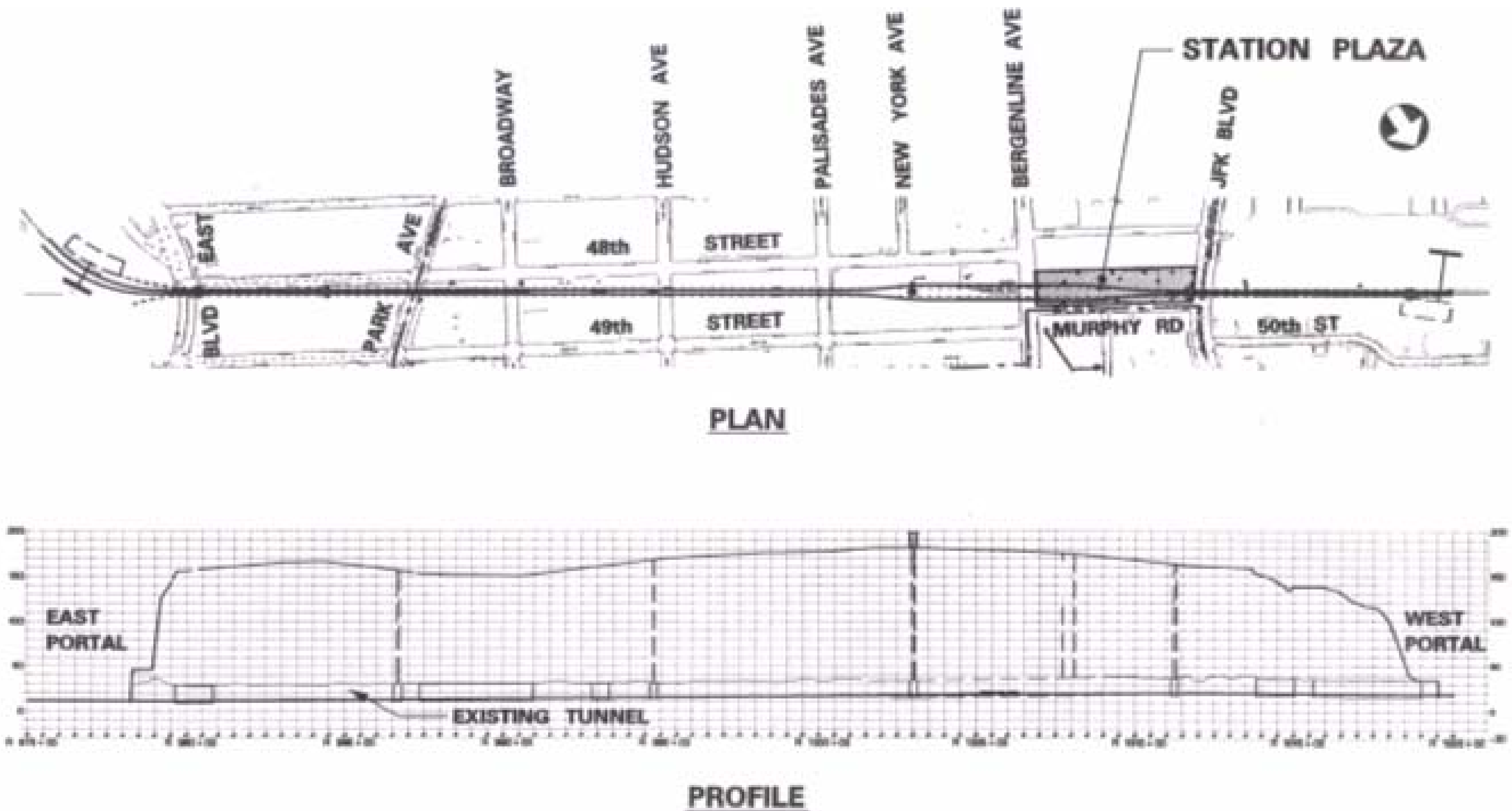

FIGURE 2 The original Weehawken Tunnel extended 4,014 ft (1,223 m) from its western portal in North Bergen Township, Hudson County, New Jersey, to its eastern portal near the Hudson River in the township of Weehawken, Hudson County, New Jersey.

West Shore Railroad

The West Shore Railroad promoted suburban development along its route, and farmland was transformed into middle-class housing developments. However, rider ship declined almost immediately following the opening of the George Washington Bridge in 1931. Then in 1937, the Lincoln Tunnel provided a second automobile route almost parallel to the West Shore's Weehawken 42nd Street ferry crossing. Finally, the Tappan Zee Bridge and the New York State Thruway were constructed in the 1950s, and by 1959 all passenger service on the West Shore Railroad was abandoned.

Public Service Electric & Gas Company

In late 1980s Public Service Electric & Gas (PSE&G) constructed two high voltage transmission power duct banks (230 kV) at the tunnel invert on either side of the railroad track, hugging the tunnel walls.

The Tunnel's Rebirth

In the 1980s, NJ Transit planned to implement the northern portion of the HBLR utilizing the tunnel. Working with its design consultant, Parsons Brinckerhoff Quade & Douglas, Inc., (PB) and the railroads, it was agreed that freight service could be shifted west to the Conrail Northern Branch Line to allow for construction of NJ Transit's HBLR Transit System.

The portion of the HBLR through the Weehawken Tunnel is known as Design Unit N30. The Design Unit includes approximately 4,300 ft (1,311 m) of LRT alignment and, a deep rock station cavern (Bergenline Avenue Station) and a 160-ft (49-m) vertical shaft for the connection with the surface bus station. The Bergenline Avenue Station will be one of the most frequently used stations on the entire system. During the peak commuter hour, a total of 1,900 persons will enter or leave the station. Contributing to this rider-ship is the large population within a walking distance, significant retail and businesses, and numerous bus routes.

PB developed the tunnel and station design for NJ Transit. A Joint Venture of Frontier-Kemper Constructors, Inc., J. F. Shea Construction, Beton and Monierbau Gesellschaft M.B.H. (FKSB) is the tunnel contractor under subcontract to the HBLR Design Built Operate Maintain (DBOM) Contractor, Twenty-First Century Rail Corporation (TFC). Construction started in Spring 2002 and is scheduled to be complete in Spring 2005.

ENGINEERING

The engineering task for Design Unit N30 consists of providing engineering design for a 4,300 ft (1,311 m) of LRT at-grade alignment of which 4,100 ft (1,250 m) is inside the existing Weehawken Tunnel, a mid-tunnel LRT station – Bergenline Avenue Station, and a NJ Transit bus plaza and station entrance at the surface located at the northwest corner of Bergenline Avenue and 49th Street.

The majority of the existing tunnel is unlined and the highly irregular rock surface is exposed. The linings are composed of brick arches and ashlar stone masonry sidewalls. Brick-lined zones are present either at rock zones of poor rock quality or at construction shafts. The construction shafts, spaced about 800 ft (244 m) on centers, are believed to have been sunk at five locations during the original tunnel construction. The existing brick linings conceal four out of five originally constructed shafts. One shaft remained open and it provided natural ventilation to the tunnel.

The geometrical requirements of the tunnel cross-sections were established taking into consideration the dynamic clearance envelope of the light rail vehicle; system and catenary clearances; construction tolerances; a maintenance walkway; presence of the existing utility duct banks and need for their uninterrupted service in all phases of construction; requirements of minimizing the excavation quantities; and the tunnel ventilation requirements. Different "modified horse-shoe" shaped cross-sections were identified for the unlined sections and fully lined sections of the tunnel.

ALIGNMENT

The LRT alignment for Design Unit N30 begins approximately 400 ft (120 m) south of the existing east portal structure of the Weehawken Tunnel. The alignment approaches the tunnel on a horizontal curve with a radius of approximately 350 ft (107 m) with a 33-mph (53 kmph) civil design speed. Track spacing reduces to 12 ft (3.7 m) on centers through this curve as the alignment enters the Tunnel. The cross section outside the Tunnel consists of Ballast Track with variable centers.

The cross section through the east tunnel segment, consists of direct fixation track, at 12 ft (3.7 m) centers, with a 36-in. (0.9 m) wide maintenance walkway located on the right (north) side of the track way. In the event of an emergency inside the tunnel the entire width between the rails of each track could be used as an emergency walkway, as it is a smooth surface of the direct fixation track work.

The second segment of the tunnel alignment includes the transition areas from running tunnel to the station, and the station area itself. As the alignment approaches the station from the east, the alignment transitions from 12-ft (3.7 m) track centers to the 49-ft (15 m) track centers required for the center platform station. The transition is attained through pairs of reverse curves meeting at a point of reverse spiral. The point of reverse spiral configuration was utilized to minimize the amount of rock excavation required for the station area.

At the west end of the station the track way centerline spacing transitions from 49 ft (15 m) to 12 ft (3.7 m) through pairs of reverse curves which are similar to those on the east side of the station. Direct fixation track construction is utilized throughout this area.

The final segment of tunnel alignment extends from the west transition area to the west tunnel portal. The cross section in this area is the same as those in the east segment of the tunnel. The N30 alignment ends approximately 50 ft (15.2 m) beyond the west portal.

BERGENLINE AVENUE STATION

Station Configuration

Bergenline Avenue Station is a center platform station located below grade with access at Bergenline Avenue and 49th Street. At the street level, a plaza is designed to provide a waiting area, a bus pick-up/drop-off, and an access to the station. A shaft 37 ft (11.3 m) in diameter provides access to the platform. The shaft contains three elevators, access stairs and the ventilation ducts.

At the bottom of the shaft, at platform level, is an elevator lobby. The elevator lobby was sized to accommodate peak level patronage. The elevators were sized to completely transfer the patronage of one peak period train from the platform level to the surface prior to arrival of the next peak period train. The elevator cabs have a capacity of 5,000 lb (2,268 kg) with a maximum load of 33 passengers. The cabs will run at 700 ft (213 m) per minute. The doors will be 4 ft, 6 in. wide, (1.4 m) allowing easy transfer of passengers on and off the elevators. With openings at both ends of the elevator cab, the cabs will load from one side and exit from the other, for peak passenger transfer efficiency. The elevator lobby at the platform level will be provided with closure roll-down “fire-shutter” doors to isolate the lobby area from either platform during an emergency.

The platform is 280 ft (85 m) long, to accommodate a train of three light rail vehicles. The width of the platform is 39 ft, 7-1/2 in. (12 m). It is divided longitudinally by a center firewall to isolate one side of the station in case of an emergency.

Large openings within the center fire wall equipped with horizontal sliding fire resistant pocket doors connect each trackway platform. In the event of an emergency, the sliding doors will be closed to isolate one half of the platform from heat and smoke allowing it to serve as a safe refuge area. Each pocket door is equipped with smaller spring loaded double swing doors to allow exiting occupants to continue to cross the separation wall after the pocket doors have closed. The resulting fire separation provides the platform occupants with a safe area to accommodate egress from the station. From the safe area, the occupants can access elevators and an exit stair for egress to ground surface, additionally, the occupants can walk out through the tunnel portals.

Two transition areas, one at each end of the platform, serve to house the tunnel emergency ventilation fans and their associated power supply. The platform ventilation fans and the associated power supply and ancillary facilities are located in the basement of the station facility at the surface.

Platform Level

The platform level consists of the station platform, a vestibule (lobby) area, and two transition areas at the ends of the station. Two transition areas, one at each end of the platform, house a total of four tunnel emergency ventilation fans. Two fans are located at each end within a two level fan plant. At either end of the fan plants are plenum areas. The plenum areas direct air flow during tunnel emergencies, by the use of by-pass dampers. The by-pass dampers are located in the plenum walls adjacent to the trackways. Based on the location of an emergency, by-pass dampers on one trackway would be opened, while the by-pass dampers to the opposite track would be closed.

Street Level

The station facilities at the street level include a plaza for waiting passengers, streetscape, bus bays for pick-up and drop-off, the shaft headhouse, the elevator machine room, the stair pressurization room and the elevator lobby. In addition, six ventilation stacks are provided for the tunnel and the platform ventilation. A separate two-story Utility building is located to the western part of the plaza at the corner of 49th Street and JFK Boulevard.

Plaza Basement

The plaza basement houses the station platform ventilation fans, and their motor control center, and the tunnel and the station ventilation ducts.

Elevator Headhouse

The elevator headhouse is a three-story structure. It houses the elevator machinery room and pressurization rooms.

Ventilation Stacks

Ventilation stacks are constructed of cast-in-place concrete walls and slabs with architectural brick walls at the outside faces. In conjunction with the ventilation stacks, emergency egress stair from the plaza basement level, employee bathroom and Emergency Aid Room are provided.

Utility Building:

The Utility Building is a two-story structure situated at the Southeast corner of the intersection of 49th Street and JFK Boulevard. A generator room, and electrical room, a mechanical room, a communication room, an Uninterrupted Power System (UPS) room, a battery storage, a Clean Air fire protection room, and a meter room are housed in the Utility Building.

Arts-In-Transit

The NJ Transit, Transit Arts Committee, set forth guidelines for aesthetics and prepared an HBLR Arts-in-Transit “Master Plan” which guided art related elements to be included in the design of the stations and station elements. The committee made every effort to incorporate the complete project including passenger stations, station elements, retaining walls, and new bridges.

The natural and cultural histories of the area provide the conceptual basis for the Bergenline Avenue art program. The Palisades are a landform with a rich geological history. It is a basaltic intrusion of Jurassic origin. When created, it lay well below the surface and only the erosion of the centuries and the ice ages exposed it in its present condition. For the first settlers in the area, it was a barrier preventing easy access to the west. With the opening of the Weehawken tunnel at the beginning of the century it became a conduit to the hinterlands, opening them for development. The communities that were established on the Palisades became home to successive waves of immigrant populations. Today, the Bergenline Avenue area is home to a wide variety of residents including Cuban, Caribbean, Latin, and South American communities.

The art opportunities for the station have been developed in three parts: the surface facility, the elevator shaft, and the station platform. The surface facility will convey the theme “The Community and the Rock” with the outcroppings of natural rock from the excavation establishing a physical connection to the Palisades. The elevator shaft will begin the theme of

"The Journey Through Time" and as passengers exit, the platform will present them with a sense of entering another dimension. The platform will express the past and future of our planet through the use of exposed natural rock and artist-designed elements.

UTILITIES

Owners

The major utility facilities within Design Unit N30 are PSE&G (underground and overhead electric lines); and Bell Atlantic/Verizon (overhead telephone lines).

All private utility companies including PSE&G and Bell Atlantic/Verizon will be relocating their own facilities. The LRT contractor in coordination will perform all other utility work in conjunction with the utility owner and local municipal authorities.

PSE&G Electric Lines

The widening of the tunnel required the relocation of two existing 230 kV line duct banks, which were located at the tunnel invert hugging the tunnel wall on either side of the existing Conrail tracks. Each of these existing 230 kV line consisted of a set of two oilostatic pipe-type cables installed inside an 8 in. steel pipe filled with a dielectric fluid under high pressure. In addition to the 8 in. (203 cm) steel pipe, each of these duct banks included one 5 in. (12.7 cm) polyvinyl chloride (PVC) conduit one of which carries a PSE&G associated fiber optic communication line and one empty conduit. These duct banks were constructed in mid 1980s directly on top of the then existing tunnel invert bedrock in order to minimize any rock excavation during the installation.

Several alternative schemes for the relocation of these PSE&G duct banks were considered, evaluated, and presented to PSE&G and NJ Transit officials in a series of joint meetings. It was finally agreed that each of the 230 kV lines will be relocated to new reinforced concrete duct banks with two 8-in. (0.8-m) steel conduits, one duct to carry one set of Oilostatic power cables (*3*) and the other will be a spare duct for future use in the event the cables fail in the operating duct while LRT is operational, and one 5-in. (12.7 cm) PVC conduit to accommodate fiber optic communication cable running along and under the proposed LRT tracks.

After the new PSE&G 230kV lines were operational, PSE&G drained the fluids from the abandoned existing 230kV circuits after removing the cables. Only after PSE&G certified that the existing abandoned ducts are environmentally safe to remove, the N30 Contractor began the demolition of these duct banks. The N30 Contractor coordinated the LRT related construction activities and the relocation of PSE&G 230kV circuits inside and outside the tunnel with the PSE&G transmission division. The N30 Contractor was responsible for developing the Construction Staging details of PSE&G circuits and LRT Construction Sequencing and obtain NJ Transit and PSE&G approvals before starting construction.

The N30 Contractor provided all necessary construction support to PSE&G in relocating their 230kV lines within the N30 design limits. This support included any excavation required at the tunnel invert and outside portals to accommodate the two 230 kV duct banks, installation of reinforcement and placing of concrete for the duct banks and the surrounding areas. PSE&G

built and tested the steel and PVC ducts needed for the relocation of their existing services in the trench provided by the N30 Contractor. All duct and cable installation, testing, tie-in and splicing to the existing cables at either ends of the 230kV circuits relocation out side of each portal, switching the services to the relocated facilities, and terminating the existing abandoned portion of the circuits was done by PSE&G's contractor.

The N30 Contractor will need to protect the 230kV circuit duct banks from all construction activities including from any debris from blasting activities.

On the surface, the Bergenline Avenue Station site development impacts some minor overhead facilities. Few poles and their associated cables require relocation.

STRUCTURAL

Tunnel

Two different tunnel cross sections were developed:

- Unlined section
- Fully lined section

A modified horseshoe configuration is used for all tunnel sections by closely matching the cross-sectional configuration of the existing tunnel in order to minimize the rock excavation.

The unlined tunnel section will be used where the rock is sound and there is no or little water infiltration. The fully lined tunnel section will be used in the bad rock conditions. The arch and the walls are lined with cast-in-place concrete.

The centerline of the tracks is offset from the centerline of the tunnel by 8 in. (20.3 cm) in order to minimize the rock excavation and limiting it, as much as possible, to the tunnel north wall.

The tunnel cross sections chosen provides an efficient and economical geometrical configuration that meet the structural and geometrical requirements yet minimize the rock excavation.

Tunnel Ground Water Control

The existing lined and unlined segments of the tunnel are subject to water infiltration and icing which is not acceptable for the operation of a transit system. Therefore, the design includes provisions for control of water infiltration.

New lined sections of the tunnel will be constructed with a waterproofing system. In the areas where full liner is anticipated, two 6 in. (15.2 cm) diameter perforated PVC pipes will be placed along the tunnel walls near the invert. Drainage fabric will be installed directly against smoothing shotcrete from the tunnel crown down the sidewalls and then wrapped around the perforated drainage pipes. A PVC waterproofing membrane will then be installed against the drainage fabric to reduce the potential for water infiltration past the fabric. Ground water will be collected at the pipes and discharged into the track drainage system.

The track drainage system consists of 10 in. (25.4 cm) or 8 in. (20.3 cm) diameter PVC drainage pipes, placed in the track slab. The pipes are sloped to drain into the drainage system outside the tunnel portals. For the ballasted track portion at the East Portal, the track drainage system consists of 10 in. (25 cm) diameter non-perforated PVC pipe, placed underneath the ballast between the two tracks, and connected to a manhole which is part of the outside drainage system beyond the portal limits

Station

The station cavern is 61 ft, 8 in. (18.8 m) wide by 32 ft, 6 in. (9.9 m) high. The structural system at the platform area consists of a 2 ft, 0 in. (0.6 m) thick concrete arch supported by 4 ft, 0 in. ((1.2 m) x 2 ft, 0 in. (0.6 m) pillars placed against the station cavern walls. The pillars are spaced 20 ft (6.1 m) on centers leaving the rock surface exposed. A center continuous footing and edge walls independently support the platform. The central wall is a 1 ft, 0 in. (0.3 m) thick self-supporting concrete wall. The upper connection of the center wall, as well as all other interior walls within the station caverns to the concrete liner, are designed so that no load from the liner is transferred to the interior walls.

Portal Structures

East Portal

The East Portal will be reconstructed utilizing a portion of the existing structure. Most of the existing portal structure has been demolished. The southern wall and a short northern wall will remain in place and will be incorporated in the support of the rock faces. New walls will be constructed of cast-in-place concrete directly in front of the existing walls and will be anchored into the rock using tiebacks. A new parapet wall and concrete ditches will be built on top of the new portal walls to collect the surface water run-off.

The new portal structure will be constructed of reinforced concrete. It will be waterproofed using the waterproofing system similar to the one described for the tunnel.

West Portal

The existing metal structures at the west portal have been completely demolished and a new concrete cut-and-cover structure will be built. The structure will be of a horseshoe shape and will extend about 50 ft (15.2 m) west from the original rock portal. West portal cut-and-cover section extends for another 45 ft (13.7 m) approximately, having the cross section that matches the section of the running tunnel.

Surface water run-off at the west portal will be channeled away from the tunnel by proper grading and by constructing an energy dissipating hydraulic structure.

Along the faces of both portals, above the overhead contact system, a 6-ft (1.8-m) high parapet wall is provided to comply with the catenary power safety requirements.

ENVIRONMENTAL CONTROL SYSTEMS

The Environmental Control System (ECS) includes the following major subsystems:

1. Tunnel Ventilation
2. Platform Ventilation
3. Stairwell Pressurization
4. Elevator Shaft Pressurization
5. Ancillary Space Heating and Ventilation (and cooling where required)
6. Comfort Heating and Air Conditioning

Functional Requirements

The ECS for the Bergenline Avenue Station and the adjacent tunnel sections is designed to provide ventilation during normal train operations and to support life safety operations during an emergency caused by a fire or a vehicle derailment.

For the Bergenline Avenue Station support facilities, the ECS is designed to control temperature and provide ventilation in electrical and mechanical rooms; and other ancillary spaces to support routine maintenance and prolong the equipment life by controlling the room temperature and humidity.

Tunnel Ventilation System

The tunnel ventilation system is designed to meet the emergency situation objectives. The primary relevant fire emergencies include fire and smoke on the station platforms, in tunnel sections or on board a train. In any of these situations the function of the tunnel ventilation system is to maintain a relatively safe path of egress in case it becomes necessary to evacuate the passengers. The platform and tunnel ventilation systems work in unison during an emergency. Together they will develop airflow patterns to assist in the emergency rescue and fire fighting personnel in gaining access to the fire scene.

For achieving these objectives, the platform and tunnel ventilation fans are operated such that they provide a source of fresh air into the evacuation path and keep smoke and heat away from the stranded passengers by exhausting air from the downstream side of the fire. Since the direction of emergency evacuation depends on the location of fire, both the tunnel and platform ventilation systems are designed with the capability of moving air in either direction.

The fire location, location of other vehicles in the tunnel, and passenger evacuation direction are the three factors that affect the required number of fans and their operating modes. The airflow capacities of the tunnel and platform ventilation systems were evaluated using the Subway Environment Simulation (SES) computer program. The SES program simulates various fire scenarios and calculates the critical air velocity required to prevent back layering of hot smoke.

Platform Ventilation System

Heat is introduced into an underground station from lights, people, electrical equipment and from the operation of trains. Mainly braking resistor grids, air conditioning condensers, friction brakes and losses from the traction power system generates train heat. This heat will dissipate into the surrounding area, and warm the tunnel and platform environment. The piston action of the moving trains will push some of the warm air from the tunnel environment, through by-pass dampers, to the atmosphere. In order to capture and exhaust some of the warm air from the station platforms, an overhead ventilation system is provided.

The results of the SES analysis for normal operations indicated that the heat sink effect of the surrounding earth and the piston action of the moving trains will be sufficient to maintain the platform temperatures below the maximum acceptable temperature of 91^{o}F (32.9^{o} C), 5^{o}F (2.9^{o} C) above the design ambient of 86^{o}F (30^{o} C). However, in order to avoid a stagnant air feeling by the passengers, the platform ventilation fans will be operated during normal condition. The operation of platform ventilation fans is controlled using temperature sensors. During rush hours and congested train operations, the platform ventilation fans may be operated manually to provide outside air circulation through the platforms. During winter, the fans may also be operated manually to provide air circulation when required.

During fire emergencies, the platform ventilation system plays a significant role by working in unison with the tunnel ventilation system.

Stairwell Pressurization System

A dedicated fan will pressurize the access exit stair from the platform level elevator lobby to the street surface. The function of this fan will be to pressurize the emergency egress stairwell with outside air during a fire emergency in the platform area to keep smoke out of the stairwell.

Elevator Shaft Pressurization System

A dedicated fan will pressurize the elevator shaft and three elevators. The function of this fan will be to pressurize the elevator shaft with outside air during a fire emergency in the station to keep smoke out of the elevators.

Ancillary Space Heating and Ventilating Systems

A supply and exhaust ventilation system is provided in ancillary electrical and mechanical rooms to remove heat produced by the equipment, and maintain the space temperature within acceptable limit. Outside air is distributed into the rooms while the warm air from the space is exhausted to the atmosphere. The supply air to rooms housing electrical and electronics equipment is filtered. The ventilation system is designed to maintain a positive pressure within the space when it is in operation.

The generator room ventilation system includes air intake wall louvers and air discharge ducts above the roof. When the generator is in operation, the radiator fan will pull air over the engine body and discharge the warm air to the atmosphere. The make up air is drawn into the room through louvers. The radiator fan is sized to overcome the pressure drop incurred by the ventilation airflow.

Toilet, janitors, valve room, and other ancillary spaces are provided with exhaust ventilation systems.

Supplemental heating is provided in all ancillary spaces to keep the space temperature above freezing in order to avoid maintenance problems. Electrical unit heaters, cabinet heaters or baseboard heaters are provided as appropriate

Air Conditioning

The Elevator Machine Room houses microprocessor and other solid-state equipment for the control of the elevator operations. During normal operations, elevators are the main egress and exit from the station platforms. Therefore, to avoid elevator breakdowns of the microprocessor due to high space temperature, a mechanical cooling system is provided. The UPS Room, which houses solid-state equipment, is also provided with mechanical cooling system. The operation of these cooling systems is controlled with space thermostats.

Comfort Heating and Air Conditioning System

The Station Attendant's Room located next to the elevator lobby at the plaza level will serve as a command post during an incident in the Weehawken tunnel. The room houses solid-state controls for the tunnel and platform ventilation equipment and fire management panel for the station. Furthermore, the station attendant will occupy the room from time to time. Therefore, a comfort HVAC system is provided. The operation of the HVAC system is controlled with a space thermostat.

FIRE PROTECTION SYSTEMS

The fire protection systems to be provided for the Weehawken Tunnel and Bergenline Avenue Station include dry standpipes, automatic sprinklers, portable fire extinguishers, and clean agent fire extinguishing systems.

The tunnel is protected with a standpipe system. The platforms and elevator lobby are protected with standpipe and sprinkler systems. The ancillary spaces are protected with a clean agent fire protection system or fire detection and alarm system as appropriate.

Portable fire extinguishers of appropriate rating are provided in platforms, the elevator lobby at platform level, and in all mechanical and electrical ancillary spaces.

ELECTRICAL SERVICE AND POWER DISTRIBUTION

Electrical Service

Two independent electrical services from Public Service Electric and Gas supply power to two outdoor 35 kV fused load break-switches and 26.4 kV-480/277 Volt, dry-type epoxy resin transformers, two 480/277 Volt Indoor Switchgear Assemblies and two 480/277 Volt Indoor Distribution Switchgear Assemblies. Incoming service feeders terminate in the electrical room of the utility building located at the plaza level. In addition to the dual incoming utility power sources, a natural gas driven engine generator will be provided to serve critical loads such as elevators, emergency egress lighting, communication systems and other loads served by the uninterruptible power system.

Power Distribution

One substation will serve the large power requirements of the tunnel and platform ventilation systems in order to avoid electrical disturbances to other facility systems caused by voltage dip of large ventilation fan motors during starting. The other substation will serve the power requirements of the station, tunnel, utility building, plaza level and ancillary spaces including elevators, elevator shaft and stairwell pressurization fans and general power and lighting loads.

PSE&G will provide dual incoming medium voltage services. Upon the loss of one utility company service, the loads will be automatically transferred to the remaining utility company service. If the remaining service fails, normal power to the facility will be interrupted. The UPS will continuously supply power to critical loads for 90 min if the generator fails to start. During that time interval critical loads and emergency loads will be transferred automatically to the natural gas driven standby generator.

Uninterruptible Power Supply

The UPS system will be used to maintain power to the emergency egress lighting, critical life safety, and communications systems in the event of power failure. The UPS system will provide capacity for 90 min time interval.

LIGHTING

The lighting system for the station platform, plaza level, Utility Building, miscellaneous mechanical and electrical will include fluorescent, metal halide, and high-pressure sodium luminaries. Illumination level for each room, location and area will be as recommended by the Illuminating Engineering Society (IES) and the American Public Transportation Association (APTA).

Tunnel Lighting

The average maintained illumination level for the tunnel interior zone will be 1.5-ft (45.7-cm) candles. Daytime average maintained illumination levels for the first 300 ft (91 m) of the tunnel from the portals is 10-ft (3-m) candles. Nighttime levels will be reduced to the interior illumination levels as in the interior zone. Controls will be via astronomical timer/lighting contactor assemblies.

Emergency Lighting

Emergency lighting will be provided to permit passenger egress from the station platform, ancillary spaces and tunnel during an interruption in service to the facility. Emergency lighting will be provided throughout the facility, stairs, platform, and tunnels. The emergency lighting will be fed from the UPS system until the stand-by natural gas driven generator comes on line. The generator will feed the emergency egress lighting system through the UPS system.

The tunnel is designed with wayside walkways. During an emergency, tunnel trackways will be used as an alternative emergency egress path. Average maintained illumination level for emergency egress is 1 ft (30 cm) candle, minimum illumination levels is .25 ft (76 mm) candles and will be designed in accordance with the applicable sections of NFPA 130, Fixed Guideway Transit System, the NFPA 101, Life Safety Code, and the APTA Guidelines for Design of Rapid Transit Facilities.

FIRE ALARM AND DETECTION

An electronically supervised, zoned fire/smoke detection and alarm system will be installed at the Bergenline Avenue Station. This system will consist of a microprocessor based Fire/Smoke Control Panel (FACP), part of the Emergency Management Panel (EMP); remote annunciate, part of the Emergency Information Panel (EIP) and other support equipment and devices such as manual alarm pull stations, smoke and heat detectors, sprinkler water flow switches, standpipe water flow switches, speakers, strobe light, etc.

The FACP will be installed in the Fire Management/Attendant Room located at the plaza level headhouse. The FACP will monitor sensing and indicating devices and initiate alarms to the Operations Control Center. Authorized personnel can monitor the status of the FACP during emergencies from the FACP or remote annunciator in the EIP located at the elevator lobby. The monitoring of alarm status and control of support equipment/devices can also be done from the Operations Control Center (OCC) through the Mechanical/Electrical SCADA system. All fire system alarms will generate visual and audible indication at the FACP.

Public Address System

A public address system will be provided in the Bergenline Avenue Station.

Intrusion Detection

All outside entries into non-public spaces at the plaza level will be monitored for unauthorized entry. Similarly, access into ancillary spaces at the platform level and Utility Building, such as equipment rooms, fan rooms, etc., will also be monitored for any unauthorized entries. The intrusion detectors will interface with the Supervisory Control and Data Acquisition (SCADA) system.

FIRE EMERGENCY MANAGEMENT SYSTEM

The SCADA system will provide operating personnel with an effective method for controlling and monitoring the ventilation and electrical systems from a central location. The primary control SCADA video display system (VDT) will be located at the OCC, and a fallback control VDT provided within the Fire Management/Attendant Room at the Bergenline Avenue Station. Within the Fire Management/Attendant Room a hardwired Emergency Control panel (EMP) will be provided and will include the following systems:

- Ventilation System (ECS Operator Panel)
- Fire Alarm Panel

- Intrusion Detection Panel
- Elevator information
- Fire suppression systems status (shown on the Fire Alarm Panel)

The SCADA will integrate these systems.

Each of these systems will continuously send data and status information, which will be available at both the OCC and at the Station's Fire Management/Attendant Room SCADA VDT. The OCC SCADA will normally be in control with the Station SCADA restricted to monitoring and maintaining a station level database of equipment status. The SCADA system will continuously monitor and alert the control center operator following any changes in status for the systems monitored. For the ventilation system the operator may manually implement operations or accept pre-programmed automatic responses to anticipated events.

The SCADA system is to be a computer-based control system with a high-speed data bus that carries information from various devices to the central control computer via reconfiguring redundant fiber optic communications cables. Redundant equipment is to be used to increase system reliability.

CONSTRUCTION

When the HBLR Minimum Operate Segment–2 (MOS-2) work effort was negotiated with TFC, Design Unit N30 was included as an open book item. This means that an allowance was provided for this work in the MOS-2 change order. As part of TFC's responsibility, N30 was to be competitively bid. NJ Transit and the design engineer, PB, were included in the presentations to the bidders and the technical review of the proposals.

PB prepared the final design drawings and specifications. TFC prepared the contractual documents. TFC was also responsible for preparing the Request for Proposals and in charge of the negotiations during the bid process. While price was the major considerations during the bid process, the technical and contractual qualifications were also considered as part of the selection process. The bid process occurred during calendar year 2001. The N30 contractor, FKSB, was selected towards the end of 2001. The agreement with FKSB was made in January 2002 and executed on March 21, 2002.

Initial Construction

The initial construction activities were setting up of the field offices, baseline survey, pre-blast survey, asbestos abatement, rodent control survey and preparation of the health and safety plan.

On June 17, 2002 Conrail Freight Service was rerouted and the tunnel was released for construction. The initial construction in the tunnel was the removal of the ballasted track and ties and soot from the tunnel walls. Also the areas along the east and west portal were cleared and grubbed along with the initial scaling of the rock walls at the east portal. The pre-blast survey of the buildings along the tunnel surface was also completed. The fencing and gates were installed at the Bergenline Avenue Station plaza site along with the demolition of the one building still remaining on the site. The one archaeological dig at the plaza area site was completed in August 2002.

The relocation of the PSEG cables in the tunnel was a joint effort between FKSB and PSEG. FKSB prepared the trench. PSEG then laid the new conduits and FKSB installed the steel reinforcing and poured the concrete for the ductbank. After the duckbank installation was complete, PSEG placed the cables, one ductbank at a time. The cables in the second ductbank were not installed until the first set of cables were installed and up and running. This work was started in October 2002 and completed in January 2003.

The work on the elevator shaft vertical bore was started in December 2002. The initial work was the drilling of an 8 in. (20.3 cm) pilot hole from the surface to the tunnel crown. After the completion of the pilot hole, an 8-ft (2.4 m) diameter raise bore was drilled from the tunnel crown to the surface plaza. This work was done in January 2003. The enlargement of the shaft to final outside diameter of 42 ft (12.8 m) was completed in August 2003. This was done in maximum blast sections of 8 ft (2.4 m). After blasting immediate support in the new shaft comprised systematic epoxy-coated-cemented grouted rock dowels, welded were mesh and 4 in. (10 cm) of steel-fiber reinforced wet mix Shotcrete.

In the first quarter of 2003, the rock excavation at the plaza level began and work progressed in filling the existing open construction shaft. Work on removal of the brick lining and blasting for enlargement of the tunnel began in April 2003. The rock from the plaza blasting operation is key pushed through the raise bore shaft and removed via the tunnel. In June 2003, drill and blast excavation of the station transition zone started.

Future Construction

The current N30 schedule calls for the following milestone dates:

Targeted Milestone	*Baseline Schedule*
Complete all blasting	12/03
Signal Room complete to Allow Work by Signal Contractor	10/04
Release Tunnel, Station, and Plaza for Systems Work	10/04
Complete Trackwork	1/05
Complete Headhouse, Utility Building, and Plaza	1/05
Complete Station	12/04
Complete All Work	1/05

This work, upon its completion, will allow the vision of the early railroaders of providing access through the Palisades to continue.

The revenue ready status of HBLR MOS-2 Phase 2B, which includes Design Unit N30, is scheduled at this time for June, 2005

ACKNOWLEDGMENTS

The writers would like to thank NJ Transit for its support in the preparation of this paper and Anna Parciak for the word processing of the document.

Floating Slab Trackbed Design to Control Groundborne Noise from Newark–Elizabeth Rail Link Light Rail Transit

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NJ Transit is in the process of final design for the Newark–Elizabeth Rail Link (NERL) project, a new light rail transit alignment that will connect with the existing Newark City subway system. A regional Performing Arts Center is adjacent to a portion of the new rail alignment and at some point in the future, the Performing Arts Center may also construct a concert hall directly adjacent to the new rail line. Detailed analysis of the potential for groundborne noise from ground vibration generated by light rail vehicles was performed by Wilson, Ihrig and Associates, Inc. (WIA) working with the BRW/Parsons Brinckerhoff (PB) Joint Venture team. WIA determined the track support system adopted for the NERL project would result in higher levels of groundborne noise than appropriate for the Performing Arts Center. The goal of the team's work was to determine an effective mitigation measure, which would control ground vibration and avoid interference from groundborne noise. The tracks will be at-grade and accessible by the public which was a significant consideration. Different track support systems were evaluated for their effectiveness in resolving the issues. A sealed floating slab system was selected as the most effective and offered ultimate protection should the concert hall be built at a later date. A unique construction technique developed by BRW/PB Joint Venture and KS Engineers for the concrete slabs supporting the track will also be presented.

INTRODUCTION

The Newark–Elizabeth Rail Link (NERL) Minimum Operable Segment 1 (MOS-1) is an approximately 1-mi, four-station extension of the Newark City Subway (NCS) that is currently being constructed by NJ Transit in downtown Newark, New Jersey. The NCS extension or NERL MOS-1 connects Newark's Penn Station on the Northeast Corridor with Newark's Broad Street Station on the Morris and Essex Lines as shown Figure 1. Between these two commuter rail stations the NERL MOS-1 runs through Newark's Arts District servicing the Washington Park office area, the Newark Art Museum, Newark Library, Riverfront Baseball Stadium, and the New Jersey Performing Arts Center (NJPAC).

The NJPAC is a $180-million performance center with two theaters that opened in October 1997. The opening of the NJPAC marked the return of Newark as a regional cultural

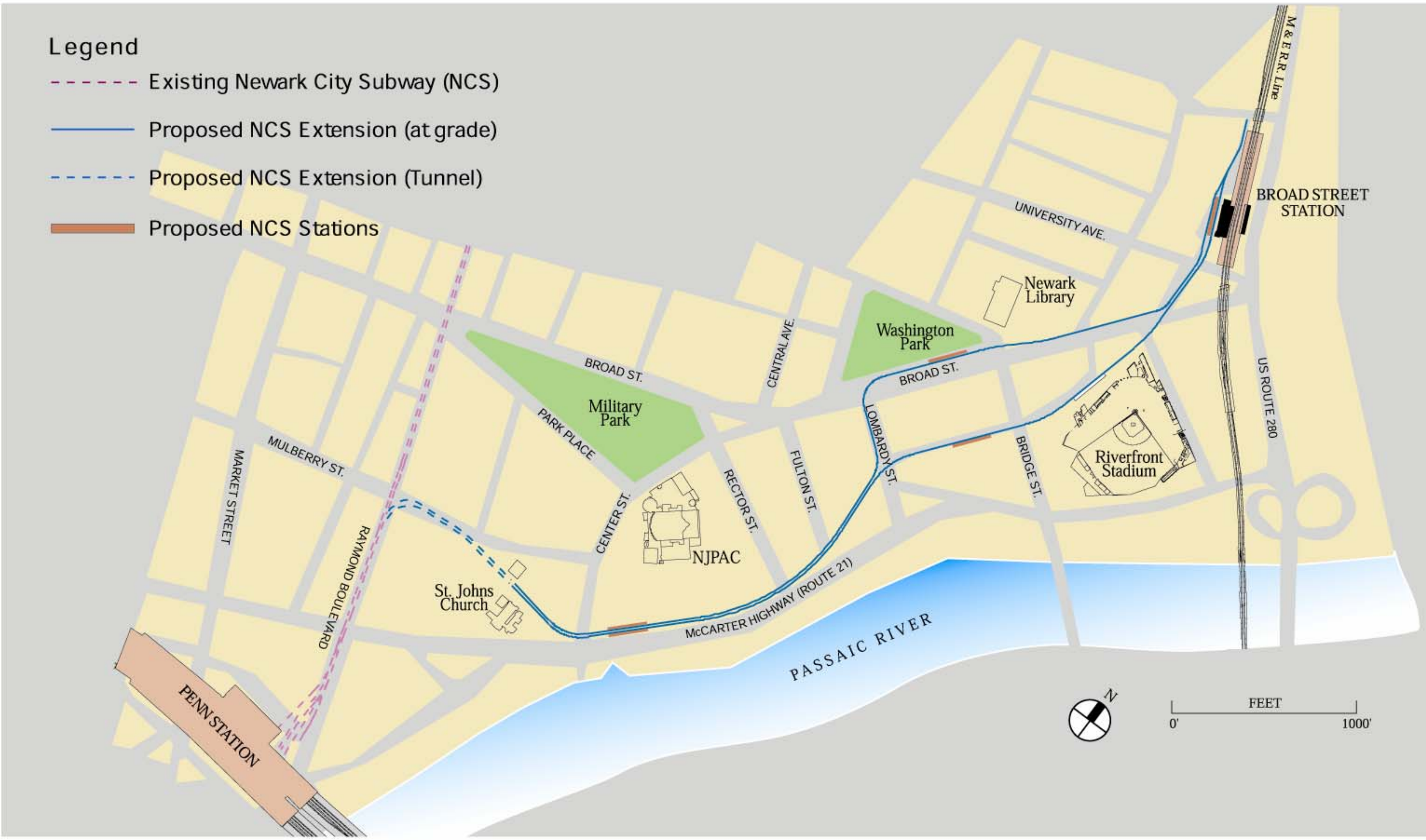

FIGURE 1 NERL MOS-1 alignment.

center and significantly improved the quality of life in downtown Newark. Many local corporations, the city of Newark, and the state of New Jersey sponsored construction of the NJPAC. An artist's rendition of the NJPAC buildings prior to construction is shown in Figure 2.

The view shown is from the north looking south towards the Passaic River with Center Street in the foreground. The planned NERL light rail transit (LRT) alignment is behind the second building (the future concert hall), which is shown in outline.

The environmental consultants to NJ Transit, URS/BRW Team, completed the NERL draft environmental impact statement (DEIS) in January 1997 prior to the opening of the NJPAC. The alignment of the NERL MOS-1 runs adjacent to the NJPAC site within 160 ft (48.8 m) of Prudential Hall and only 50 ft (15.2 m) from the site of a future concert hall that had been planned at the time of the NERL DEIS. The NERL MOS-1 will include an NJPAC station near the possible location of the potential concert hall. The NJPAC was under construction during the NERL environmental analysis and its completion was highly anticipated. The potential for the NERL trains to have detrimental noise and vibration impacts on performances was of great concern to NJPAC and the city of Newark. These concerns were communicated to NJ Transit and their consultants URS/BRW.

URS/BRW requested that Wilson, Ihrig &Associates, Inc., (WIA) evaluate the potential impacts of the LRVs on the NJPAC and recommend ways to mitigate the potential impacts. The DEIS and final environmental impact statement identified the NJPAC as a sensitive receptor that would require mitigation measures to reduce groundborne noise and vibration. WIA looked at multiple methods of vibration isolating the LRT, including special rail support systems, ballast mats, and a floating slab. NJPAC officials withdrew their objections and supported the NERL project once they were briefed on the various ways to mitigate the potential impacts to their performance halls by the consultant team, who promised further investigation during final design.

During preliminary engineering, NJ Transit evaluated the different options for mitigating potential groundborne noise and vibration impacts to NJPAC facilities. NJ Transit wanted to be certain that there would be no discernable impact from noise or vibration to the NJPAC. The final design was awarded to the BRW/Parsons Brinckerhoff Joint Venture (PBJV) and included WIA as the noise and vibration consultant. In the final design phase of NERL MOS-1, and after much deliberation, the recommendation to use a floating slab track (FST) system for the track alignment adjacent to the NJPAC site was made by WIA to the BRW/PB JV engineering team, and accepted by NJ Transit. The NERL MOS-1 is currently under construction and the plans call for an 840-ft (256-m) floating slab running from Center Street through the NJPAC Station to Rector Street in the vicinity of the NJPAC.

Several design issues presented themselves in this project to develop the FST system for NERL MOS-1. Some of these issues were practical in nature, and others involved the performance of the FST system. In this paper, the issues are discussed and the approaches taken to address them presented. Details of the design are presented along with the performance expectations for the FST system designed for NERL MOS-1.

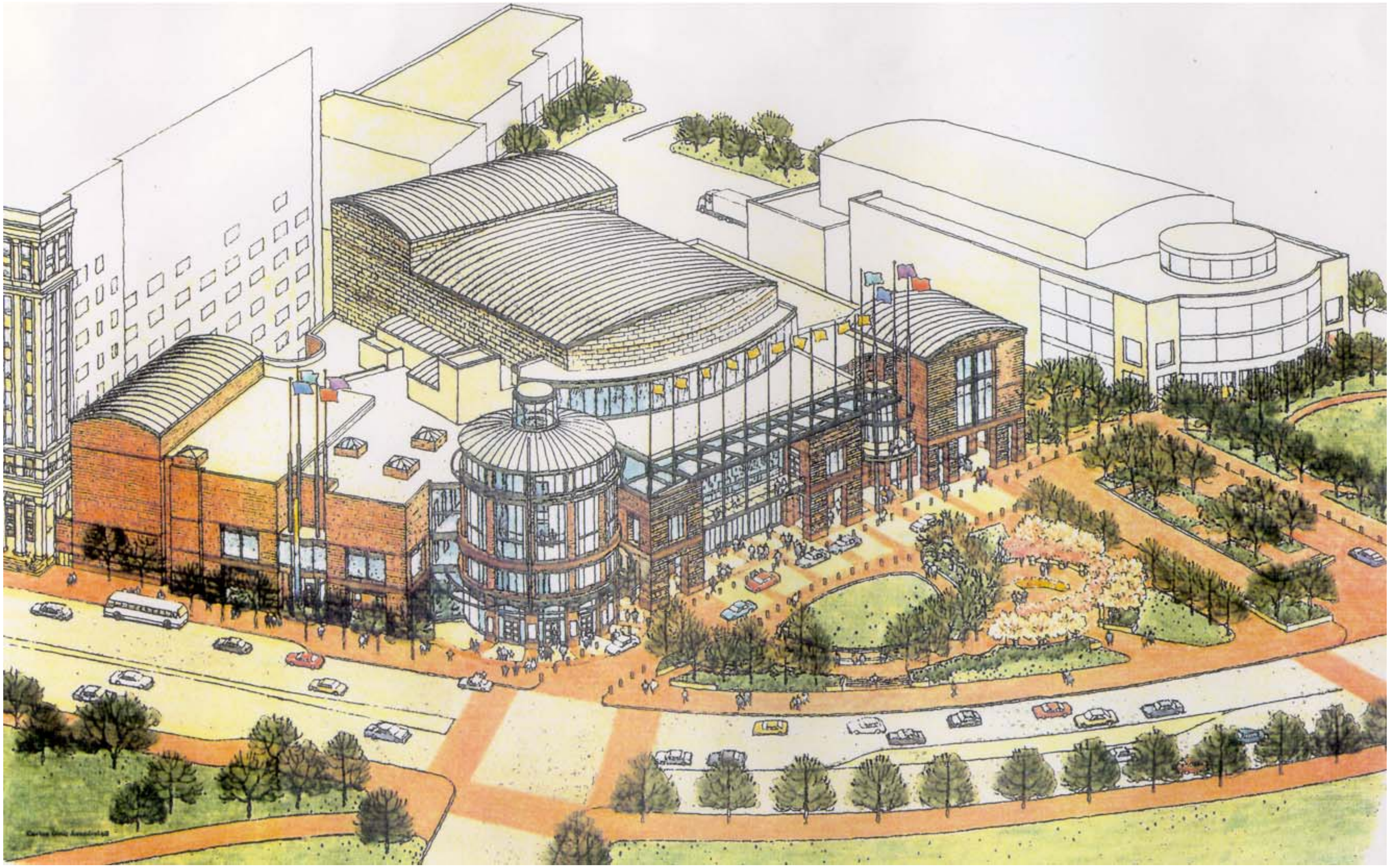

FIGURE 2 Overview of NJPAC fronting Center Street.

FLOATING SLAB DESIGN ISSUES

In basic terms, an FST system is comprised of springs and masses, which are designed to isolate vibration coming from wheel or rail interaction, and decrease its transmission to the surrounding track support structure. The amount of isolation necessary depends on the amount of vibration reduction required by the particular circumstances of each situation. The circumstances involve many factors, including

- Sensitivity of the affected building,
- Speed of the transit vehicles,
- Rail roughness,
- Dynamic interaction between the vehicle's trucks and the rail system,
- Response of the soil underlying the track,
- Ease of propagation of vibration through the soil between the track and the building,
- Response of the building to ground vibration, and
- Manner in which vibration is transmitted through the building.

The amount of vibration reduction that can be achieved using an FST depends on the dynamic characteristics of the transit vehicle, but is dictated to a large degree by the primary natural frequency of the FST system. The FST can be idealized as a simple spring-mass and damper system as depicted in Figure 3. In actuality, it is a more complex dynamic system than this, but for determining the basic performance of the FST it often suffices to model it in this manner. Field tests performed on a full scale FST mock-up (*1*) demonstrated this, in particular when the FST is under vehicle load.

The FST system will have a natural frequency determined by the stiffness of the supporting springs and the amount of mass the springs support. Contrary to an occasionally expressed opinion, the mass of the transit vehicle does not affect the natural frequency of the

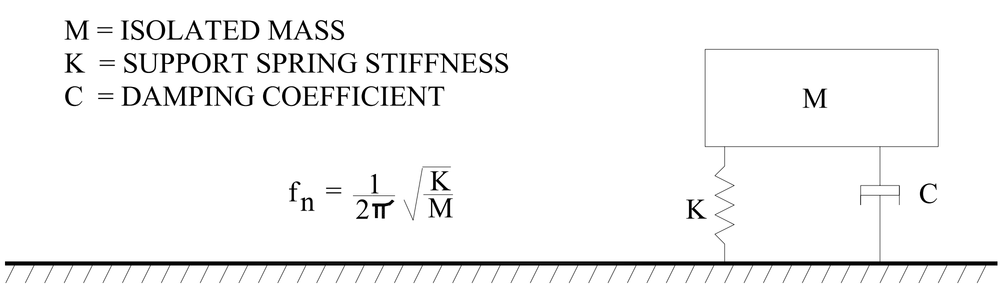

FIGURE 3 Idealized spring-mass and damper system.

FST, except by changing the stiffness of the support springs due to its static loading on the FST. The secondary suspension, located between the truck and vehicle, has a low natural frequency (typically 1 to 2 Hz) effectively decoupling the vehicle from the FST. However, the mass of the truck or some portion of it (e.g., wheelset), depending on the nature of the primary suspension system, will contribute to the dynamic mass of the FST system.

Once a suitable natural frequency for the FST is determined, the appropriate stiffness of the springs and mass of the slab trackbed can be determined. The NERL MOS-1 FST system design uses "natural rubber" springs and a hybrid pre-cast concrete slab combined with a cast-in-place (CIP) slab. Natural rubber has been used for over 100 years as a material for structural isolation elements, and has been found to be highly durable and an ideal spring material for FST after its first use in North America in this manner in 1970.

A generic FST system concept is shown in cross section in Figure 4. The rubber pads rest on either an invert or the bottom of a concrete tub as in the case of NERL MOS-1. The concrete slabs (masses) are placed on top of the rubber pads. To restrain lateral and longitudinal motion of the concrete slabs in the horizontal plane, discrete rubber pads or continuous rubber strips are used around the edges and in-between adjacent slabs.

The FST system for NERL MOS-1 presented several design issues and challenges not typically encountered. Two of the main issues involving groundborne noise and vibration projections were

- Lack of an existing fleet of vehicles to use for measurements, and
- No at-grade operations for measurements.

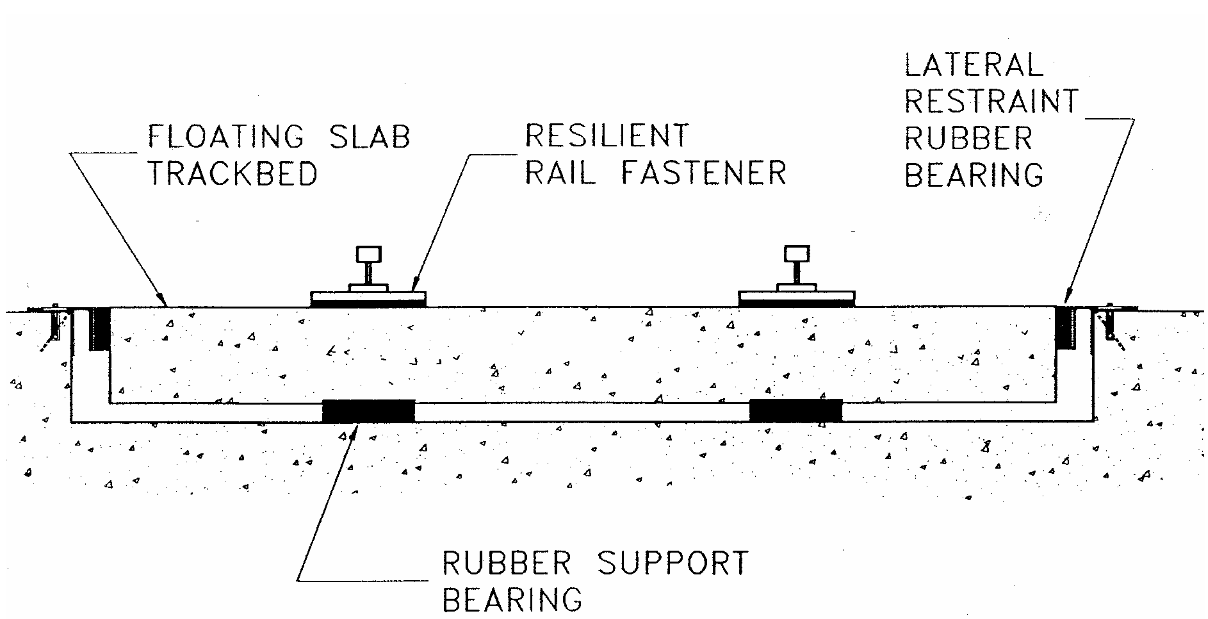

FIGURE 4 Typical FST configuration.

Most often an FST system is designed for an existing transit system with a fleet of transit vehicles already in use. Thus characteristic vibration data for the transit vehicles and track can usually be obtained by measurements using the existing system. The NCS and eventually the NERL extension are scheduled to switch over to new low-floor vehicles. These vehicles were not available during the final design phase of the NERL project. NCS/NERL will use essentially the same low floor vehicles, which were being manufactured at the time by Kinkisharyo of Japan and assembled in Harrison, New Jersey. While the existing NCS LRT is a subway, the NERL system will be essentially all at-grade, except for a short transition to grade connecting the two. These two factors complicated the task of predicting groundborne noise and vibration levels, which would be generated by the future NERL System.

Fortunately the nearby and then brand new (1998) Hudson–Bergen LRT system was finished prior to completing final design engineering for NERL MOS-1, and it was conveniently available for making vibration measurements to characterize the Kinkisharyo low floor LRV. The Hudson–Bergen system has a very similar embedded track system to NERL, and uses identical vehicles, which operate at-grade. It was therefore possible to obtain relevant measurement data on the groundborne vibration characteristics for the NERL system using the Hudson–Bergen system as an exemplar.

The receptor side of the problem also presented a challenge. NJPAC's Prudential Hall had been on the site and open to the public for a little over a year at the start of the NERL MOS-1 final design, but design and construction of the concert hall was and still is somewhere off in the future. The Prudential Hall will be 160 ft (48.8 m) from the nearest track, whereas the future concert hall, if and when it is built, would be much closer to the new tracks. Consequently, of the two buildings, the one that is more critical for groundborne noise and vibration had yet to be built during the NERL MOS-1 design process.

Although the appropriate noise and vibration criteria for the NJPAC buildings were not in question, it is unusual for a building to be protected by an FST system not to exist. Most FST systems are designed and constructed after the buildings they will protect are. In such situations, it is possible to physically measure the vibration response of the specific building and incorporate the measured data into the prediction model. In the case of NERL, since the concert hall did not exist it was not possible to do this. Consequently, data from previous measurements on similar buildings were used instead.

The Prudential Hall is a noise and vibration sensitive public facility, but it is not as critical as the concert hall would be if it were built. The concert hall would be more sensitive to noise than Prudential Hall and it would be considerably closer to the NERL MOS-1 tracks. The two theaters in Prudential Hall were designed by the acoustical consulting firm of Artec Consultants Inc. to have very low ambient noise. Measurements made by Artec (*2*) after the theaters were constructed confirmed this. However, Prudential Hall will be 100 ft (30.5 m) farther from the new tracks, than the concert hall would be. This additional distance will attenuate the vibration transmitted through the ground due to damping in the soil and spreading losses.

Contemporary concert halls are designed to have an extremely low ambient noise environment inside the performance space, with considerable effort expended to design and construct an ultra-quiet heating, ventilation, and air conditioning (HVAC) system. The noise criterion used in designing new concert halls is often driven not so much by the need to avoid interference during a live performance, but the common practice of renting the performance space for making audio recordings of concert or other types of music. Modern audio recording technology places a severe demand on the ambient noise level that is acceptable in premier performing arts facilities.

By applying state-of-the-art design techniques and carefully constructing the performance space in the new facility, noise levels in the performance space as low as N1 (a special acoustical design criterion developed by Artec) can be obtained even with the HVAC system operating. This criterion is essentially the threshold of audibility except at frequencies below 125 Hz for which the acceptable noise levels are even lower to avoid interference with recordings.

Another factor considered was the existing ambient vibration coming from traffic on the highway adjacent to the possible concert hall site. The site for the concert hall is in close proximity to McCarter Highway (Route 21), which has substantial truck traffic. NERL MOS-1 will be in-between McCarter Highway and the NJPAC site. During preparation of the DEIS for the NERL project, ambient vibration measurements made by WIA in 1995 at the approximate location of the potential concert hall façade closest to McCarter Highway indicated that the overall vibration levels are between 55 and 67 VdB (1 micro-in./s). McCarter Highway will be relocated slightly to accommodate NERL tracks and the highway will be resurfaced. Moving the highway further away would tend to reduce the ambient vibration at the concert hall site. Resurfacing the highway will, at least in the short-term, make the road smoother, which will also tend to reduce the vibration generated by vehicles (especially trucks) traveling on it. Designers of the concert hall would need to address the presence of motor vehicle generated ambient vibration and special building isolation measures may be needed to avoid interference with the performance space.

There were also practical design issues, which make the NERL FST requirements somewhat unique. The design team had to consider ways to overcome the following issues:

- Shared right-of-way
- Public access to track

Most FST systems, in particular those for heavy rail transit, are installed in subway tunnels or at least in an exclusive right of way. There are relatively few FST systems installed on LRT systems in North America. Examples are San Francisco, California; Buffalo, New York; and Toronto, Ontario. LRT systems typically share the right of way with motor vehicles, and it is common for pedestrians to have access to the trackway, such as will be the case with NERL, a factor which imposes practical design constraints on the FST.

Except for the connection with the NCS, the LRT alignment for NERL MOS-1 will be in the public right of way. Some portions will be in the roadway and others directly adjacent to the sidewalk. For public safety reasons, the basic NERL MOS-1 track was designed to be an embedded construction using a rubber “rail boot” system. The 115# RE rail and the encasing boot are held in place with an anchoring system that uses Pandrol clips.

Where pedestrians have access, but not motor vehicles, the top surface of the NERL MOS-1 standard track will be surfaced with paving stones. Where the tracks are in the street, the paving will be asphalt. For obvious aesthetic reasons, the NERL FST had to incorporate these same features into the design. Although not a major consideration in the FST design, it is a secondary factor in determining the mass of the concrete slab to be used in the FST system.

Most FST designs have gaps around the edges of the slabs to allow air to move freely from under the FST, in which case the air does not contribute to the vertical stiffness of the FST spring-mass system. For the reason stated, the NERL FST design could not have gaps at the edges, and was therefore designed as a "sealed system."

Air is nearly incompressible, and, in a sealed FST design under dynamic loading, the air, to a large degree, is trapped and acts as a spring. The stiffness of the trapped air contributes substantially to the dynamic stiffness of the spring support system for the FST. Consequently, the added dynamic stiffness of the air affects the appropriate dynamic stiffness of the rubber support pads. For a given total vertical stiffness desired for the FST, the additional air stiffness reduces the allowable rubber pad stiffness. Whereas, under static loading, it is possible for the air to move and the stiffness of the air has little or no effect. These two competing factors must also be taken into account in the design.

Other LRT FST systems designed by WIA, which are in-street installations [San Francisco Municipal Railway (Muni), in a residential neighborhood on Noe Street in San Francisco], or accessible by pedestrians [Niagara Frontier Transportation Authority (NFTA), at the Marine Midland Bank in Buffalo] have designs similar to the NERL FST. The tracks for the systems are embedded and the top surfaces of the trackbed are paved with asphalt. Both of these FST systems are also sealed designs. A major difference though between these two designs and the NERL FST is the construction of the concrete slabs.

Both the Muni and NFTA track slabs used a CIP construction, whereas the NERL FST is a hybrid pre-cast concrete and CIP slab design. In a CIP construction, a metal pan is used as the concrete form and is left in place, becoming part of the system. The FST rubber pads support the metal pan underneath, into which the concrete is poured. The thickness of the metal pan must be sufficient enough to support the weight of the wet concrete as it cures. The span between support pads is therefore critical, if the bottom of metal pan is not to sag substantially.

The NERL MOS-1 FST system sits in a concrete "bathtub" construction with the rubber pads resting on the bottom of the tub. Each track will have its own independent FST system. The decision to use a pre-cast design was primarily dictated by the presence of a center drain in the concrete invert. The center drain affects the number of rows of support pads that can be used, in that it precludes a design with an odd number of rows. The typical pre-cast FST slab designs (e.g., Bay Area Rapid Transit, Metropolitan Atlanta Rapid Transit Authority, and Toronto) have two rows of support pads. The Muni FST has three rows of support pads, because it has no drain channel in the invert.

Using four rows of pads would result in the pads having to be much smaller than most conventional pad designs. Since the NERL MOS-1 FST is a sealed system, the air stiffness would also require that the support pads be smaller than in an open FST system. These factors would have resulted in a substantial increase in the total number of pads and use of a non-conventional pad size. Consequently, it was decided to use two rows of pads and stick as close to an existing conventional pad design as possible.

To prevent the metal pan from sagging, the BRW/PB team decided to use a hybrid pre-cast concrete and CIP slab design. As a sub-consultant to the BRW/PB team, KS Engineers (of Newark) assisted in developing the design concept and design details (in particular the steel reinforcement and attachment details), and were responsible for the contract drawings for the NERL MOS-1 FST. Each of the slabs will be constructed in two pours. The pre-cast slabs will be installed on top of the rubber pads and serve as the bottom of the concrete form for the second pour.

FLOATING SLAB DESIGN DETAILS

The final design selected for the FST is shown in plan view in Figure 5 depicting the basic slab units. The basic NERL MOS-1 FST design consists of concrete slabs that are nearly 30 ft (9.1 m) long. The concrete slabs will be a two-pour system as shown in Figure 6. Pre-cast, steel reinforced slabs that are 29 ft, 5 in. long (897 cm), 6 ft, 5 in. (196 cm) wide, and 12 in. (30.5 cm) thick will form the base for the second-pour concrete. The weight of the pre-cast slabs will be approximately 29,000 lb (125,000 N). The second-pour will add another 1 ft, 2 in. (35.6 cm) of concrete that is 7 ft (213 cm) wide. The second-pour is also steel-reinforced. Where needed, a recess formed in between the two embedded rails will contain Belgian block masonry paving stones, which are 5 in. (12.7 cm) thick. Every 200 ft (61 m) there will be a drain clean-out hole in the slab.

Except at the ends of the FST system, there will be two longitudinal rows of natural rubber support pads spaced at 33 in. (83.8 cm) apart lengthwise. This resolved the problem of the drain under the FST. At the two ends of the FST, the number of rubber support pads is increased to provide a transition in stiffness from the relatively soft vertical support for the FST to the much stiffer embedded rail support system used everywhere on the NERL MOS-1 system.

Around the perimeter of the slabs will be natural rubber strips that will be held in place with metal channels. The rubber strips will be pre-compressed during construction and will act to resiliently restrain the FST from horizontally movement. The metal channels holding the rubber strips will be welded to metal angles that will be attached to the pre-cast slab with anchor bolts prior to the second pour of concrete.

The support pads will be located on the invert using steel rings that will be preset on the bottom of the concrete tub. The steel rings also provide a form for grout that will be used to level and provide the correct base elevation for the rubber support pads. The support pads will be manufactured from natural rubber and will be 12 in. (30.5 cm) in diameter, 4 in. (10.1 cm) thick, and have a nominal 4 in. (10.1 cm) hole in the middle. The actual size of the hole in the pad is dependent on the manufacturer obtaining a specified static and dynamic stiffness for the pad. The chemical composition of the rubber to be used in manufacture of the pads was carefully developed 20 years ago for FST applications and is clearly specified in the contract documents.

The NERL vehicles will be 90 ft (2743 cm) long, and only one truck can be on top of a slab section at a time. The AW2 vehicle load is 128,000 lb (581,000 N). The NERL MOS-1 FST is designed to have a natural frequency of 10 Hz when loaded by a transit vehicle.

The pre-cast slabs will probably be constructed off site and trucked to the construction site. After the rubber pads are placed in the concrete tub, the pre-cast slabs will be lowered onto them with a crane, with care taken on placement of the slabs. The rail and its support system will be set at the correct location with jigs prior to pouring the concrete.

PERFORMANCE OF NERL FST

Projections made by WIA of the expected interior noise generated by NERL LRVs traveling at 25 mph on the NERL MOS-1 track indicated that the N1 criterion would likely be exceeded somewhat in the Prudential Hall theaters, if the proposed standard embedded track were used as planned. The noise projections for the proposed concert hall indicated that the N1 criterion would be exceeded by substantially more for the same conditions. These noise projections were

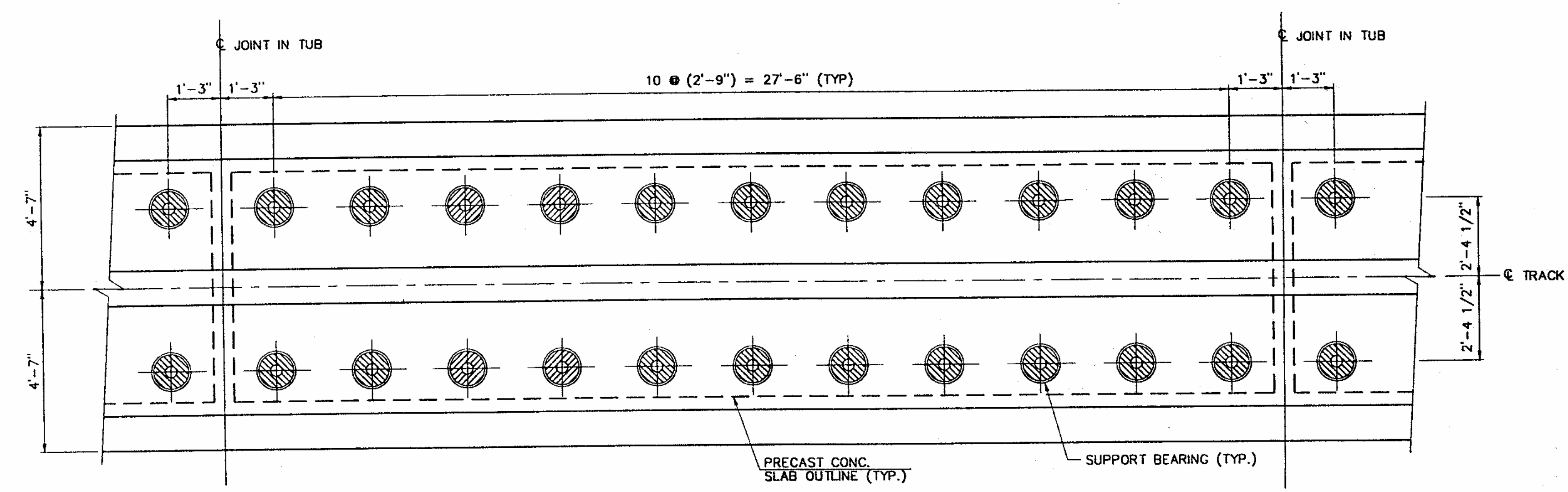

FIGURE 5 Plan view of NERL MOS-1 FST.

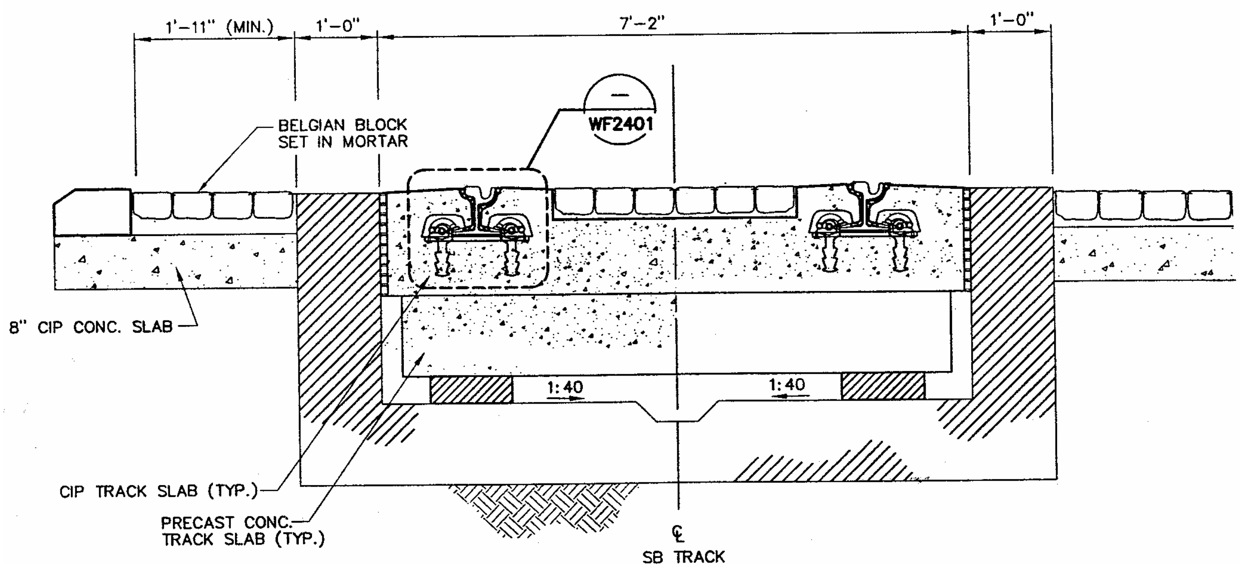

FIGURE 6 NERL MOS-1 FST.

obtained using the groundborne noise and vibration model developed by Nelson and Saurenman (*3*) and adopted by the FTA (*4*). The model relies on measurement of the transit vehicle vibration characteristics when combined with the rail system, the propagation of vibration characteristics for the surround soil, and of the response of the building of concern.

The FST system is the most effective means available for reducing vibration at the track. Without the FST, vibration would propagate through the ground and could interfere with the use of the adjacent performance spaces. It would be possible to isolate the new building or a part of the building from vibration as has been done with some concert halls (e.g., Benaroya Concert Hall in Seattle, Washington) and other critical performance spaces. However, at the time of the final design for the NERL MOS-1, it was not possible to rely on this as the sole means of groundborne noise and vibration control, as there was not even a conceptual design existed for the concert hall.

The implementation of an FST would substantially reduce the groundborne noise for both of the NJPAC buildings. The rail boot system proposed for NERL MOS-1 is relatively stiff. In comparison and FST would produce, depending on the frequency of vibration, 25 dB or more of reduction. Figure 7 shows the amount of reduction in ground vibration expected for the NERL MOS-1 FST when compared with the NERL embedded track system. Groundborne noise projections for the two NJPAC buildings were recalculated including the insertion loss provided by a 10 Hz FST in comparison with the rail boot. Groundborne noise inside the existing Prudential Hall theaters, with the NERL LRVs operating on the planned 10 Hz floating slab, is projected to be considerably less than the N1 criterion. If it were not for the possible concert hall, it would have been possible to use another, less substantial form of groundborne vibration control than the FST.

For the future concert hall, groundborne noise and vibration model projections indicate that noise levels inside a conceivable performance space have a slight chance of exceeding the N1 criterion, but an equally likely chance of being less than the N1 criterion. However, if the

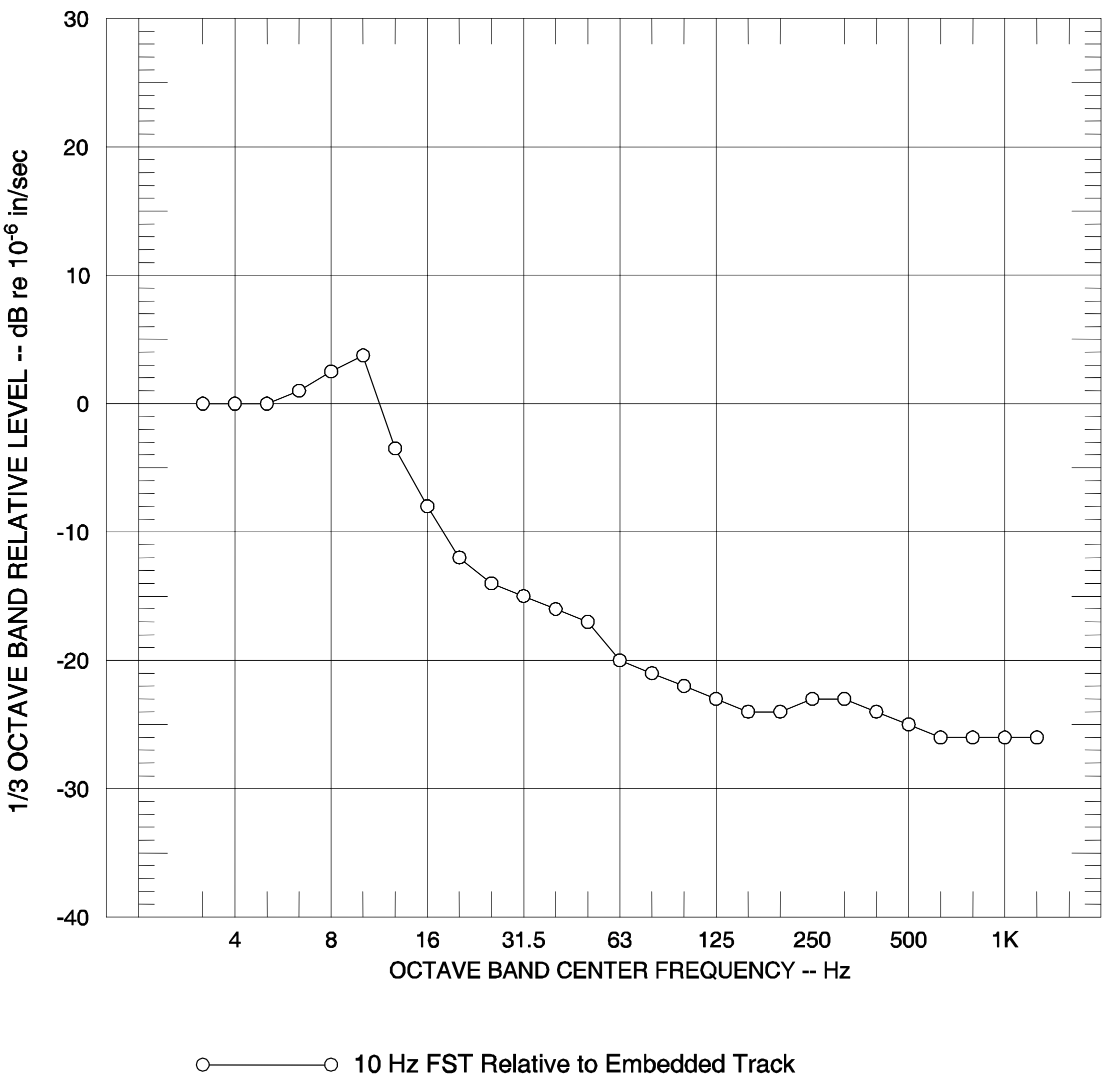

FIGURE 7 Expected vibration reduction performance for NERL MOS-1 FST.

building is removed from the model, and we consider just the vibration levels in the ground where the concert hall could be, then we can compare the predicted vibration with the existing ambient vibration. In this case, it is clear that, with the NERL MOS-1 10 Hz FST, the LRV ground vibration would be comparable to vibration generated by roadway traffic on McCarter Highway. This would be at a location which would be the future concert hall façade closest to McCarter Highway.

The analysis indicates the NERL LRT system should not affect the concert hall design anymore than the hall would by affected by McCarter Highway. The NERL LRT system therefore would impose no constraints beyond what McCarter Highway now imposes on the NJPAC site. The concert hall designers would, however, have to make a further assessment of this when the concert hall becomes a reality. That assessment would involve deciding whether the building could be designed to adequately reduce ambient ground vibration, both motor vehicle and LRT, through use of a particular foundation design (e.g., caisson) or if inclusion of some form of resilient vibration isolation system within the building to control exterior vibration would be required.

CONCLUSIONS

The NERL MOS-1 project has demonstrated that it is feasible through implementation of an FST system to have rail transit in close proximity to noise- and vibration-sensitive buildings such as a concert hall. Furthermore, it has been shown that a sealed FST system can be designed that allows public access to the right of way and incorporates the architectural aspects of the design of the rest of the LRT system. The FST design was a collaborative team effort of the BRW/PB JV, KS Engineers, and WIA that adequately resolved various constraint issues, which arose during its design. The NERL MOS-1 FST will be constructed over the course of the next year or so after selection of a contractor in the middle of this year. Start-up of revenue operation of the NERL MOS-1 is anticipated for Spring 2006.

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Fitting Light Rail Transit into Historical Centers
The Rome Experience

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The city of Rome is world renowned for its massive presence of ancient monuments, churches, and historical buildings, mostly dating from the Roman period to the Baroque Age. The noble marbles and the facades of those monuments have been severely damaged by air pollutants, mainly emitted by private and public vehicles.

To face this problem, ATAC (Agenzia per il Trasporto Autoferrotranviario del Comune di Roma), the public company which manages public transportation in Rome and in its region, began a long-term project in 1993 called "Zero Pollution Public Transportation in the Center of Rome," which aims to convert all fossil fuel operated public transportation into electric transportation through the introduction of battery operated small buses, trolley buses, and, above all, light rail transit (LRT) systems.

To accomplish this, new problems had to be faced and solved. LRT systems crossing the historical center must have minimal environmental impact; that is, unobtrusive overhead wire systems and sites for substations are necessary, as are, above all, vibration reducing track structures, since vibrations can severely damage ancient buildings. This paper will deal about the experience in Rome since 1994 with designing and testing different techniques for vibration reducing track structures.

First, the design of the two main vibration reducing track structure systems that were produced and tested will be detailed, focusing on the differences between them. Second, results of the measurements carried out at two sites in the center of Rome, before and after the installation of the vibration reducing track structures ("before works" and "after works"), will be presented.

Finally, a comparison table showing the vibration reduction and the cost of each system will be presented, the solutions adopted on the recently constructed tramway Line 8 will be shown, and information will be given about the future application of such systems in Rome.

THE VIBRATION PROBLEM

While in motion, light rail transit (LRT) vehicles dynamically impact the numerous track structure components, thereby generating unwanted vibrations, which propagate through the ground and reach the foundations of the buildings close to the LRT line (*1*).

From the foundations, vibrations extend to all the structural components of the buildings, and may also create objectionable noise levels in the apartments where people live (*2*, *3*).

Typically, there are two types of vibrations: vertical vibrations and transverse vibrations.

TECHNICAL REGULATIONS IN FORCE

For vibration limits, one must respect Technical Rule UNI 9614 (Vibration measurement in buildings and annoyance evaluation) in Italy, which is substantially in compliance with other international rules (such as ISO 2631, DIN 4150/2, and BS 6472).

For damaging effects caused to the buildings by vibrations in Italy, one must respect Technical Rule UNI 9916 (criteria for the measurements of vibrations and the assessment of their effects on buildings), which is also substantially in compliance with international rules ISO 4866, DIN 4150/3, and BS 6472.

The vibration thresholds to limit the disturbance to people inside their homes are

- Daytime—10.0 mm/s^2 (80 dB) for vertical acceleration; 7.2 mm/s^2 (77 dB) for transverse acceleration; and
- Nighttime—7.0 mm/s^2 (77 dB) for vertical acceleration; 5.0 mm/s^2 (74 dB) for transverse acceleration.

FLOATING PLATFORM SYSTEM

The floating platform system is basically made up of several layers.

The technical drawing of the floating platform system is shown in Figure 1, where the several layers are shown:

1. Stabilized base, about 5 to 10 cm (2 to 4 in.) thick;
2. Reinforced concrete platform, about 20 cm (8 in.) thick;
3. Anti-vibration mat (neoprene), about 2.5 cm (1 in.) thick, installed below the precast concrete platform;
4. Precast concrete platform, about 600 cm (236 in.) long, 230 cm (90 in.) wide, and 25 cm (10 in.) thick;
5. Side/central precast concrete slabs, which are joined to the platform by means of bolts;
6. Block pavement or asphalt; and
7. Specifically mixed rubber sections inserted along the rails, whose main functions are to reduce transverse vibrations and to take into account slight movements of the track structure.

The rails are joined to the precast concrete platforms by means of elastic fasteners. Besides the vibration abatement, the main feature of this system is that the concrete platforms and the concrete slabs are precast, and then they are sent to the construction site where they are simply installed, dramatically reducing construction time. Maintenance operations on tracks are also made easier: in the case of rail substitution, basically all the operators need to do is loosen the nuts and bolts that keep the precast concrete slabs in their place, raise the slabs up by a crane, substitute the rails and then put everything back in its place.

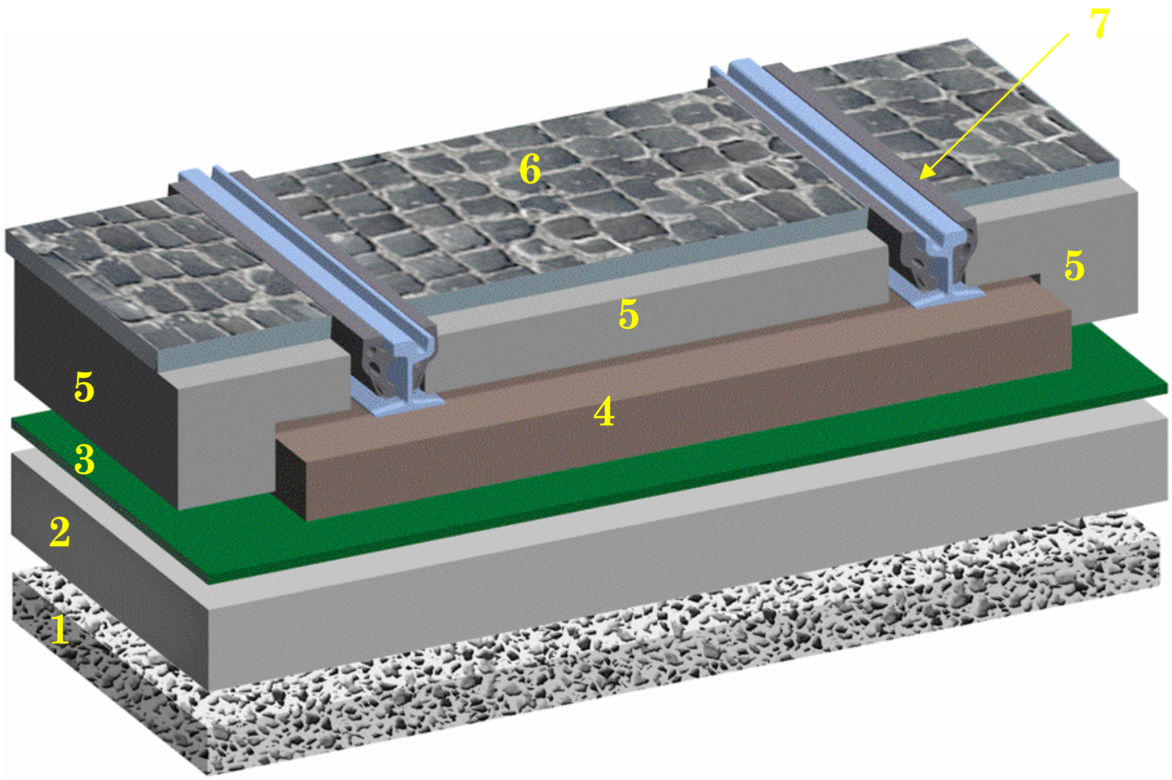


FIGURE 1 The floating platform system.

FLOATING MASS SYSTEM

The floating mass system is basically made up of several layers of concrete, precast concrete sleepers, and an anti-vibration mat (neoprene).

The main difference, with respect to the floating platform system, is that the floating mass system is not precast; instead, its construction coincides with the laying of the tracks.

The technical drawing of the floating mass system is shown in Figure 2, where the several layers are shown:

1. Stabilized base, about 5 to10 cm (2 to 4 in.) thick;
2. Reinforced concrete platform, about 20 to 25 cm (8 to 10 in.) thick;
3. Anti-vibration mat (neoprene), about 2.5 cm (1 in.) thick;
4. First layer of concrete, about 10 to 15 cm (4 to 6 in.) thick;
5. After the first layer of concrete is laid, concrete sleepers, about 27 cm (10.6 in.) long, 230 cm (90 in.) wide, and 16 cm (6.3 in.) thick, are put in place;
6. After laying, leveling, and alignment of the tracks a second layer of concrete is laid, about 16 cm (6.3 in.) thick, up to the top level of the concrete sleepers;
7. Specifically mixed rubber sections inserted along the rails, whose main functions are to reduce transverse vibrations and to take into account slight movements of the track structure; and
8. Finally, block pavement or asphalt is applied.

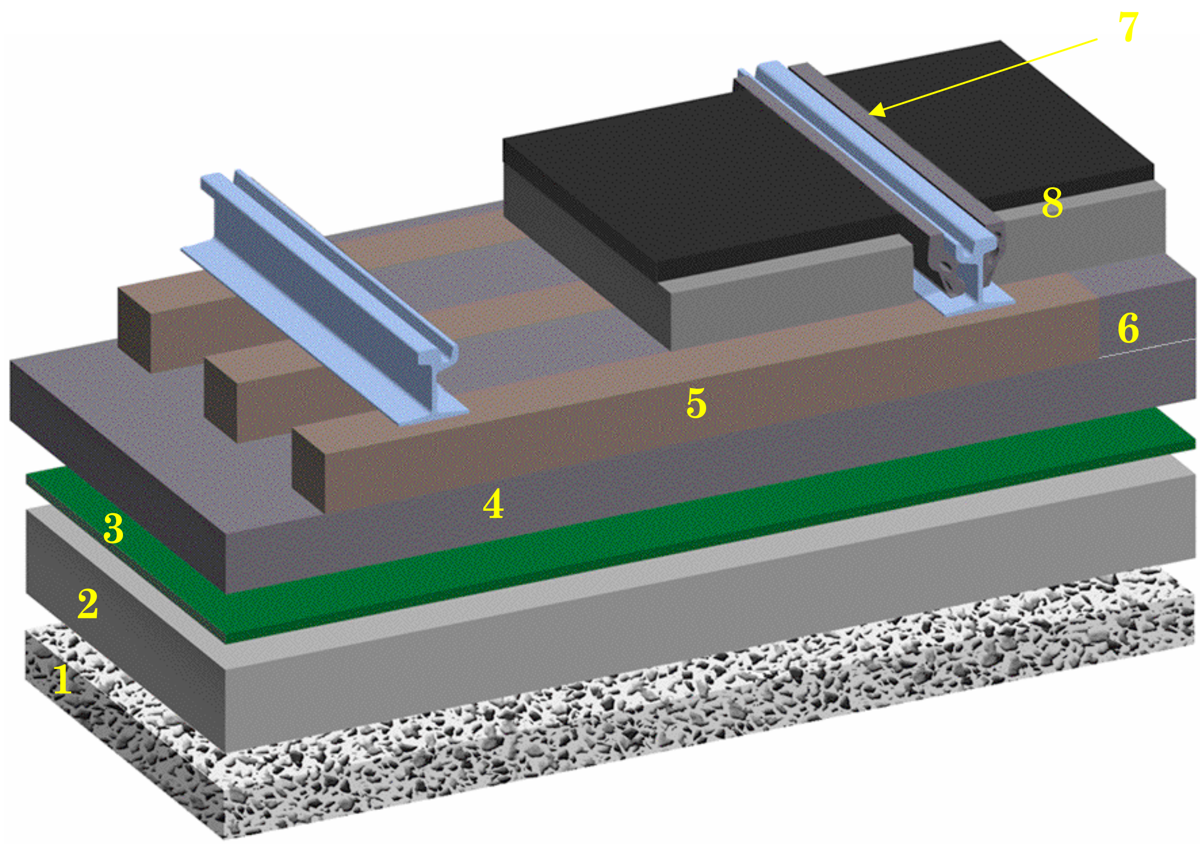


FIGURE 2 The floating mass system.

The noise and vibration abatement performances of the floating mass system are excellent. Also, a floating mass system is highly adaptable for curves and rail intersections. Unfortunately, construction time can be long and difficulties may arise when it comes to leveling and aligning the tracks.

SITE 1—VIALE REGINA MARGHERITA IN ROME

Viale Regina Margherita represents one of the major avenues radiating from downtown, and has been a tramway route since 1930.

From 1994 to 1995, large refurbishment works were carried out, with the substitution of the floating platform system for the traditional track structure system.

An aerial view of the site during the construction is shown in Figure 3, while the final result of the works is shown in Figure 4, with vehicles operating revenue service.

By means of accelerometers, vertical and transverse vibration levels were measured while the tramway vehicles were passing by, before and after the works. The measurement conditions were the same, before and after the works: same vehicles, same acceleration and velocity. Also, the "before works" rails did not show any particular signs of wear or roughness on the running surface.

The measurement points were all at ground level, at three different positions along the

FIGURE 3 Aerial view of the construction site (last phase) on Viale Regina Margherita.

FIGURE 4 The final result of the works with vehicles operating revenue service on Viale Regina Margherita.

line. At each position, three measurements were made, 7, 10, and 13 m (7.7, 11, and 14 yd) away from the tracks (nine measurement points in all). Since measurements were made before and after the works, a total of 18 measurements were taken. The building are 13 m (14 yd) away from the tracks, so the 13 m measurement gives a very good estimation of the vibration level inside the buildings.

The results of the measurement campaign are shown in Figure 5. In the Y-axis the level L_5 is shown, which is the level exceeded by the modulus of the vertical and transverse vibration for no more than 5% of the measuring time.

The results show excellent vibration reduction. The accelerometer closest to the buildings shows an average value for L_5 of 76.3 dB after works, versus 83.7 dB before works, which is approximately a 60% reduction.

SITE 2—PIAZZA VITTORIO EMANUELE IN ROME

Until last year, one of the most important open-air markets in Rome was situated in Piazza Vittorio Emanuele, a big square very close to Termini Station, the main railway station in Rome. Now, the market has moved and the square has gone back to its old look, a crowded place with a park in the middle.

The square underwent heavy reconstruction, which included substitution of the floating mass system for the traditional track structure system.

The site during the track substitution is shown in Figure 6 and Figure 7.

The before works and after works measurements have been taken at ground level, during tram traffic, at three different points along the tracks, 1.5 m (1.6 yd.) away from the tracks. Two accelerometers were used, one for vertical vibrations and another for transverse vibrations.

As for Site 1, the measurement conditions were the same, before and after the works (same vehicles, same acceleration and velocity), and the before-works rails did not show any particular signs of wear or roughness on the running surface.

The results of the measurement campaign are shown in Figure 8. In the Y-axis the average levels for vertical and transverse acceleration are shown, both before works and after works.

Also in this case, the results show excellent vibration reduction—about 12.5 dB for vertical acceleration and 14 dB for transverse acceleration, which is approximately a 75% to 80% reduction in both cases.

The average frequency spectra of the measured signals are shown in Figure 9. A constant decrease can be seen, throughout the whole spectrum. Moreover, a slight shift towards low frequencies is present, due to the increased mass of the track system.

CONCLUSIONS

Comparison of the floating mass system and the floating platform system is summarized in Table 1.

Both systems exhibit excellent vibration abatement, as the before works and after works measurements presented in the paper have shown.

In Table 1 information is given about the cost of the systems and the cost differential with respect to traditional track structures.

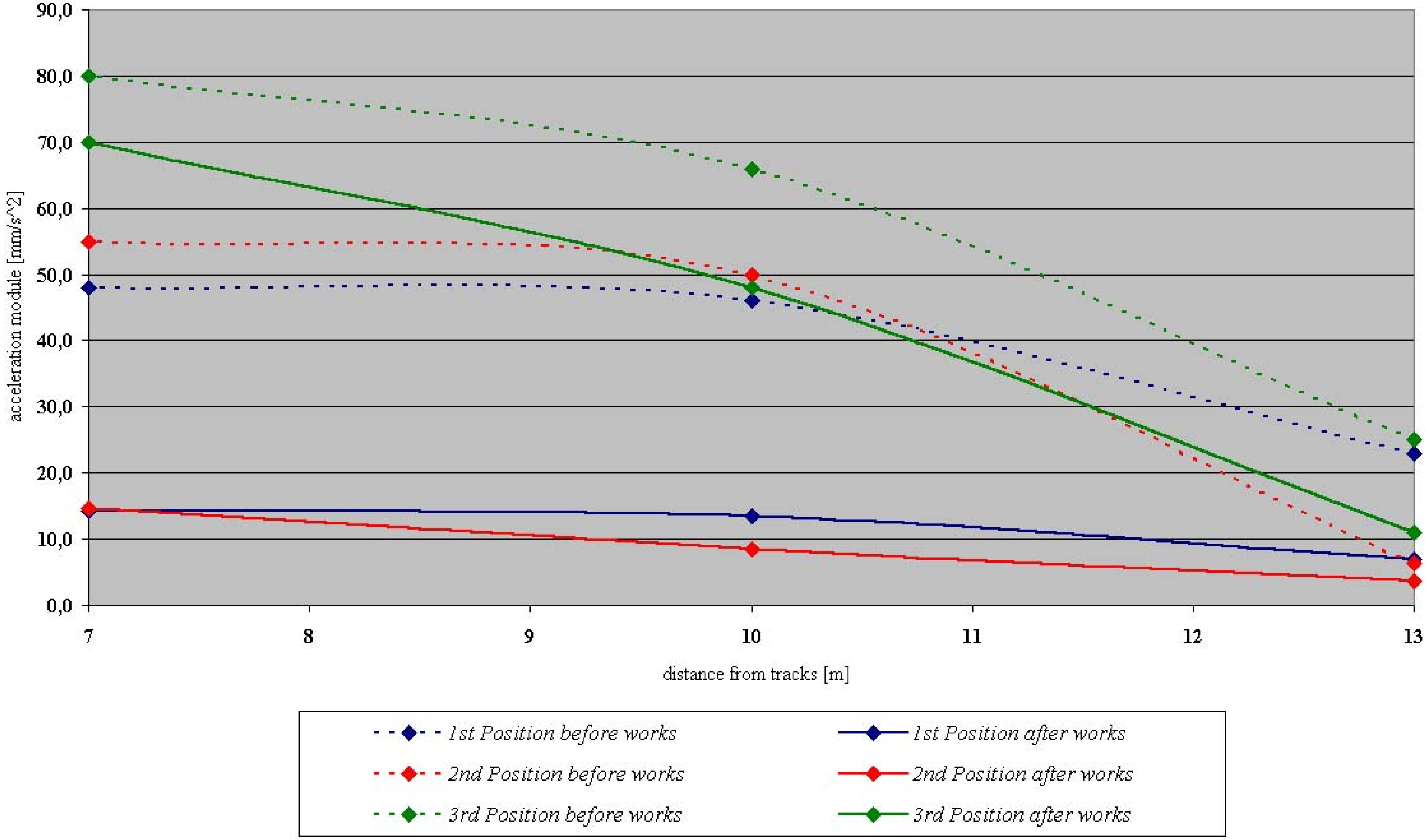


FIGURE 5 Vibration measurement results in Viale Regina Margherita.

FIGURE 6 Piazza Vittorio Emanuele under construction. On the left, the first layer of concrete has been laid. On the right, the second layer of concrete has been laid, up to the top level of the concrete sleepers.

FIGURE 7 Piazza Vittorio Emanuele under construction.

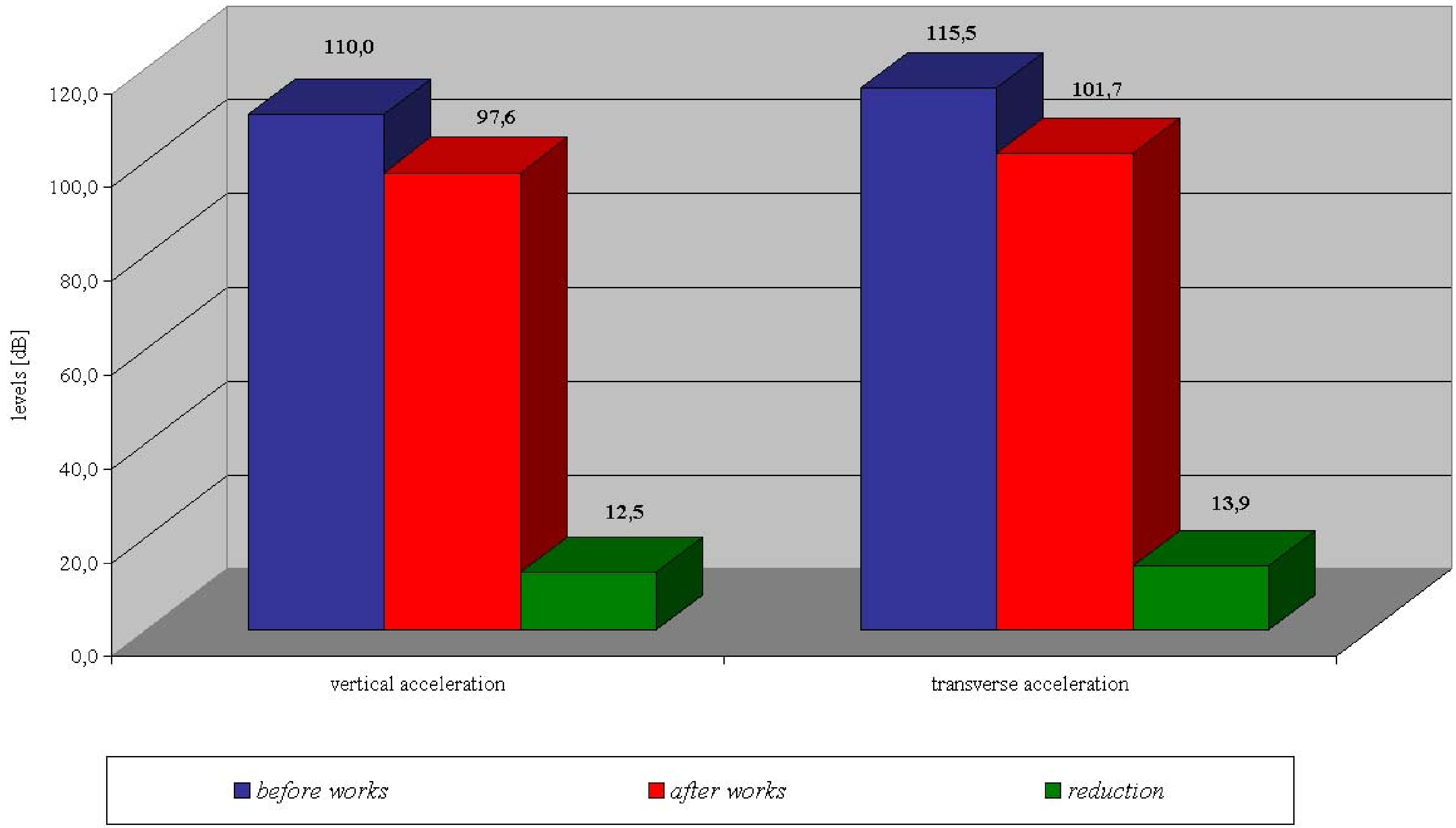


FIGURE 8 Vibration measurement results in Piazza Vittorio Emanuele.

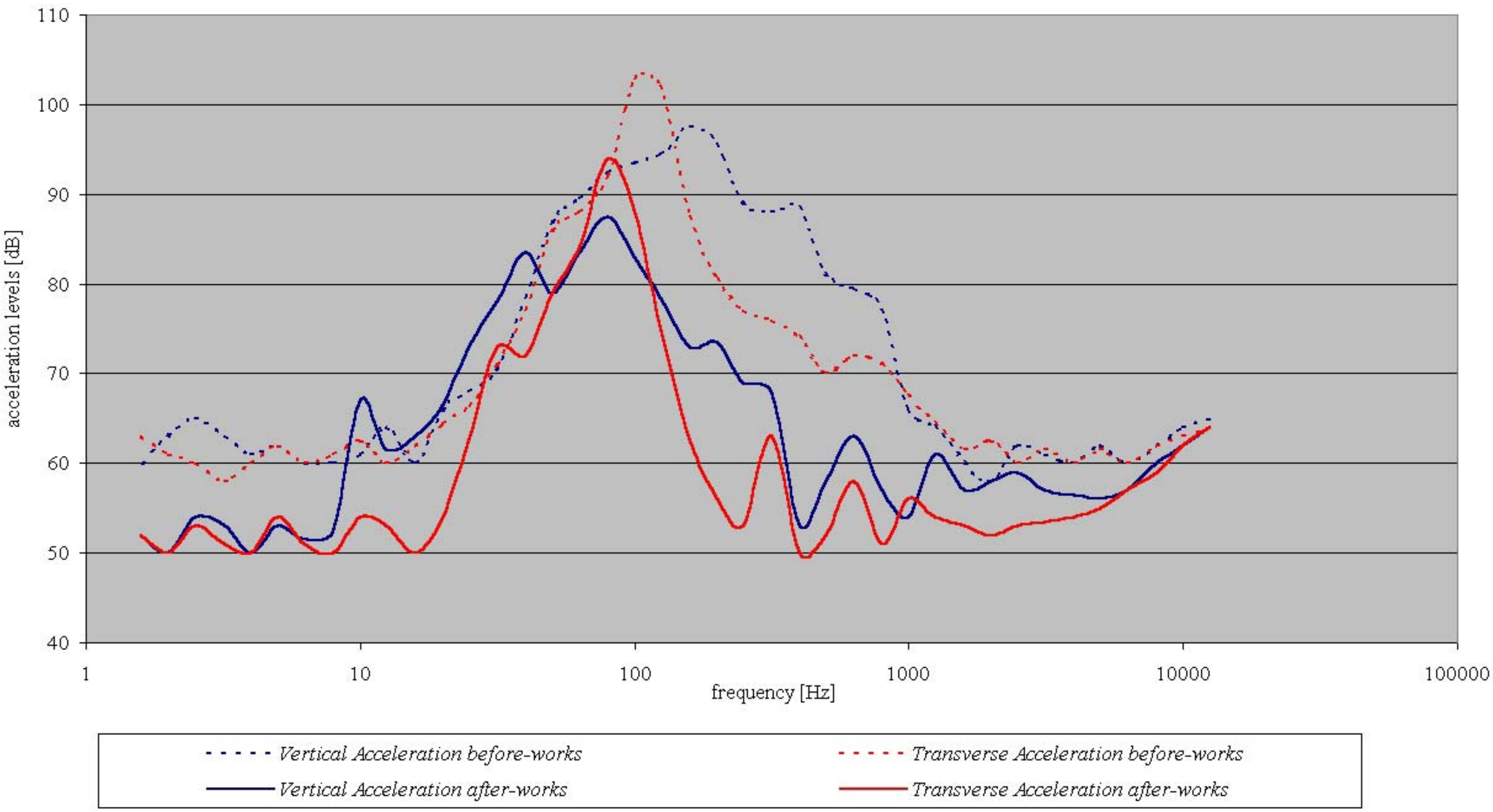
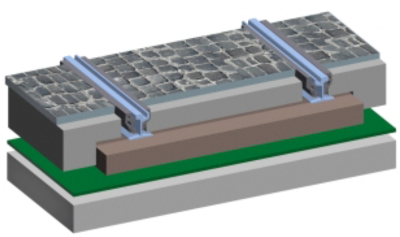
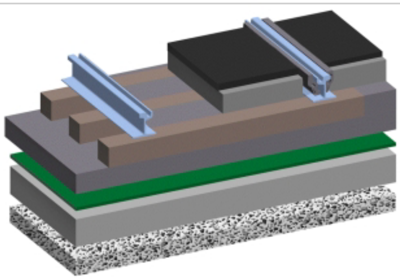


FIGURE 9 Vibration frequency analysis results in Piazza Vittorio Emanuele.

TABLE 1 Comparison Table

ANTIVIBRATION SYSTEMS	TYPICAL VIBRATION REDUCTION	COST (*)	COST RISE ON TRADITIONAL SYSTEM (**)
	dB	€x1000 per s.t.m.	%
FLOATING PLATFORM SYSTEM 	11	2,5	78
FLOATING MASS SYSTEM 	14	1,9	36

(*) <u>s.t.m.</u>: single track meter; (**) <u>Traditional track structure cost</u>: 1,4 €x1000 per s.t.m.

FIGURE 10 Piazza di Torre Argentina: tracks have been laid upon the precast concrete platform near Roman monuments dating back to the 3rd century B.C.

FIGURE 11 Side and central precast concrete slabs have been laid and joined to the precast concrete platform in Piazza di Torre Argentina.

Even if the cost differential is considerable, it should be borne in mind that usually vibration reducing track structures are needed just for relatively short sections of an LRT line, so the incremental cost may be acceptable.

The higher cost of the floating platform system is due to the precast concrete platforms and slabs, but this cost is compensated by shorter construction time and easier maintenance operations on tracks.

Tramway line number 8 is shown in Figure 10 and Figure 11, which is the most recent tramway line built in Rome (opened to revenue service in 1998, while actual construction ended in 2000). One of the terminals of the line is in Piazza di Torre Argentina, the heart of the historical center, where Roman monuments dating back to the 3rd century B.C. and historical buildings are located. There, a floating platform system was adopted, including an elegant block pavement on top of the track structure.

In March 2003, the preliminary phase of project design for the extension of line number 8 was concluded. Within the next 3 years, line number 8 will be extended to Termini Station, the main central railway station in Rome, crossing Piazza Venezia and Via Nazionale, doubtless among the most important places in the historical center.

Along the whole extension, about 1.6 km (1 mi) long, the floating platform system will be adopted, which will also minimize construction time.

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CIVIL DESIGN

Embedded Track Design and Performance

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The industry is gravitating to concrete slabs for embedded track designs. Designs with concrete have proven economies in construction and long-term performance. Yet there are concerns, such as the following:

- Structural codes state that the codes do not apply to slabs on grade that are intended as the principal structural support.
- Agencies press for minimum slab thickness to minimize conflicts with existing utilities.
- Configurations to meet criteria for stray current control and ground vibration control may effect track integrity.
- Design factors for future utility trenching around embedded track are inherently uncertain, potentially governing a design, or, if ignored, jeopardizing long-term track performance.
- The effects of elastomers on the slab performance have little research.
- Embedded track is among the least maintenance friendly. Improvements in design analysis will allow confidence in new low maintenance concepts.

This paper provides embedded track analysis methodology and results that are the basis for engineering decisions on, and increased confidence in, long-term embedded track performance. The analysis method treats the rail and support slab as two continuous beams interacting through an intermediate pad, with the support slab on a continuous elastic foundation (soil). The method produces deflections, moments, and stresses independently for the rail and support slab, and pressures in the intermediate pad and supporting soil. This information allows long-term maintenance assessments (slab life, required soil load capacity, and pad criteria) of a design.

The methodology is explained and the results for the practical range of embedded track configurations are presented.

BACKGROUND

The purpose of this paper is to present analysis methodology and illustrative results for embedded track, and to identify criteria or guidelines useful for assessing the integrity of an embedded track design.

A central interest is in assessing performance differences between different embedded track arrangements including those with and without elastomeric elements between rail and slab.

The development of the analysis methodology established the following goals for the modeling:

- To reflect the individual behavior of the rail and the slab,
- To incorporate methods for realistic slab support (foundation modulus for the soils),

- To assess performance under service (fatigue, life expectancy), and
- To allow single axle or multiple axle loading and thermal rail loading.

The model uses Beam-On-Elastic-Foundation (BOEF) theory.
This report treats the material in the following order:

- Track configurations and conditions
- Description of methodology
- Illustration of methodology results
- Summary

TRACK CONFIGURATIONS AND CONDITIONS

The general embedded track configurations are

1. Rail fully embedded in concrete (Figure 1), and
2. Rail embedded in a non-structural material supported by structural concrete slab (Figure 2).

These basic configurations can be used to model any practical embedded track form (i.e., configurations) using one of these basic configurations, including track continuously in-street, grade crossings, or track at-grade (above ground slabs).

The analysis is specifically for track that contains concrete (other than concrete ties) as a structural support or as a fill material between the rails. Any other embedded track (e.g., ballasted track with asphalt in-fill) is designed simply using conventional ballasted track engineering, and is outside this discussion.

Configuration 1: Rail Fully Embedded

The first configuration, rail fully embedded, is the case where there is a base concrete slab and concrete fill to the top of rail (Figure 1).

This configuration includes construction that is poured as a monolith (single pour from base of slab to top of rail), or sequential pours (base slab poured first, followed by one or more top pours). Also included within this configuration are trough track (troughs for rail are cast in, with rail, fasteners and trough infill material placed after the trough is complete), any form of direct fixation track where concrete is the infill material between limits of the fasteners (or covering the fasteners and rail, except rail head). Some slab designers prefer reinforcement bar within or into the rail infill concrete, particularly if there are multiple pours; such additions of rebar are consistent with the assumptions in the analysis, but have no effect on analysis results.

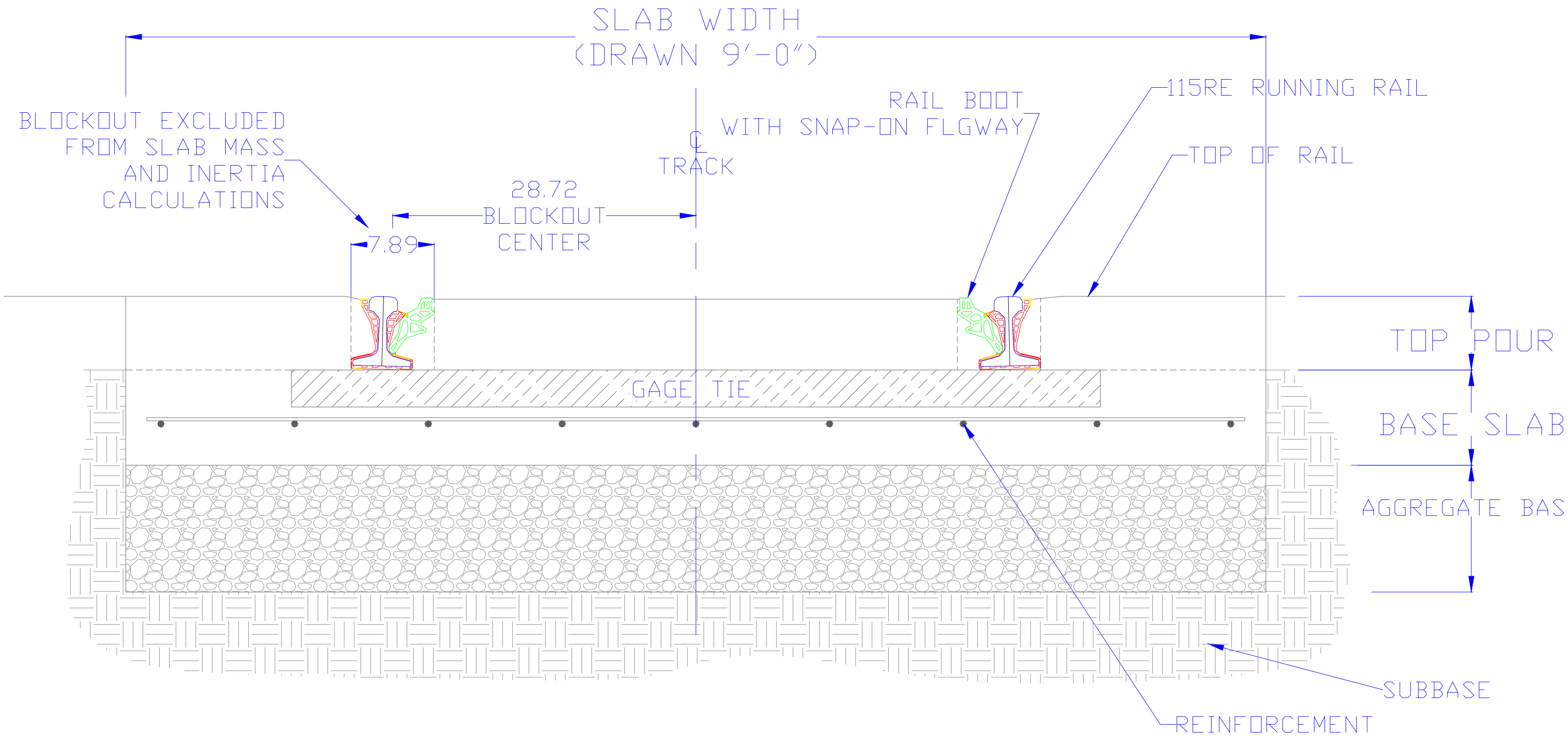

FIGURE 1 Configuration 1: Rail fully embedded in concrete (illustrated with Rail Boot).

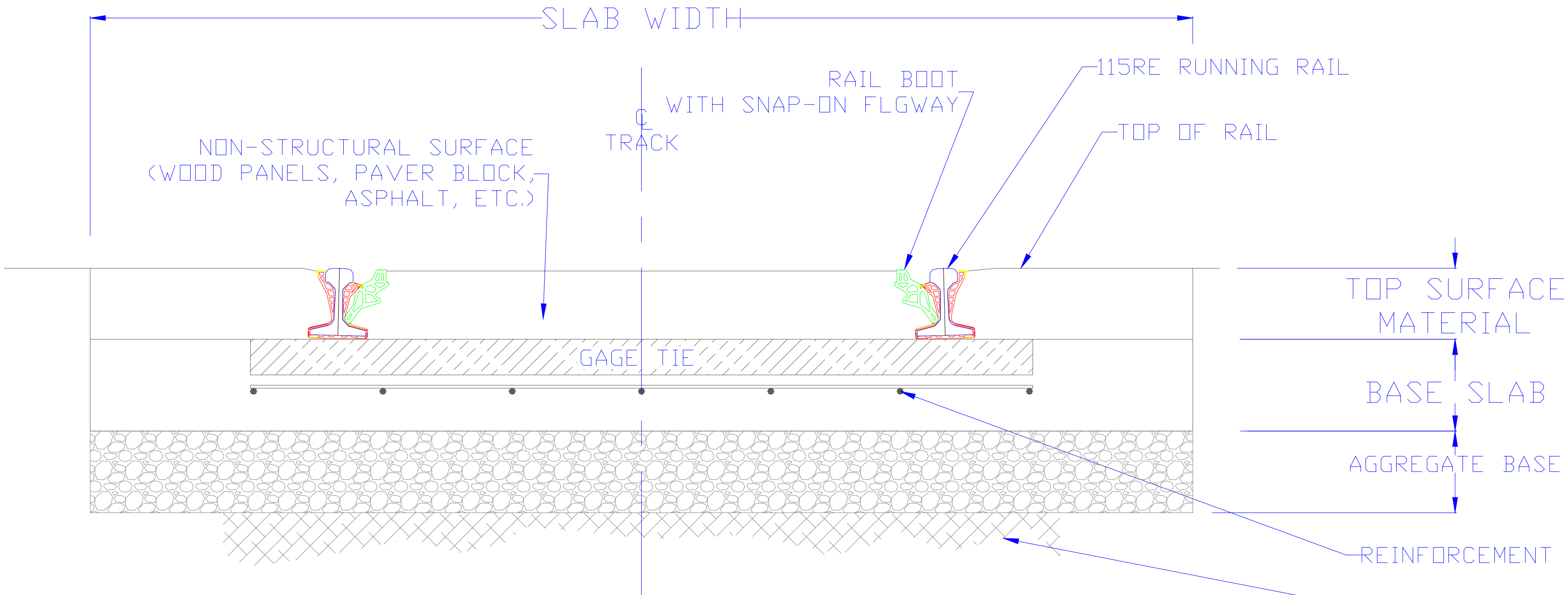

FIGURE 2 Configuration 2: Rail embedded in a non-structural material supported by concrete slab (illustrated with Rail Boot).

Configuration 2: Rail on Concrete Slab

The second general configuration is that of a concrete slab supporting the rail, with or without non-structural infill or top pour material (Figure 2).

With this configuration, infill material between rails is considered non-structural, meaning that it has no consequence in supporting rail loads other than adding dead weight to the structural supporting slab. This configuration includes any material as the “top pour” (from top of support slab to top of rail), such as asphalt paving, paver blocks, lightweight concrete, or paneling of any type. Any rail fastener arrangement is acceptable.

BEAM-ON-ELASTIC-FOUNDATION ANALYSIS PROCEDURES

Analysis Method and Idealizations

The methodology implemented in this paper is the BOEF theory generally used to analyze track, except this method allows two beams (the rail and slab) where the traditional analysis allows only one beam (the rail). This method, developed by Hetényi (*1*), allows assessment of the rail and slab interaction through a continuous series of springs between the rail and slab.

These intermediate springs represent rail pads, rail fasteners, or any membrane (such as Rail Boot) that have measurable stiffness (spring constants). The model can be used to represent any intermediate material, as well as the case with no intermediate material (a very stiff spring). An elastic material (the soil or aggregate) supports the slab in the model. These idealizations, illustrated in Figures 3 and 4, can represent any continuous track, including floating slab track as well as embedded track.

This analysis is linear, which means that this analysis allows cumulative addition of individual effects, such as two adjacent axles on a truck where the effects of each axle are superimposed.

The model considers wheel loads as “point” loads, and loads from rail thermal expansion and contraction are “distributed” loads, placing a load continuously along the length of the slab. The effects of distributed loads can be superimposed on the wheel load results in this model implementation.

Parameters in Analysis

The model includes a fairly complete set of influence parameters including vehicle parameters, train operation (braking, speed, traffic density), soil characteristics, track geometry (horizontal and vertical curves), environmental (thermal), structural parameters (slab geometry, concrete material design properties, reinforcement type, and configurations), rail properties, and elastomer properties for the rail support.

Slab Structural Analysis Method

The structural analysis procedures applied to model results are in accordance with applicable American Concrete Institute Code (*2*).

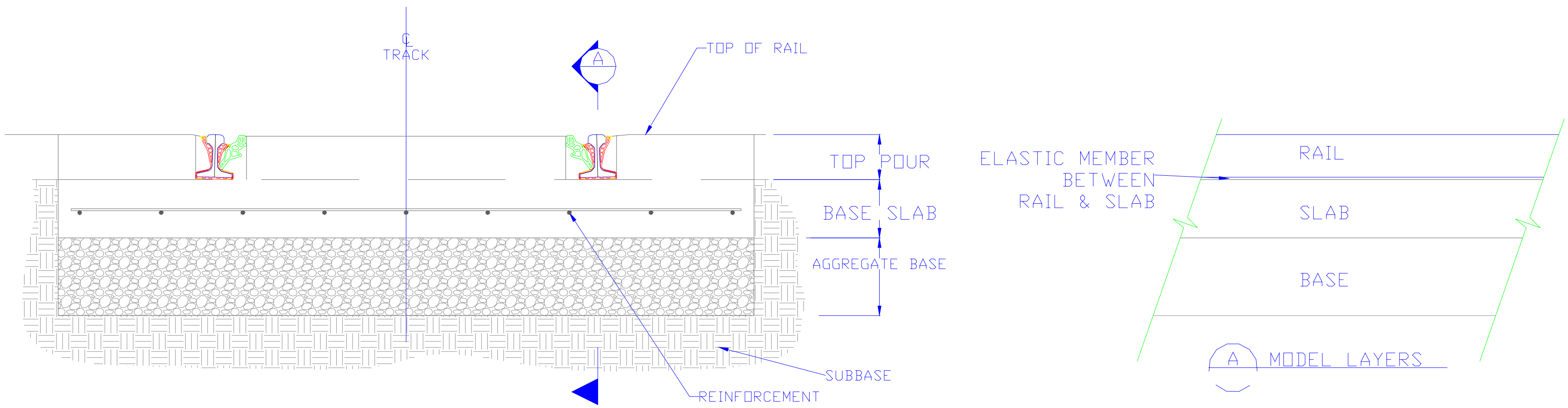

FIGURE 3 Track section and associated model elements.

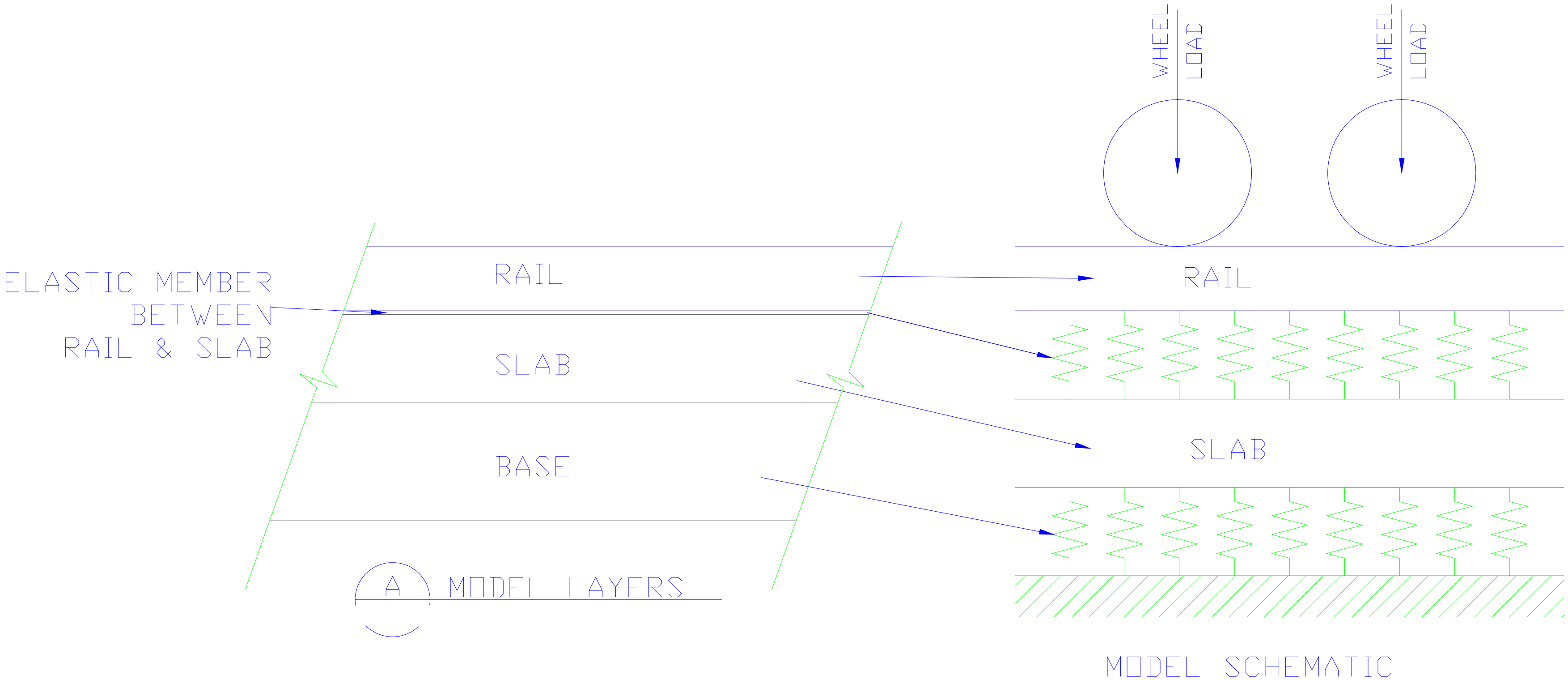

FIGURE 4 Idealizations for modeling.

Fatigue Calculations

Fatigue is calculated as slab life in years of service. Fatigue is calculated at the design load, the nominal wheel load multiplied by a design factor (typically 2). Fatigue for slabs under special trackwork (frogs, switches, and rail crossings) uses the full design load. This reflects a notion that loads are generally higher as wheels cross frog flangeways and flangeways in rail crossings.

Outside special trackwork, the track slabs will endure infrequent loadings at the full design load. The procedure incorporates a "load distribution factor," a percentage of the design load, that may be used for fatigue analysis of normal embedded track.

Fatigue is calculated by methods developed by the Portland Cement Association (*3*).

Base Support Stiffness

The support stiffness of the base soils, gravel, or other material directly under the slab is critical in embedded track analysis.

The considerable literature on soils and foundations lacks data in terms required by embedded track analysis. The design guideline (ACI 360, Design of Slabs on Grade) requires field measurements to obtain the required modulus, not entirely useful for preliminary assessments, or for parametric studies of embedded track configurations. This requirement can hinder new track design processes unnecessarily because the track configuration is central to the design development of all other facilities associated with a railroad or transit. In reality, the urban environment provides new embedded track with engineered base materials (streets, previous rail routes) and ample past borings nearby to provide information suitable for track design.

A preferred available approach is to estimate support modulus for the assumed base materials, generate track designs compatible with all criteria, then confirm the track design when geotechnical data is eventually produced. Assuming base materials and their properties has little risk not only because the urban base material is well known, but also because there is reliable consistency in properties that effect embedded track design in existing urban environments.

In the cases where embedded track may be placed other than in existing infrastructure, it is then, by definition, virgin development that necessitates knowing the requirements for the embedded track base in advance of all other project parameters. The required base will then be engineered to meet track base requirements.

The available method for calculating a reasonable support modulus for a variety of circumstances is provided by Richart et al. (*4*) using straightforward selection of the soil type and the geometry of the slab and base course. The method's authors developed a series of curves from tests relating soil shear modulus, soil void ratios, shear wave velocity, soil grain type, slab dimensions, and base course thickness to spring rates (foundation modulus). The method allows consideration of confining pressure (the pressure from adjacent soils on the base material when it deforms under load), important in embedded track applications. The method provides results for vertical spring rates, horizontal spring rates, and rocking spring rates (the stiffness against slab twist about its longitudinal axis).

The Richart et al. data and methodology are implemented in this analysis for base support stiffness.

Results Available from Analysis

The results of the BOEF analysis are estimates of the rail and, separately, the slab deflections, moments, and shear force. The results also include pressures between rail, rail support (elastomer, Rail Boot, rail pad, Direct Fixation fastener), slab, and slab support (gravel or soil base).

The analysis uses these fundamental results to calculate rail and slab stresses and strains, which in turn are used for fatigue life estimates.

The analysis also provides other useful values such as slab safety factors and allowable stresses, and estimates of rail and slab natural frequencies, as design or evaluation aides.

The analysis provides structural results for reinforced slabs and non-reinforced slabs, and calculates slab reinforcement for crack control.

BOEF Analysis Results

This section presents results from the BOEF analysis.

These results illustrate trends in slab reactions (deflections, moments, and stress) and performance (fatigue life). This demonstration explores these results for the following parameter ranges:

- Slab configurations: full slab (Configuration 1) and base slab (Configuration 2)
- Rail support stiffness (elastic property of the rail pad, Rail Boot, or Direct Fixation fastener): 100,000 lb/in. to 3,000,000 lb/in.
- Slab thickness below the rail: 6 in. to 20 in.

For perspective, Rail Boot static stiffness is about 400,000 lb/in., Direct Fixation fasteners typically have a stiffness between 100,000 and 200,000 lb/in., and rail pads generally have stiffness values between 750,000 and 3,000,000 lb/in.

All other parameters in the model are held constant (see Table 1) to allow a direct comparison of results, although a number of parameters such as soils, temperature variants, curvature, and so on would be adjusted in practice for particular circumstances.

The loading and vehicle parameters are typical of a high-floor North American light rail vehicle (LRV). The trends in these results are also indicative of that expected for heavy rail vehicle loading, because the heavy rail vehicle weights and capacities are within 20% of those for LRVs, and heavy rail higher speeds are insufficiently different to effect these types of analysis.

All results are from calculations of both single axle and double axle loads, where the higher value from either is used when appropriate for each parameter explored. Double axle loads are those from two adjacent wheels representing a single truck.

Track Modulus and Support Stiffness

This is a brief aside to clarify the physical meaning of track modulus and support stiffness, and how those apply in this analysis. A support stiffness and its associated foundation modulus are directly related but different.

TABLE 1 Parameters and Values Held Constant

Parameter	Assumed Value
Wheel Load	17,000 lb
Load Factor (design safety factor)	2
Curve	200 ft radius
Track Superelevation	2 in.
Design Wheel Load (includes load factor, curve forces, etc.)	36,302 lb
Vertical Load Reduction Factor (for normal track fatigue analysis)	95% of Design Wheel Load
Rail	115 RE
Track Gage	56.5 in.
Train Speed	25 mph
Vehicle Axle Spacing	72 in.
Axles per Truck	2
Maximum Vehicle Brake Rate	3 mphps
Annual Traffic Volume	355,300 Axle loads per year
Slab Concrete Strength	4,000 psi
Slab Reinforcement	2 layers of #6 rebar at 12 in. spacing
Base Course (slab support material)	12 in. thick gravel

The physical measure of an elastomeric material is its stiffness, a simple spring rate obtained from a measured load deflection curve. The modeling uses a related, but different value called the foundation modulus to represent the idealized series of springs.

Also, where traditional BOEF calculations have a single track modulus, this analysis has two, which are more correctly called the rail foundation modulus (between the rail and slab) and slab foundation modulus (the material supporting the slab).

In this paper, the term support stiffness refers to spring rate unless specifically qualified.

Slab Life Expectancy

Life expectancy is the most useful indicator of embedded track slab performance because it intuitively provides a sense of track degradation processes that escape clear definition in other terms.

The life estimates are those earlier referenced portland cement concrete methods that depict concrete (as well as other roadway materials) cracking, and loss of useful structural integrity. The life estimates presented here are the predicted life to slab replacement. The life estimates apply equally to reinforced and non-reinforced concrete.

Normal, Non-Special Trackwork, Track on Base Slab (Configuration 2) Life expectancy is presented first in Figure 5 for the most common embedded track configuration: normal track (any track outside special trackwork) installed on a base slab. This is Configuration 2 (see the section on Track Configurations and Conditions). Recalling the description of Configuration 2, the material surrounding and between the rails is not considered as contributing to the structural support, consisting of asphalt, paver blocks or road crossing panels. In this configuration, the rail is supported by a rail pad, a Direct Fixation fastener, or is surrounded by the Rail Boot.

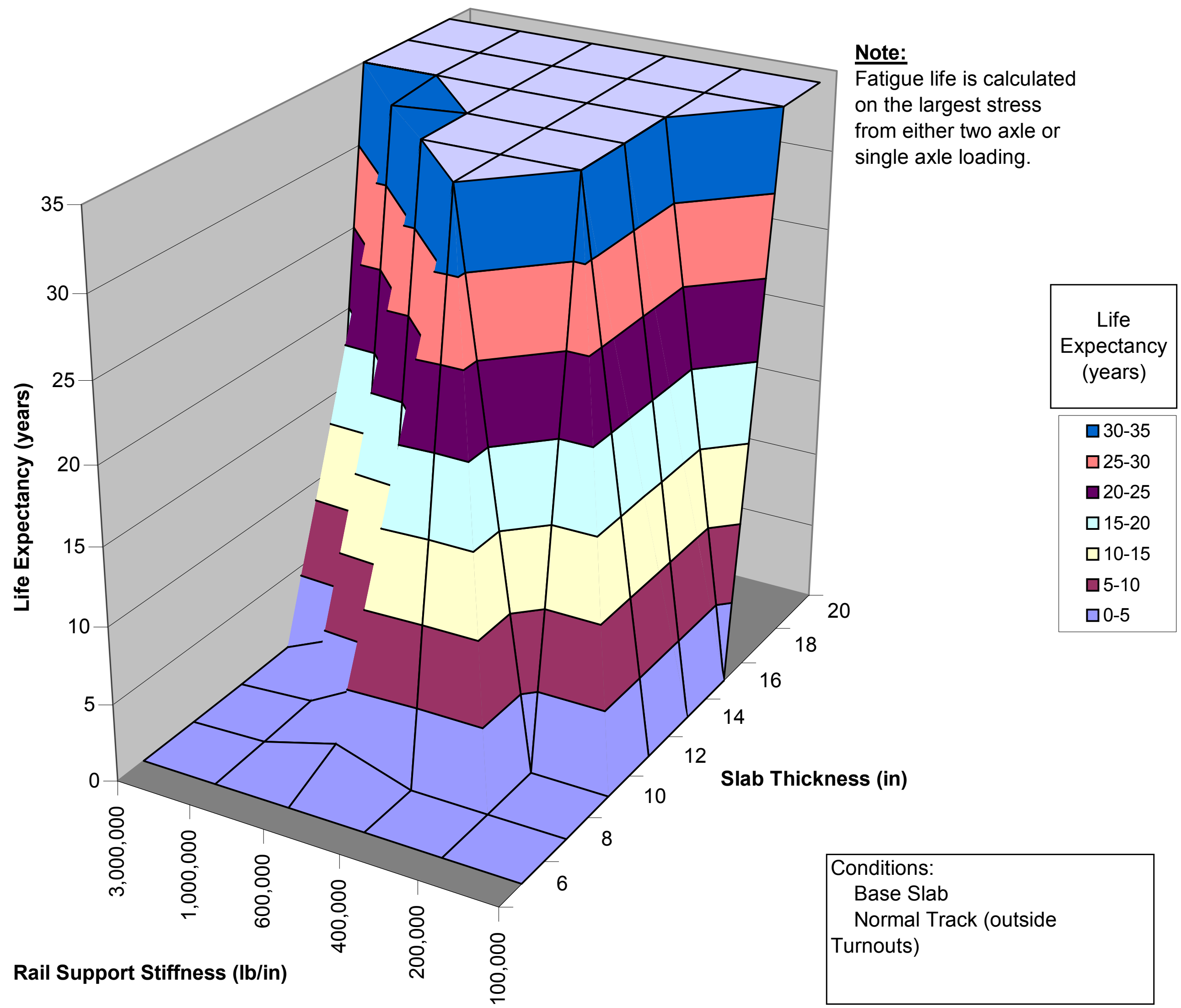

FIGURE 5 Life expectancy for normal embedded track on a base slab, only (Configuration 2).

Figure 5 assumes all wheels will produce a load that is 95% of the design wheel load. The idea is that actual wheel load populations will be lower than the design wheel load, with only incidental occurrences near the design wheel load from derailments or severely flat wheels. The 5% reduction in wheel load is very conservative, where the actual population of fatigue loads would be expected to distribute the static vehicle load (wheel load is 17,000 lb in this case, or less than half the design wheel load).

This reduced loading is applied to the fatigue calculation only. The structural calculations use the full design wheel load.

This and following charts are truncated at 35-year life expectancy, representing infinite slab life for practical purposes.

Figure 5 shows that life expectancy increases with the base slab thickness in normal (non-special trackwork) track.

Insights from Figure 5 include

- Rail Support Stiffness between 200,000 lb/in. and 600,000 lb/in. improves life expectancy for slabs less than 16 in. thick, compared to softer or harder rail support stiffnesses.
- Embedded track with rail support stiffness of 1,000,000 lb/in. or greater require a minimum base slab thickness of 14 in. to have any life expectancy, and 16 in. thickness to achieve reasonable life expectancy.

Special Trackwork on Base Slab (Configuration 2) The next example in Figure 6 is for special trackwork loading on a base slab (Configuration 2). The only difference between Figures 5 and 6 is that Figure 6 assumes all wheel loads are at the design wheel load, whereas Figure 5 assumes the wheel loads are 95% of the design wheel load. This assumption reflects a belief that special trackwork will eventually, if not initially, produce increased loads on the embedded track, and the increased loads will approximate a full impact load (double the static load).

Figure 6 shows that

- Thicker slabs are required under special trackwork for equal life expectancy of normal track with the same loading.
- The minimum base slab thickness under special trackwork should be 16 in. to achieve reasonable life expectancy.
- Rail support stiffness has little influence on life expectancy for turnout loads.

Normal, Non-Special Trackwork, Track, Full Slab (Configuration 1) The increased strength of a full slab is beneficial to life expectancy, as would be expected, with all but the thinnest slabs (6 in.) having infinite life for all rail support stiffness values.

Special Trackwork, Full Slab (Configuration 1) Analysis results for special trackwork assumptions (full impact load) with a full slab indicate that the minimum slab thickness under the rail for a full slab should be at least 12 in. where the base slab requires a minimum of 16 in.

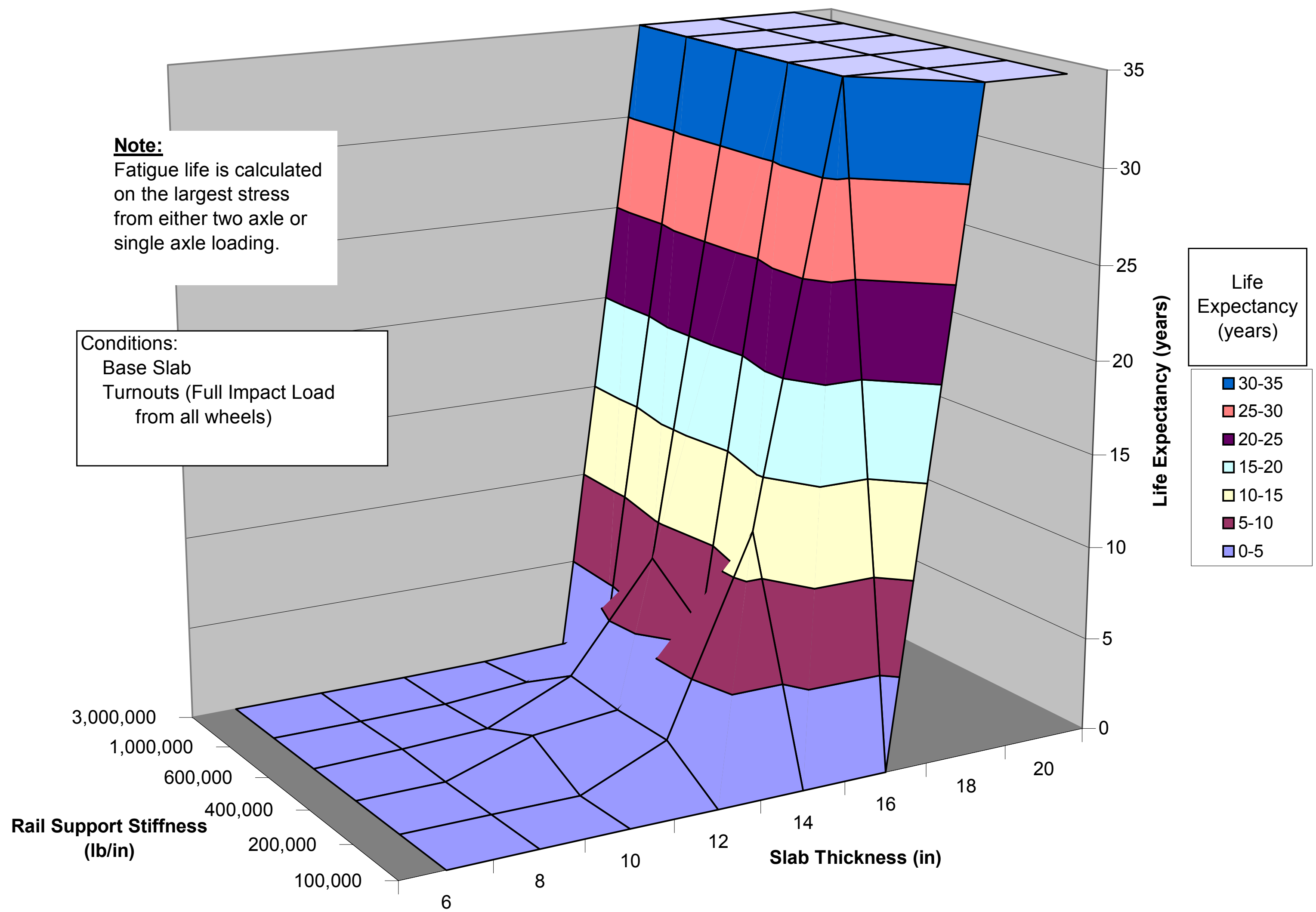

FIGURE 6 Life expectancy for special trackwork on a base slab, only (Configuration 2).

Fatigue Life and Slab Structural Design

The fatigue analysis applies equally to reinforced and non-reinforced slabs, and is not influenced by structural details of reinforcement.

The fatigue calculation and slab design calculations both use the slab maximum stress from the BOEF analysis. This means that the analysis selects the maximum bending moment (from which stress is calculated) from single-axle loading or two-axle loading, whichever produces the higher stress.

The fatigue life and the slab structural design are based on the same basic parameters, bending moment and stress, but are calculated independently. This approach identifies those designs that meet required criteria (safety factors, etc.) but have an undesirable life expectancy. In the foregoing fatigue life presentation, the configurations that have unsatisfactory life expectancies meet required criteria.

The separate calculation of fatigue life and structural safety factor allow the possibility that an increased structural safety factor may not result in a commensurate extension in fatigue life.

Deflections, Moments, Shear Force, and Pressures

This section discusses response to loading.

The analysis shows cases where the peak or maximum rail deflections from two axles are less than from one axle, even though two axles obviously have twice the load. The second axle cancels a portion of the rail and slab bending, thereby reducing the deflections produced by a single axle. The stress in the slab is similarly reduced. This effect can be significant, depending on parameter values, with a 10% to 15% decrease in two-axle deflections and stress from that of a single axle.

This effect is more pronounced as the rail support stiffness is reduced below 1,000,000 lb/in. In other words, as the rail support becomes softer, beneficial stress reduction from two axles is greater compared to single axle deflections. As the rail support stiffness increases above 1,000,000 lb/in., the two-axle response (rail deflections, slab stress) is greater than the single-axle response because the increased rigidity defeats rail bending over the interval between axles.

These findings raise the point that the design of slab tracks must consider both single-axle and multiple-axle loading.

Although infrequent, derailments most likely will commence as a single-axle event. More frequent, wheels traversing frog points and rail crossings approach the single-axle load condition. These conditions will govern the design in many cases.

However, multiple-axle loading may govern the design where, for example, the rail support stiffness is high.

Importantly, the designer should analyze both the single axle and multiple axle cases because the specific configurations and choices of parameter values may end with either case producing the larger response, which then becomes the governing case for design.

Pressures: Rail Support

One of the unique results of this modeling method is the ability to calculate the pressure of the rail on its support (rail pad, Rail Boot, etc.), of the rail support material on the slab, and of the slab on the material under the slab.

This information is very useful in the design of elastomers for the rail support and design of the subbase layer under the slab, usually an engineered selection of gravel base course and select soils.

Figure 7 shows pressures by the rail on the rail support (rail pad, Rail Boot, etc.) for a base slab, with two axle loading.

The pressure by the rail is negative for the stiffer rail supports. The negative values mean that the slab is deflecting more than the rail. The negative values mean the rail and slab are placing the rail support material in tension.

The magnitude of these negative pressures can exceed 900 psi at the stiffest (3,000,000 lb/in.) rail support. Under this condition, any fastener holding the rail to the slab will incur significant loading because these pressures are continuous along the rail and any fasteners are necessarily discrete devices that must accommodate the fully developed pressure each side of its location. For example, the load on a pair of anchor bolts (holding the rail to the slab) is over 25,000 lb, or 12,500 lb per anchor bolt, and where anchor bolt pairs are spaced at 24 in. Slab anchor inserts typically are designed for a maximum 12,000 lb pull-out load. While typical anchor bolt configurations could be altered, it is better to use a softer rail support to avoid this condition.

Pressures: Slab Support

Slab pressures on the slab support material (gravel, soils, etc.) range from 5 to 10 psi (Figure 8), acceptable for most soil conditions. It should be kept in mind that embedded track is used most often in urban streets where there are numerous underground utilities. Utility activity over the life of the slab can include trenching beside and burrowing under the slab. This activity can cause uneven slab support if not properly back-filled. The slab must therefore have reserve structural capacity for bridging unknown future support conditions.

Slab Natural Frequency

While not a dynamic model, the information in the model allows calculation of undamped natural slab frequencies, useful for understanding qualitatively at least the relationship ground vibration created from train vibrations. The slab structure will filter train vibrations greater than the slab's natural frequency, will amplify any vibrations near the slab's natural frequency, and will transmit all train-induced energy that occurs below the slab's natural frequency.

For a 9 ft wide slab, a full depth slab (Configuration 1) will have a natural frequency between 13 and 17 Hz, with little variation among slab thickness values. Base slab (Configuration 2) natural frequency varies from 30 Hz for 8 in. thick slabs to 17 Hz for 20 in. thick slabs.

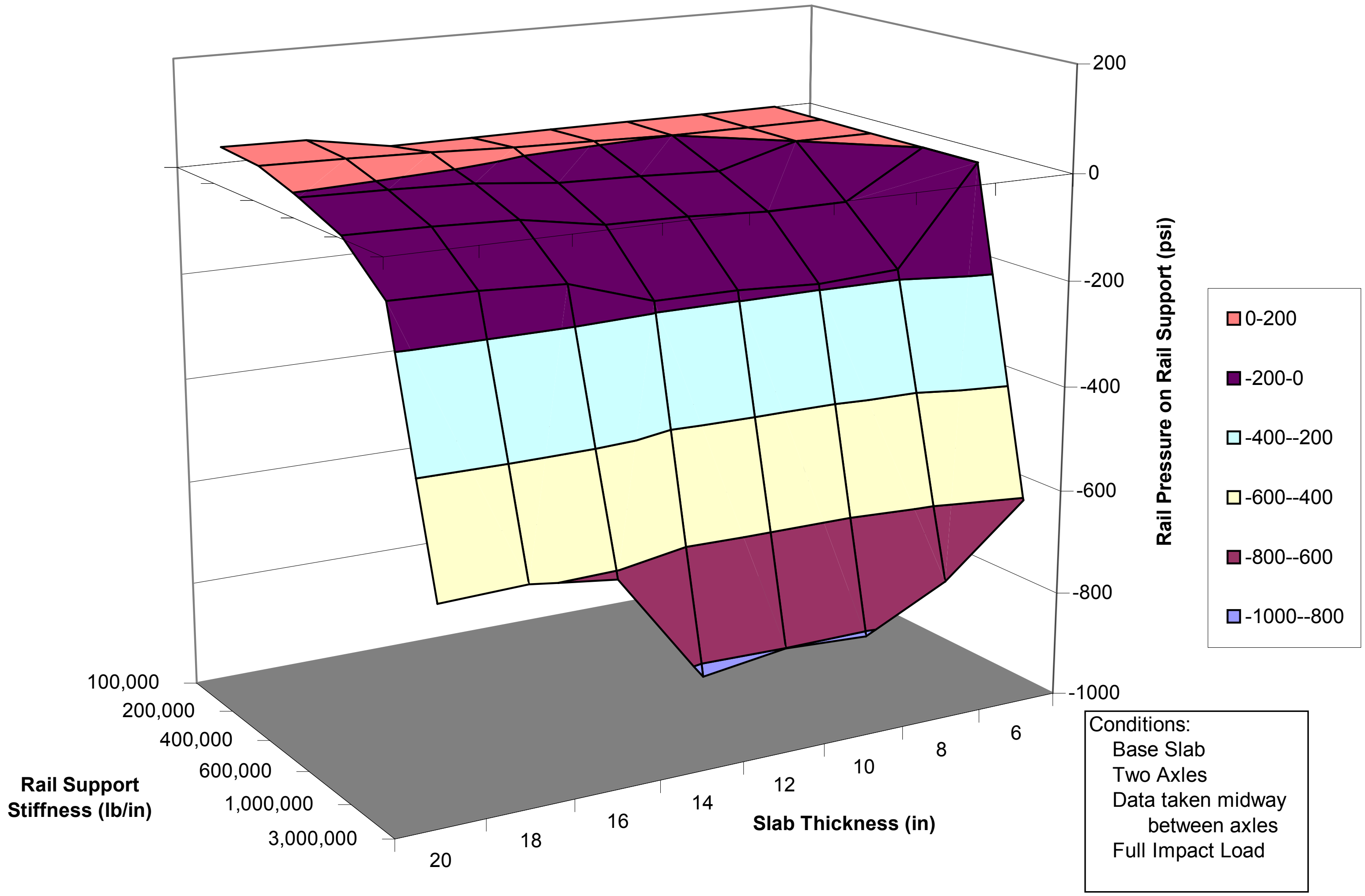

FIGURE 7 Rail pressure on rail support, base slab.

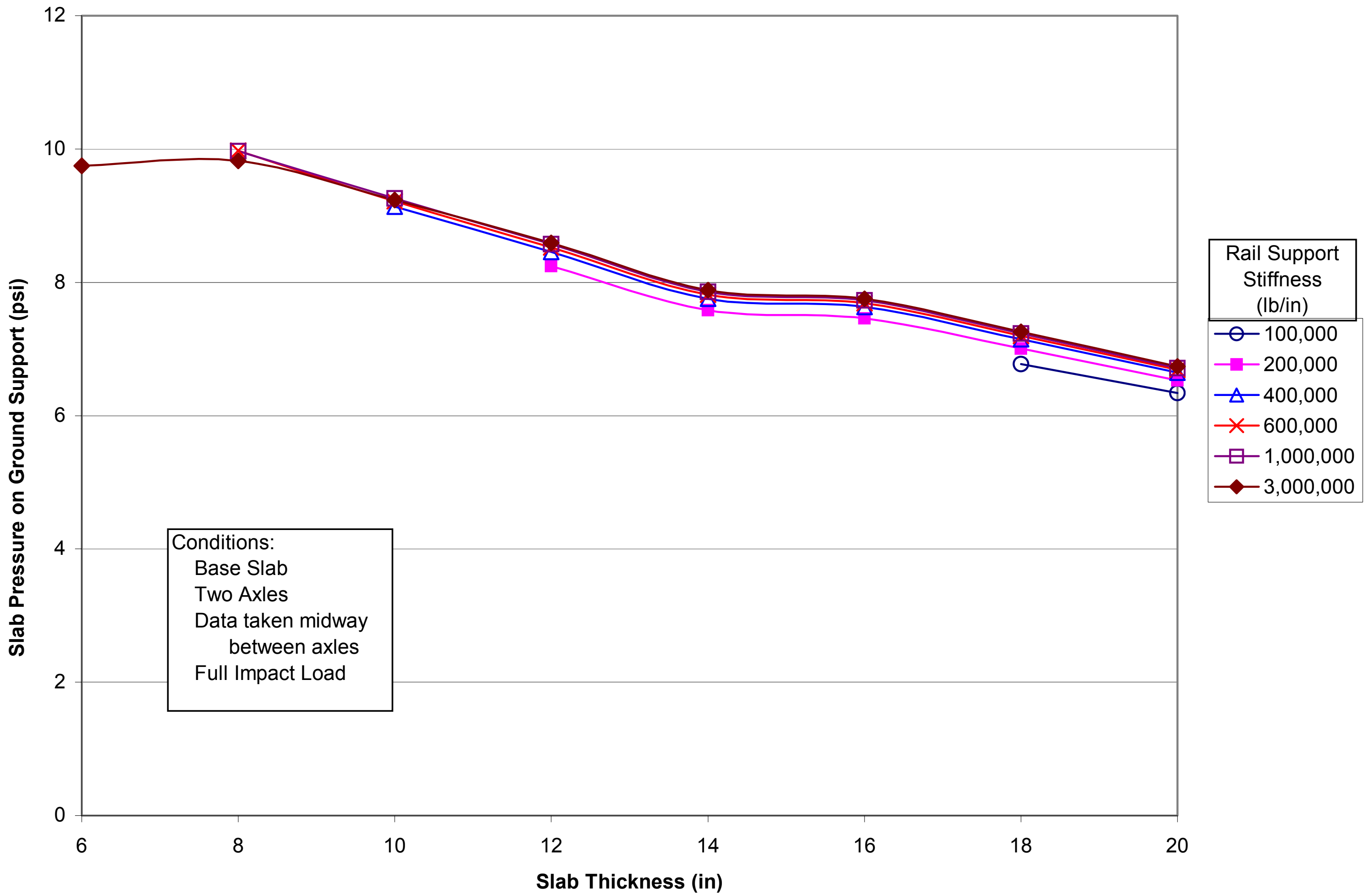

FIGURE 8 Full slab pressure on its support (gravel layer, prepared soil base).

Additional Notes on the Analysis

This subsection explains how the model treats the rail support elasticity.

In the foregoing presentation, we observed circumstances that had the rail deflecting less than the slab, meaning that the slab and rail were separating. We would have expected the rail and slab to move together, and, if anything, the rail deflect a little more into its support elastomer than the slab because the rail has more of the direct load and is a much more slender beam than the slab. This expectation is realized when rail support stiffness is 600,000 lb/in. or less.

When the elastomer stiffness approaches or exceeds 1,000,000 lb/in., the rail modulus becomes much greater than the slab support modulus, creating the circumstance for slab to deflect more than the rail.

In the latter circumstance, the rail is of course fastened to the slab or constrained by embedment concrete, thus the rail will deflect with the slab. However, this circumstance induces tensile load in rail fasteners or shear forces in constrain concrete that could cause degradation or failure.

Evidence that the rail support is too stiff would be sprung elastic rail clips, loose anchor bolts (either pulled out from concrete or loss of bolt torque), or concrete cracks (where rail is fully embedded in concrete) parallel to and within about 7 in. of the rail.

Effects of Rail Thermal Loading on Embedded Track

The analysis includes estimates of loads produced by thermal contraction and expansion of the rail.

In horizontal or vertical curves, Continuously Welded Rail thermal effects create radial loads on the rail support. The force in horizontal curves is determined by rail temperature difference from the rail neutral temperature and the curve radius. In vertical curves, the force is determined by the rail temperature difference, the change of grade through the curve and the length of the vertical curve.

This force is inversely proportional to the curve radius (i.e., the smaller the radius, the higher the force). The rail size has a lesser effect.

This force is a distributed force, meaning that the force is uniform along the length of a curve and is stated in pounds per unit rail length. Figure 9 shows thermally induced rail loads on vertical curves for the practical range of grade changes and curve lengths.

For even the most severe grade change and shortest curve length, the distributed loads are fairly low (under 140 lb/rail foot) compared to vehicle loads, assuming a 90°F rail temperature difference from the rail's neutral temperature.

Figure 10 shows the lateral rail force in horizontal curves for the practical range of curvatures and temperature differences. The horizontal loads on slabs from rail thermal effects can become significant for curves with a radius of 200 ft and less. A track assessment would consider whether this effect along with other circumstances present (wheel loads, rail pre-curving) is within the rail restraint capacity.

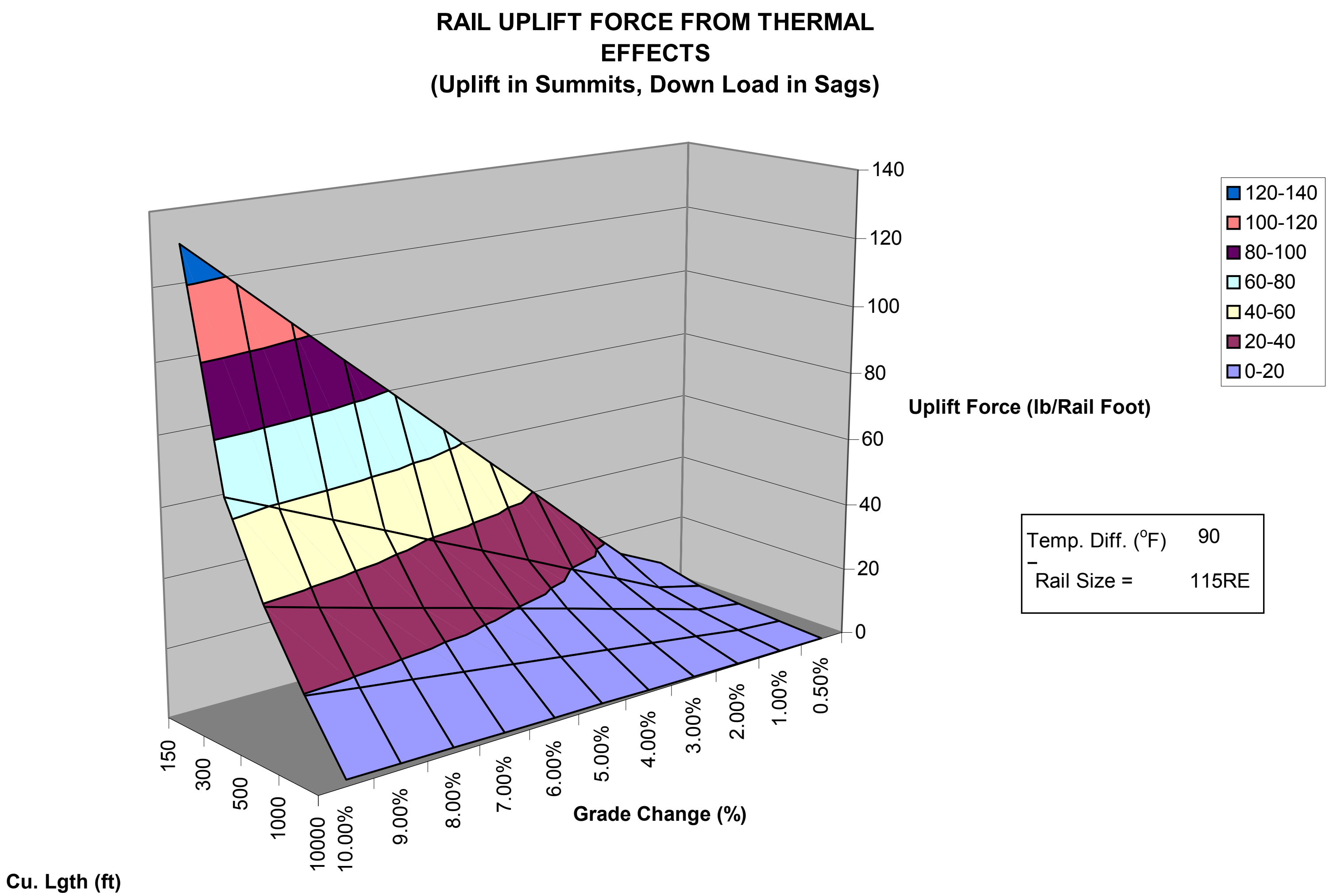

FIGURE 9 Vertical rail force on slabs in vertical curves from thermal effects.

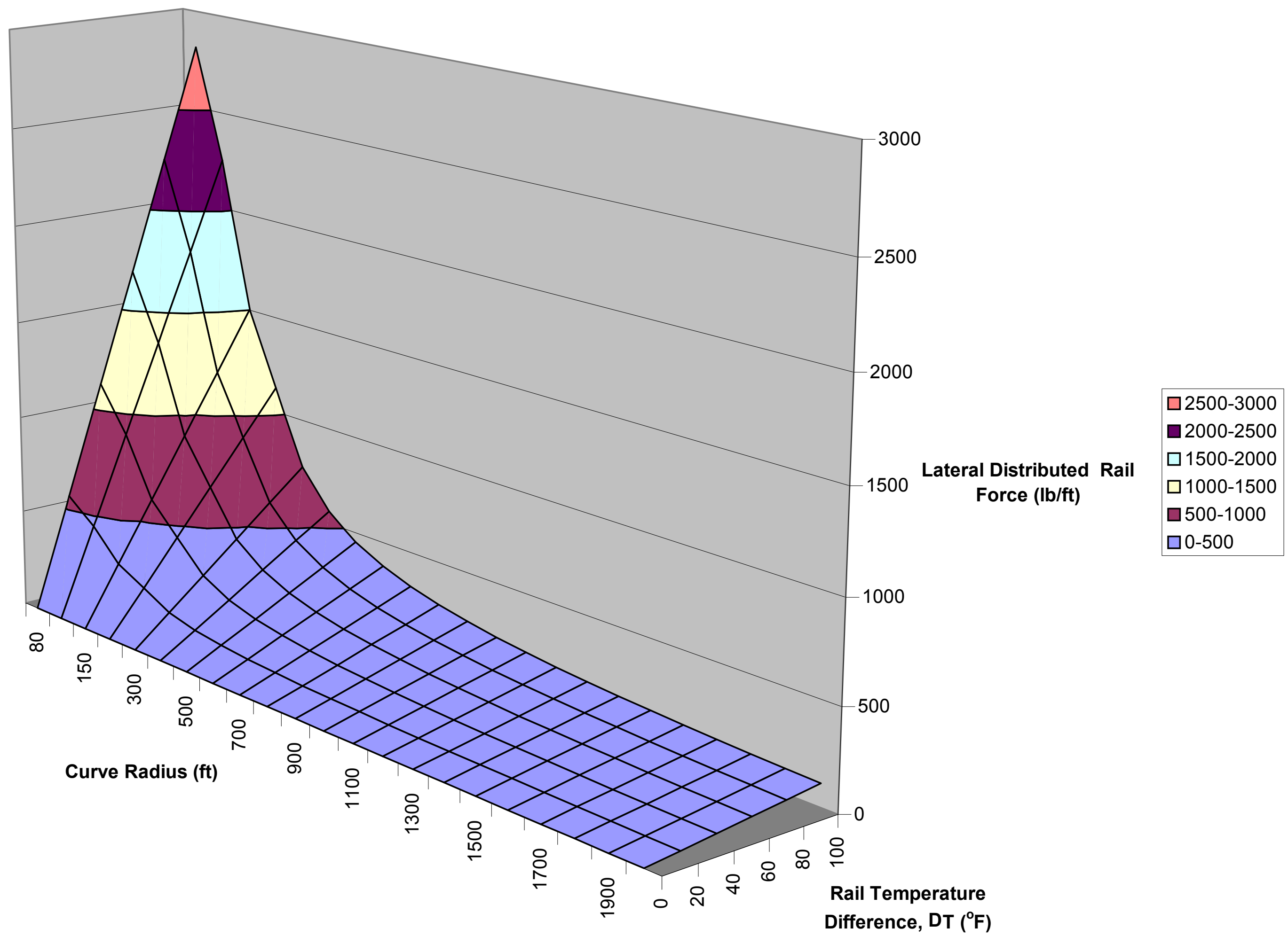

FIGURE 10 Lateral rail force on slabs in horizontal curves from thermal effects.

SUMMARY

Analysis of embedded track using a multi-layer model provides insight on performance of embedded track. The model is a static, linear representation (compared to dynamic, non-linear representation) of elasticity within the track system of rail, rail support elastomer, slab, and ground support of the slab. The analysis method incorporates subordinate methods for estimating ground support for a practical range of conditions, for determining rail thermal effects on slab loading, for determining slab natural frequencies, and for estimating track life.

The method is demonstrated for a practical range of slab thickness and rail support elastomer values. The summary of these results is

- Rail support stiffness, the spring rate (not the track modulus) between rail and slab, generally has a significant effect on slab life and stresses.
- Rail support stiffness of 400,000 to 600,000 lb/in. is the ideal range for overall slab response and performance.
- High rail support stiffness (above 1,000,000 lb/in.) creates high slab stresses requiring thicker slabs.
- Typical slabs (those with simple base support and a non-structural fill between rail) should be at least 14 in. thick in normal track and 16 in. thick in turnouts to avoid fatigue deterioration. Full depth slabs (concrete to top of rail) may be 6 in. thick in normal track and 12 in. thick in turnouts for acceptable life expectancy within the ideal rail stiffness range (above).
- Rail deflections are less than slab deflections when rail support elastomer stiffness is greater than 1,000,000 lb/in. In these circumstances, the rail will place upward force on the concrete and any rail fasteners. The upward force can exceed current fastener allowable force, or damage embedment concrete, at the stiffest elastomer values.
- Rail deflections from a single axle are generally greater than deflections from two axles when the rail support stiffness is less than 1,000,000 lb/in. This means that slab evaluations should analyze both single axle and two axle loading cases.
- Maximum allowable slab tensile stress will be exceeded when slab thickness (base slabs only) is less than 12 in. and the rail support stiffness is 3,000,000 lb/in. or greater.
- Slab natural frequencies (important to ground vibration issues) are estimated.
- Slab pressure on its support (gravel, engineered soils) is between 5 psi for thicker slabs to 10 psi for the thinnest slabs.
- Rail upward or downward force from thermal effects in vertical curves on slabs is innocuous, attaining 140 lb per rail foot for a 90°F temperature above a neutral temperature, 150 foot curve length, and 10% grade change.
- Rail lateral force from thermal rail effects in horizontal curves may require additional lateral restraint for curves with radius 200 ft and less. The effect on rail restraints should be assessed in combination with other circumstances (wheel curving loads, lack of rail pre-curving).

NOTES

1. Hetényi, M. *Beams on Elastic Foundation*. The University of Michigan Press, Scientific Series Volume XVI, copyright 1974, pp.179–185.
2. American Concrete Institute, *Building Code Requirements for Structural Concrete (ACI 318-02) and Commentary (ACI 318R-02)*. However, ACI 318 specifically excludes structural at-grade slabs from the scope of its Code. The analysis incorporates *Design of Slabs on Grade (ACI 360R-92)*, reapproved 1997, partially addressing some track slab design issues. The interpretation of ACI 318 and ACI 360 that is most appropriate for track slab design, and implemented in this work, is in *The Structurally Reinforced Slab-on-Grade*, published by the Concrete Reinforcing Steel Institute in Engineering Data Report Number 33 (1989), and re-published by ACI in *Practitioner's Guide, Slabs on Ground*, American Concrete Institute PP-4 (1998). The results reported are by the "rational method." Four other structural design procedures are available in the calculations.
3. Packard, R. G., and S. D. Tayabji. New PCA Thickness Design Procedure for Concrete Highway and Street Pavements, *Proc., Third International Conference on Concrete Pavement Design and Rehabilitation*. Purdue University, 1985, pp. 225–236.; as implemented by Dr. Yang H. Huang, P.E., Finite Element Analysis of a Proposed Trackbed, prepared for Iron Horse Engineering Company, Inc., Feb. 19, 1991, p. 2.
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Debate of At-Grade Versus Grade Separation Construction Interstate MAX Project, Portland, Oregon

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The 5.8-mi Interstate MAX Light Rail Project, extending from the Rose Quarter due north to the Expo Center, is currently under construction and is targeted to open for service in September 2004. As part of the former South/North Corridor Study Project, a draft environmental impact statement for the project was completed in 1998. The final environmental impact statement along with preliminary engineering was completed for the Interstate MAX as an independent project in 1999. The majority of the Interstate MAX alignment is designed and built with ballasted track within the existing right-of-way along the middle of North Interstate Avenue.

The alignment starts to divert from the main roadway at Argyle Street, the beginning of the Expo Segment. The Expo Segment crosses major roadways including Columbia Boulevard, Union Pacific Railroad, Schmeer Road, Highway 99W, and Victory Boulevard, as well as the Columbia Slough, a tributary of the Willamette River. During the last 40% of the final design, there was much controversy regarding the vertical light rail train (LRT) alignment crossing Schmeer Road and Victory Boulevard, the crossing of Highway 99W, and the Highway 99W southbound to Victory off ramp. This paper discusses at-grade versus grade separation options through analyzing the special crossing situations, the selection process perspective, LRT operations, and project aesthetics and economy. A comprehensive analysis of all project elements ultimately favored grade separation with aerial structures from Columbia Boulevard until north of Victory Boulevard. The alignment evaluation and decision making process provide valuable experience to future planning and design of similar facilities.

INTRODUCTION

As the northern portion of the original South/North Corridor Study Project, the Interstate MAX Light Rail Transit (LRT) Project extends 5.8 mi from the Rose Quarter Area due north, along the existing Interstate Avenue (Highway 99W) to a north terminus at the Expo Center just south of the Columbia River. The project was ranked number one to be funded in 2000 by the Federal Transit Administration (FTA) based on federal criteria. Currently under construction, the project is projected to open for revenue service by September 2004.

The Interstate MAX LRT Project, including a total of 10 new stations, was subdivided into three segments for design and construction: the Rose Quarter Segment (10A), the Upper Interstate Segment (10B), and the Expo Segment (10C). Segments 10A and 10B are situated in the established north Portland neighborhood characterized by a mixed community of commercial, residential and industrial uses. With extensive involvement of the local communities, the trackway of these two segments were designed within the existing Interstate Avenue right-of-way. The trackway is primarily ballasted track, located exclusively in the middle of the roadway. One of the highlights of the design is no business or homes will be displaced because of the project. Segments 10A and 10B were designed at-grade construction without much question due to right-of-way constraint and the well-established business and residential communities. As the project progresses northward reaching Argyle Street where the Expo Segment starts, the LRT alignment and profile began to generate much controversy. Should the existing Denver Viaduct and the Columbia Slough Bridge be replaced with new bridges to support both Denver Avenue (Highway 99W) traffic and the LRT? If new independent bridges are built for the LRT, how far away should they be from the existing bridges? These issues directly affect the crossing location at Schmeer Road at the north side of the Columbia Slough. Where and at what angle should the alignment cross Highway 99W? Where should the alignment cross Victory Boulevard and the Highway 99W to Victory Boulevard ramp? And finally, perhaps the most difficult question to answer was what the LRT profile should look like between Argyle Street and Victory Boulevard.

It was an easy decision to make during all stages of the process that grade separated constructions should be assumed over Columbia Boulevard due to terrain characteristics and Union Pacific Railroad (UPRR) tracks along Columbia Boulevard, over the Columbia Slough for obvious reasons, and at Highway 99W and the Highway 99W to Victory Boulevard off ramp due to crossing this major highway. However, the decision regarding how to cross Schmeer Road and Victory Boulevard, was not easily made. During review of the 60% final design submittal, FHWA rejected the idea of having an at-grade crossing at Victory Boulevard. At-grade crossings of Schmeer Road and Victory Boulevard were part of the design since the draft environmental impact statement (DEIS) and the final environmental impact statement (FEIS). This late FHWA's rejection led the design team to reconsider other alternatives.

The planning and design process of this Interstate MAX Project provided valuable lessons that can be applied to future similar projects. This paper discusses the argument regarding the decision of elevated versus at-grade construction through the thinking of both soft and hard aspects. The soft aspects included discussion of decision making in the early planning of the project and involvement of interested parties. The hard aspects included analyzing LRT track geometry, safety issues, operation concerns, aesthetics, and project economy.

THE EXPO SEGMENT LRT ALIGNMENT

As illustrated on Figure 1, existing Highway 99W is supported by a bridge over Columbia Boulevard called Denver Viaduct. Highway 99W is then at-grade north of Columbia Boulevard before it is again on bridge over the Columbia Slough. The Columbia Slough extends 18 mi between Fairview Lake on the east to the Willamette River at Kelley Point Park on the west. Construction of any new bridge over the Columbia Slough requires obtaining a U.S. Coast Guard permit. After various studies and field inspections, it was determined that it would not be practical to replace or widen either the existing Denver Viaduct or the Columbia Slough Bridge to accommodate the new LRT trackway. First, it would be very difficult to stage the construction in order to detour the busy traffic along Highway 99W for replacement. Second, it would be difficult to restore the historical merits of the existing bridges. Third, from a cost standpoint, it is

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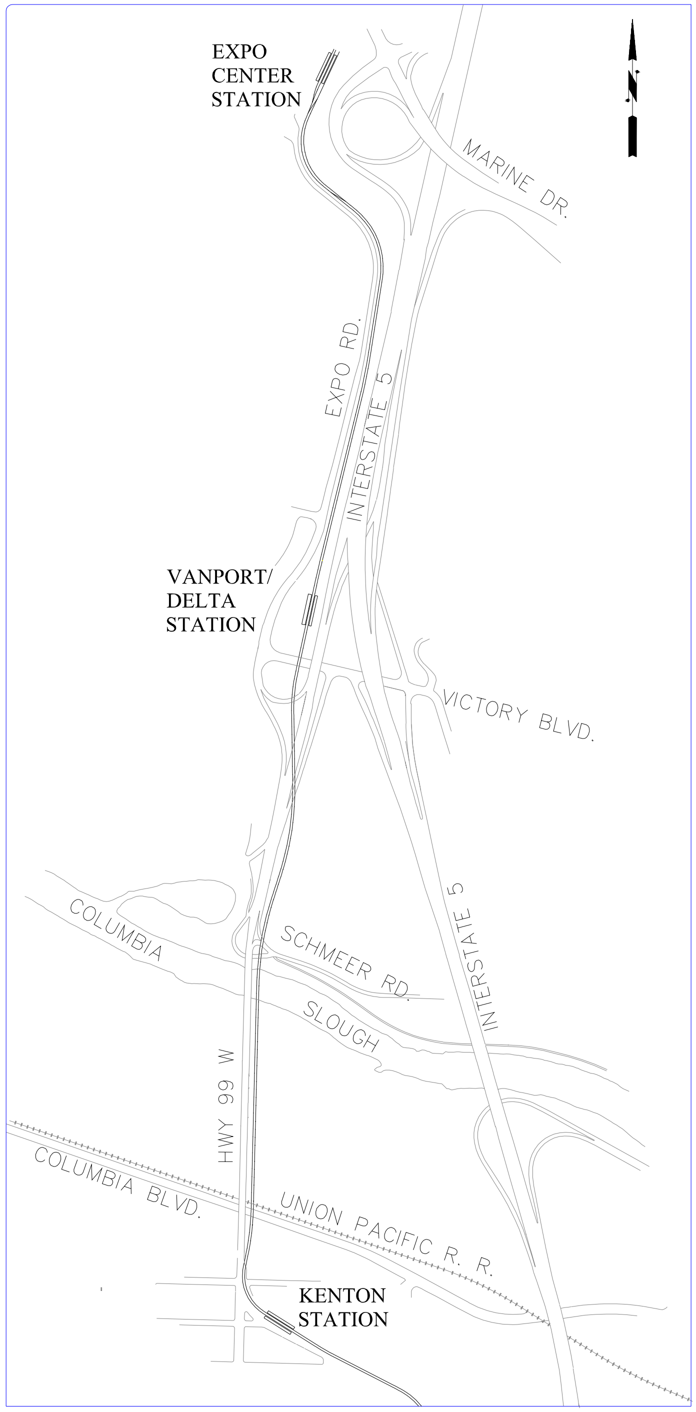

FIGURE 1 Expo Segment LRT alignment.

more expensive to replace or widen the bridges than to build new separate LRT bridges. Therefore, it was determined that independent structures would be built for the LRT trackway between Argyle Street and Schmeer Road. Based on the space and right-of-way availability, the LRT alignment on the east side of Highway 99W was selected.

To shorten the overall alignment length before it ultimately reaches it destination, Expo Center, the alignment was kept as close to Highway 99W as possible. More reasons will be discussed later on the crossing requirements at Schmeer Road. In order to serve the Portland International Raceway (PIR) and lead the alignment eventually to the Expo Center located west of Interstate 5 and just south of Marine Drive, the ideal LRT alignment is to cross Highway 99W between Schmeer Road and Victory Boulevard. North of Delta Park Station, the LRT alignment was chosen to run parallel to Expo Road with minor realignment of the existing Expo Road and wetland mitigation west of Expo Road. The whole alignment of the Expo Segment was designed for double track operation, except a short third track was designed to serve an event platform at the terminal Expo Center Station. Three surface park-and ride lots were designed adjacent to the Delta Park Station.

SPECIAL SITUATION OF SCHMEER ROAD AND VICTORY BOULEVARD CROSSINGS

Schmeer Road Crossing

Currently, Schmeer Road ends at Highway 99W on its west end (Figure 2). Highway 99W northbound is accessible to Schmeer Road at-grade through a tight curve at the north end of the Columbia Slough Bridge, while Highway 99W southbound accesses to Schmeer Road through grade separation under the Columbia Slough Bridge. Schmeer Road has access to Highway 99W northbound, and there is no access to Highway 99 southbound. If at-grade crossing for the LRT were assumed as it was planned during the preliminary engineering and early final design stage, the intersection of Schmeer Road would need to be modified as demonstrated in Figure 3. The flood levee on the north bank of Columbia Slough is immediately adjacent to the south edge of Schmeer Road. This would prohibit lowering the existing Highway 99W southbound to Schmeer Road off ramp to provide enough vertical clearance underneath the LRT structure. As a consequence, a traffic signal would have to be added at the intersection to allow at-grade left turn movement from Highway 99W southbound to Schmeer Road. At least two railroad gates would have to be installed at each approach to Schmeer Road, and there was a question regarding whether a third gate would have to be installed to prevent traffic from entering the LRT operation envelop from Highway 99W southbound to Schmeer Road. Other components required to ensure the at-grade crossing to work properly would include necessary electrical circuits to control the signals, gates and crossing panels at the conjunction of the LRT track and the roadway.

Victory Boulevard Crossing

The vicinity of the intersection of Victory Boulevard, Interstate 5 and Highway 99W is already a complex traffic operation center, especially during auto racing events at the PIR just west of the intersection. Victory Boulevard has full access to Interstate 5 and Highway 99W. After diverging

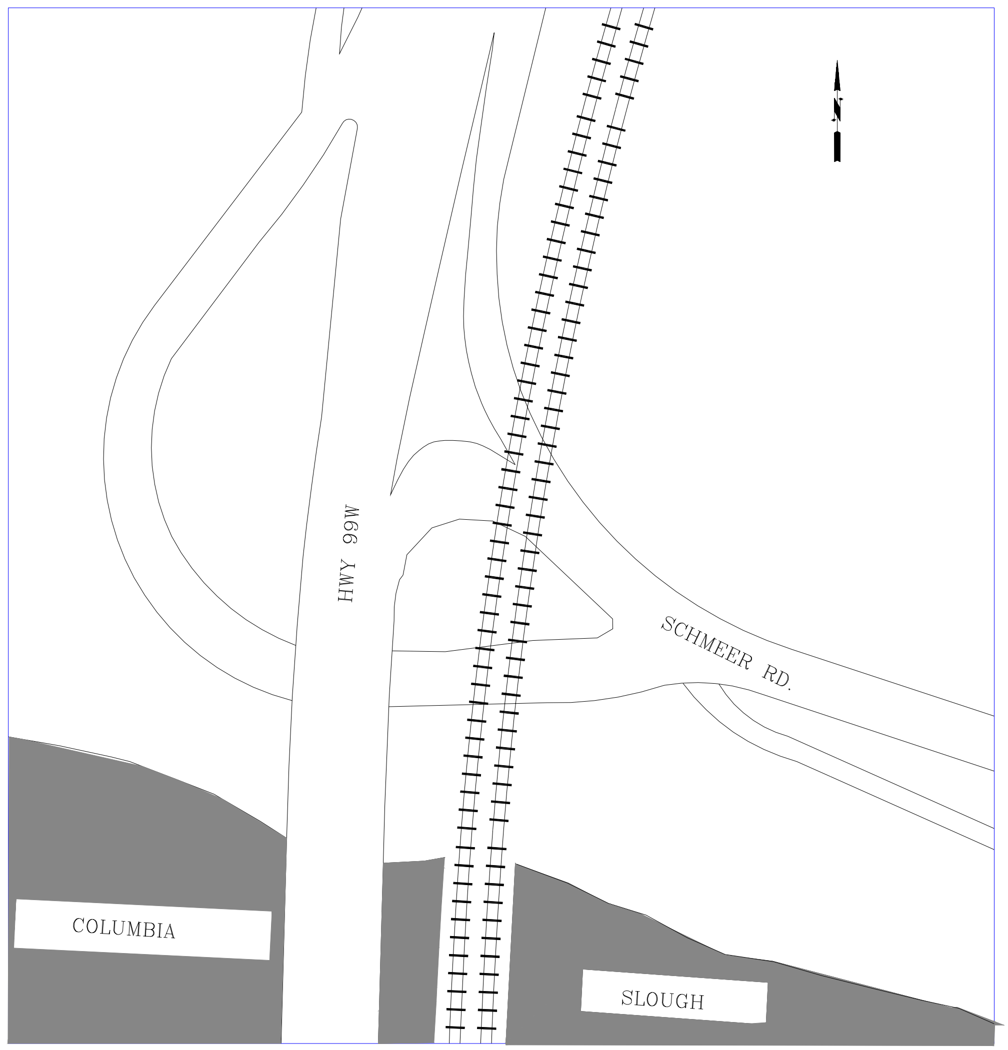

FIGURE 2 Existing Condition of the Intersection of Highway 99W and Schmeer Road with New LRT alignment.

off Interstate 5 north of Victory Boulevard, Highway 99W is less than 300 ft west of Interstate 5. The space between the on and off ramps for Interstate 5 and Highway 99W is already in a substandard operation mode. An at-grade crossing for the LRT within the vicinity would further complicate the traffic operation.

The proposed LRT alignment is about 60 ft west of Highway 99W. If the LRT profile were to pass under Highway 99W, as it was determined early in the project, it would also need to go under the Highway 99W southbound ramp to Victory Boulevard in order to cross Victory Boulevard at-grade. Gates and special electrical circuits would be required to control the crossing. In addition, due to the LRT coming out from a depressed profile and tunnel, retaining walls on both sides of the trackway as well as the aerial structure of Highway 99W would limit the sight distance. Therefore, advanced warning devices would be required for westbound traffic on Victory Boulevard east of Highway 99W.

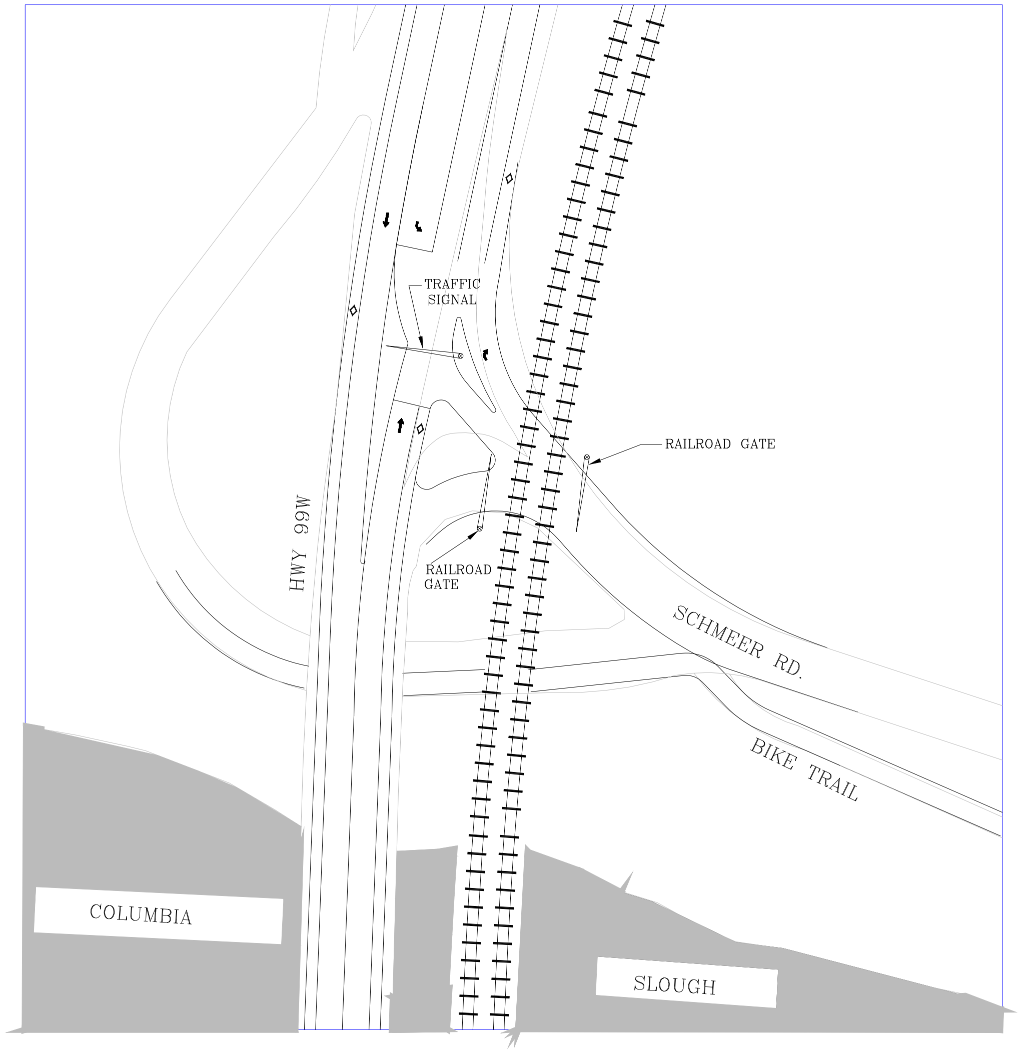

FIGURE 3 Modification of the intersection of Highway 99W and Schmeer Road due to the LRT at-grade crossing.

EARLY DECISION MAKING ON THE LRT PROFILE

As mentioned earlier, this Interstate MAX Project had gone through the DEIS process for the former South/North Project when the conceptual engineering was carried out. Then, preliminary engineering was completed as requirement for the FEIS. Immediately after the at-grade crossing Argyle Street, the LRT profile was brought up with a 5% grade so that impacts to a historic-eligible building south of Columbia Boulevard could be minimized. Like the existing Denver Viaduct, it was decided that the LRT would be supported on a bridge over Columbia Boulevard, simply because of the natural terrain; Columbia Boulevard is more than 30 ft below the existing Highway 99W grade. In addition, UPRR has railroad tracks in parallel to Columbia Boulevard. The U.S. Coast Guard requires minimum vertical clearance over the Columbia Slough due to its navigability. The city of Portland has a strategic bike route proposed along the north bank of Columbia Slough (Figure 3). The city required a 10-ft vertical clearance. With a high profile over the historic building and Columbia Boulevard, as well as a bridge over the Slough, it was a logical decision to connect these two major structures by a low profile aerial structure that would span over business driveways and avoid conflicts with at-grade crossings of these business access points.

During all phases of engineering until 60% final design, the LRT profile was always assumed to have an at-grade crossing at Schmeer Road and Victory Boulevard and grade separating Highway 99W and the Highway 99W to Victory Boulevard Ramp by tunnel since grade separating from Schmeer Road to Victory Boulevard with aerial structure(s) was considered too expensive. This decision was made assuming that the LRT could operate under railroad pre-emption through Victory Boulevard and Schmeer Road. The decision was based on consultations with local state transportation personnel. In hindsight, the lack of involvement by the FHWA was a critical oversight.

EVALUATION OF THE HIGHWAY 99W AND VICTORY RAMP UNDERCROSSING

The undercrossing was the proposed structure consisting of cut-and-cover tunnels under Highway 99W and the Highway 99W southbound off-ramp to Victory Boulevard, with retaining walls for the tunnel portals. The undercrossing structure would begin with a cantilever retaining wall on the west side of the alignment to retain the fill slope for Highway 99W. A short retaining wall might have been required on the east side of the alignment before the alignment reached the tunnel's south portal. The tunnel crossed under Highway 99W at a skew of approximately 18°, requiring 515 ft to reach the highway's west side. After the north portal, the structure transitioned to a U-shaped open-top box. The cut-and-cover tunnel structure was required again as the alignment crossed under the Highway 99W to Victory Boulevard ramp. A U-shaped structure with variable height retaining walls was proposed for retaining the ramp fill slopes for approximately 45 ft after the north portal.

Through in-depth research of the existing utilities within the corridor, it was discovered that several utilities would require protection or relocation during construction of the proposed undercrossing. Most surprisingly, there is a buried U.S. West (now Quest) fiber optic telephone line in the northbound shoulder of Highway 99W, crossing the LRT alignment. Consultants for U.S. West had proposed a temporary bridge to support this line during construction of the cut-and-cover undercrossing. U.S. West estimated the cost for this temporary bridge at $300,000. This line would be lowered and re-embedded in the roadway after completion of the undercrossing. The vertical alignment of the tunnel would require accommodating the minimum cover requirements of the telephone line. As a result, the original LRT vertical profile from preliminary engineering would have to be lowered substantially. Consequently, a sag curve with a low point in the tunnel near the middle of the undercrossing appeared to be necessary to achieve sufficient clearance under the roadway to avoid conflict with the existing buried telephone line. This low point would cause problems with drainage and water disposal. The tunnel would need to include a drainage system to remove water inflows. The inflows could come from the following major sources:

1. Seepage from water infiltration;

2. Rainfall on the tunnel approaches;
3. Rainfall entering through open-top box section;
4. Flood events; and
5. Fire flows during emergencies

Water entering the tunnel would need to channel to a floor drain at the low point of the tunnel. A sump would need to be built beneath the bottom of the tunnel slab. A permanent sump pump would be required to discharge the water.

The estimated cost for this undercrossing increased more than 50% from the estimate performed during preliminary engineering because of the new drop in vertical alignment. Because of this drop, excavation depths increased beyond the capacity of conventional sheet-pile shoring assumed in the original estimate. This also resulted in increased quantities of tied-back shoring walls, excavation, and backfill. In addition, concrete and reinforcing had to increase due to additional soil loads at the new alignment depth.

FHWA's REJECTION ON THE AT-GRADE CROSSING AT VICTORY BOULEVARD

Upon reviewing the 60% final design submittal, FHWA rejected the at-grade crossing proposal at Victory Boulevard based on the following reasons:

1. The crossing is within the interchange vicinity of Interstate 5 and Victory Boulevard. The at-grade crossing is too close to major freeway and highway ramps.
2. The at-grade crossing would create intrusion to the highway access control zone.
3. Advanced warning devices could not guarantee safety because of the limited sight distance.
4. The at-grade crossing could induce potential problem during events at PIR.

All these reasons are associated with safety issues. Additionally, FHWA and Oregon Department of Transportation (ODOT) maintained their opposition to the Schmeer Road at-grade crossing.

TriMet immediately conducted management and technical level workshops to review options with ODOT and FHWA. Seven different alignment options were investigated. Options included variations on the location of the Delta Park Station, flyovers, undercrossings, and at-grade crossings. Factors considered for each included safety, local access to the station, and proximity of the station to the park-and-ride facilities, security, impacts on PIR events, traffic impacts, and cost impacts.

Although it was not critically affecting the decision whether or not at-grade crossing should be placed at Victory Boulevard, it might be worthwhile to mention that during early final design, Multnomah Drainage District discovered that the LRT profile north of Vanport/Delta Station cut into the flood control levee by 13 ft. If the crossing at Victory Boulevard altered to grade separation with an aerial structure, this flood levee issue was almost automatically solved.

An elevated LRT profile over Victory Boulevard would naturally require an aerial structure over the Highway 99W to Victory ramp and over Highway 99W to grade separate the ramp and the highway. Once the design of long bridges south and north of Schmeer Road were being considered, there was little hesitation to grade separate this intersection as well, since the at-grade crossing had complicated the intersection as discussed previously.

After review of all design, construction, cost, and operating factors, TriMet decided to change the Expo Segment design to incorporate a single elevated structure, extending from Argyle Street to the Delta Park Station. ODOT and FHWA concurred with this decision.

OPERATIONAL ADVANTAGES OF GRADE SEPARATION

The resulting 4,000-ft long aerial LRT structure between Argyle Street and north of Victory Boulevard would ensure train service with no interruption between Kenton Station and Delta Park Station (Figure 4). Some of the operation advantages with grade separation versus at-grade crossing through Schmeer Road and Victory Boulevard include:

1. With no sight distance constraints coming out from the tunnel and U-shaped box, the aerial structure would allow light rail vehicles to operate in full design speed. It could speed up the service by saving up to 20 s each trip to the north.
2. No railroad gates are needed at the intersections, therefore the maintenance at the crossings, such as weekly inspections and occasional replacement of gates is not required. Crossings of business driveways are also avoided, thus increasing overall safety and reducing the impacts to those businesses.
3. No additional traffic signal is needed at the intersection of Schmeer Road and Highway 99W. Highway 99W southbound to Schmeer Road ramp can operate as they are today without traffic interruption on Highway 99W.
4. Additional illumination is not needed at the at-grade crossing; and no additional control circuits for the gates and signals are required.

FIGURE 4 Grade-separation with long aerial structure between Argyle Street and Victory Boulevard (looking south from Delta Park Station).

AETHETICS CONSIDERATION

There are no residential homes along the entire Expo Segment. Only limited industrial businesses are situated along the southern portion of the long aerial structure. If there is a visual impact from the project, it would be mainly on the travelers driving along Highway 99W. However, since the structure has to be high to clear the historic building and the cross roadways (Schmeer, Highway 99, Highway 99 to Victory ramp and Victory), for most part of it, travelers on Highway 99W would able to see through and under the structure (Figure 5). Although there might be different opinions on this aspect, the authors believe that looking through the bridge piers is better than looking at the catenary power poles and wires at a typical at-grade design.

COSTS COMPARISON

Table 1 tabulates costs comparison for the affected construction items between Argyle Street and the Delta Park Station. With more detailed investigation and design, the costs estimate for the cut and box under Highway 99W and Highway 99W to Victory Boulevard ramp during 60% final design was approximately $4 million higher than that during preliminary engineering. As previously discussed, this significant increase of costs for the cut and cover box is due to the fact that the LRT vertical alignment has to be lowered substantially in order to protect major fiber optic telephone line.

The final estimated cost comparison between the original design and the change to an aerial structure actually indicated a savings of over $600,000 to the project. Consideration of the cost saving from construction, plus that from maintenance for gates and signals down in the road during operation, would favor the grade separation option.

FIGURE 5 LRT structure (near side) higher than the highway structure (far side).

TABLE 1 Estimated Costs Comparison of At-Grade Versus Grade Separation Crossings

Key Evaluated Cost Items	100% PE: At-Grade Crossing at Schmeer and Victory and Cut and Cover Box Under Highway 99W	60% Final Design: At-Grade Crossing at Schmeer and Victory and Cut and Cover Box Under Highway 99W	Final Design: Grade Separation with Long Aerial Structure
Overall Civil Construction	$12,650,000	$16,360,000	$16,151,000
Cut and Cover Box	$4,570,000	$7,620,000	N/A
Mobilization/Permits	$330,000	$550,000	$1,000,000
Design Costs	$510,000	$840,000	N/A*
Additional Aerial Structure	N/A	N/A	$8,000,000
Insurance	$470,000	$570,000	$550,000
Track Materials	$600,000	$600,000	$740,000
Signals	$470,000	$470,000	$0
Contingencies	$2,300,000	$2,900,000	$2,800,000
Total Evaluated Costs	$16,490,000	$20,900,000	$20,250,000

* Hidden in civil construction as design–build contract.

CONCLUSIONS

An independent LRT structure and alignment parallel to the existing Highway 99W for the Expo Segment of the Interstate MAX Project has proved to be a wise decision. Utilizing the limited right-of-way seamlessly, the alignment crosses Highway 99W and Victory Boulevard at the right locations in order to maintain the locations of the Delta Park Station and PIR park-and-ride.

The determination of an optimum, and approvable, vertical LRT profile proved to be more complicated than was assumed earlier in the project. A more detailed and complete analysis prompted by the actions of FHWA demonstrated that a long aerial LRT structure, grade separating the LRT from the ground traffic operation between Argyle Street and Victory Boulevard, will be a better vertical alignment. The full grade separation with aerial structure has proved not only safer and faster for operation, but also more cost competitive based on the actual construction contract bid.

Through the evolution of the Expo LRT alignment, valuable lessons-learned can be provided for other similar projects. The important ones include:

1. Identification and involvement of all interested parties early in the project is essential to the success of project down in the road.
2. Keep an open-mind for options during early project planning to produce optimum outcome.
3. “Guesstimate” of tunnel construction costs without knowing all underground utilities and construction staging issues could be hazardous.
4. Comprehensively analyze all the project elements to gain a thorough understanding of the true cost benefits should make the project a great success.