12 / TECHNICAL REPORTS

HYDRAULICS REPORT
STRUCTURAL REPORT

TECHNICAL REPORTS: HYDRAULICS REPORT

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TECHNICAL MEMORANDUM – RHCAP HYDRAULIC REPORT

TO:	Lee Alexanderson, P.E., Los Angeles County Public Works Dan Sharp, P.E., Los Angeles County Public Works
PROJECT:	Rio Hondo Confluence Area Project Task Order 1 - Signature
TASK NUMBER:	6.3
SUBJECT:	Rio Hondo Confluence Area Project Draft Hydraulic Report
SUBMITTED BY:	Al Preston, Ph.D., PE, Geosyntec Mark Hanna, Ph.D., P.E., Geosyntec Joe Goldstein, P.E., Geosyntec
DATE:	12 June 2020
MEMO NUMBER:	6.3-Hydraulics

The following Technical Memorandum summarizes the computational fluid dynamics (CFD) hydraulic analyses performed to assess feasibility for specific aspects of the Rio Hondo Confluence Area Project (herein referred as the "Project"). Geosyntec performed the CFD modeling and acknowledges the vast contributions by OLIN and Gehry Partners in building the 3D geometries used in the modeling.

INTRODUCTION

The Project area consists of approximately 2 miles of the Los Angeles River (LAR) channel (from River Mile [RM] 11 to RM13) and approximately 0.8 miles of the Rio Hondo (RH) channel. Both channels are of trapezoidal shape with concrete bottoms and predominantly grouted stone side-slopes.[1] The channels are leveed (i.e., the elevation of the channel top of bank is higher than surrounding land) and most portions have additional parapet walls ranging from approximately 4 to 10 feet in height to increase flood capacity. The hydraulics in the area are complex, with regimes alternating between supercritical and subcritical flow, near-critical regions with unstable flow, bridge crossings at oblique

[1] Portions of the Rio Hondo had a smooth concrete overlay applied to the side-slopes as part of the LACDA improvements to improve hydraulic performance and allow adequate conveyance past the UPRR Bridge crossing the Rio Hondo (USACE 1999).

angles and with skewed piers, confluence of a high velocity flow in the LAR with a lower velocity flow in the RH, and designated overtopping weirs (Geosyntec 2019).

Project concepts involve creating:

- platform parks spanning portions of the rivers, that would be supported on walls within the channels,
- additional pedestrian bridge park to provide connectivity and additional park space,
- terraces and ramps within the channel sides to provide space and access to the channel,
- modifications to the low flow channel, including a braid design, shifting the channel nearer to the west bank to create more usable space for amenities and events, a crossing to enable connectivity and access for maintenance, and a diversion side channel to enable pumping of water to nearby wetlands.

Many of these concepts were evaluated in a feasibility phase using 1-D HEC-RAS models (USACE 2004, 2005), engineering judgement and best practices (Geosyntec 2019). However, due to the complexities of the hydraulics in the area, and noting the previous channel improvements in the 1990s and early 2000s utilized physical modeling studies to augment 1-D calculations (USACE 1999), it was recommended that additional more complex analyses using multi-dimensional numerical models and/or physical modeling studies be conducted.

This technical memorandum summarizes the 3-D CFD modeling that was subsequently performed as to further evaluate feasibility of proposed concepts. CFD modeling is typically not carried out in the feasibility phase, but the design team recognized the limitations of the previous analyses and wanted to only pursue and present concepts that are realistic. The goal of this CFD modeling is to provide additional insights into design, further evaluate feasibility, and to rule out concepts that clearly are not feasible. Additional CFD modeling will be required for design phase and for permitting (e.g., with the LACFCD and USACE).

The following sections describe the development and verification of the baseline model, evaluations of the proposed platform parks and Blue Park including modifications to the model and results and evaluations. Conclusions and recommendations for further study are also provided.

BASELINE MODEL

The CFD model was developed using Flow-3D[2] software and covers approximately 3.8 miles of the LAR channel and 1.2 miles of the RH channel, as illustrated in Figure 1. Notable features included in the model include the Union Pacific Railroad (UPRR) crossings over the LAR and RH, the I-710 and

[2] https://www.flow3d.com/

Imperial Highway crossings over the LAR, and the overflow weir on the East bank (Figure 1). The 105 Freeway interchange supports and piers between RM10.7 and RM10.5 in the LAR are downstream of the project area and were not included in the model. Similarly, the Garfield Avenue and Southern Avenue bridges on the RH are upstream of the project area and were not included in the model. Further details on the baseline model are provided in the following sections.

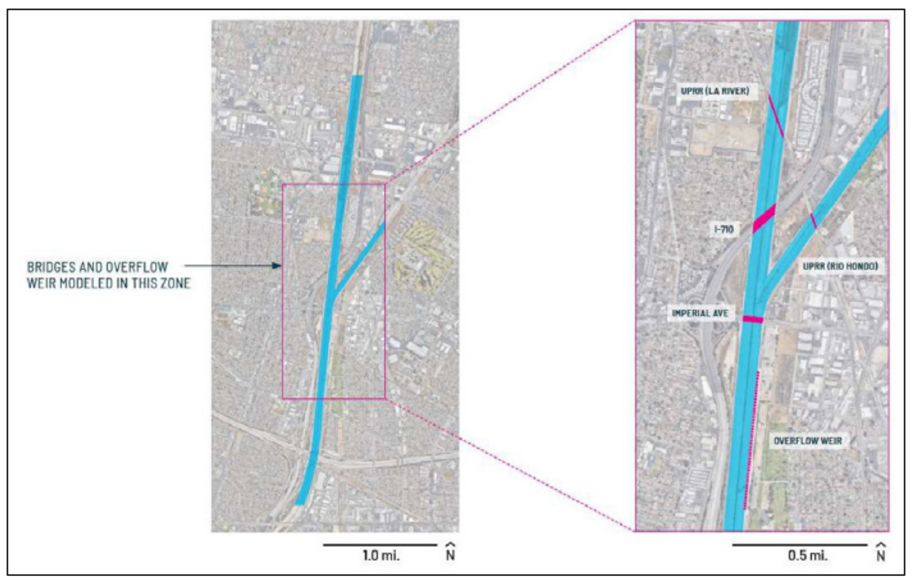

Figure 1. The CFD model covers approximately 3.8 miles of the LAR channel from RM14.1 (approximately 0.8 miles upstream of Firestone Blvd) to RM10.3 (approximately 0.3 miles downstream of 105 Freeway, and approximately 1.2 miles of the Rio Hondo channel.

Geometry and Meshes

The model geometry was constructed using Los Angeles Region Imagery Acquisition Consortium (LARIAC) LiDAR terrain data from 2016 as a base. Channel and bridge geometries were then developed as 3D surfaces in Rhino[3] by OLIN and Gehry Partners based on as-built plans and field observations (e.g., Figure 2). A local horizontal coordinate system (x,y) was used, while the vertical coordinate (z) used NAVD88 as the datum. The geometries were exported into stereolithography (STL)

[3] https://www.rhino3d.com/

files and imported into Flow-3D. The geometries were rotated 7.5 degrees counterclockwise[4] to better align the main flow paths in the LAR channel with the Flow-3D model cartesian meshes that align with the local ordinate x, y, and z axes.

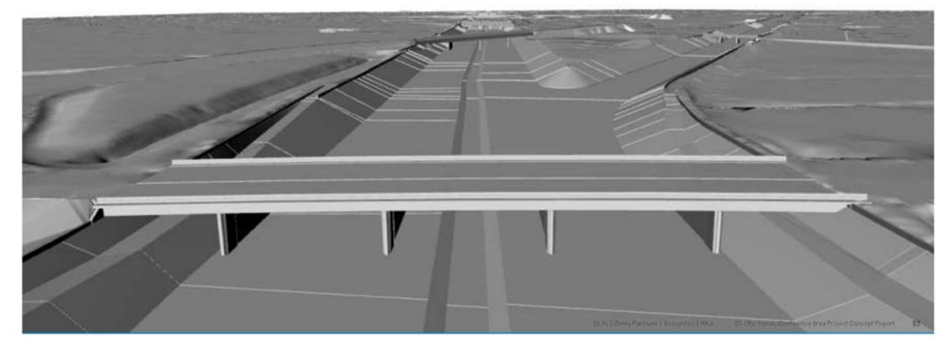

Figure 2. Perspective view of the domain indicating the LAR channel with Imperial Highway bridge and 710 Freeway crossings, and the RH channel with UPRR crossing. The CFD model geometry was based on LiDAR data (LARIAC, 2016) used directly for the overbank areas and 3D surfaces for the channels and bridges built in Rhino based on as-built plans and field reconnaissance. Source: OLIN, Gehry Partners.

The model was discretized by defining a range of different meshes with resolution and dimension (i.e., 2-D or 3-D) determined by the level of detail required. The external terrain was not a focus of the study and was thus modeled with 20 ft cell size using a 2-D[5] mesh (Table 1). The majority of main channel was also modeled with a 2-D mesh, using smaller 10 ft cell size to better resolve the details of the trapezoidal channels which are approximately 250 ft (RH) to 450 ft (LAR) wide. Within the 10 ft channel mesh additional 2-D and 3-D nested[6] meshes with cell sizes ranging from 1.5 ft to 2 ft were added to resolve flow around bridge piers and bridge decks (Table 1). The model for the Southern Pacific Railroad (SPRR) and Firestone bridges over the LAR did not include bridge decks, even though

[4] The geometries were rotated about (x,y,z) = (0,-1400,0) feet.

[5] A 2-D mesh solves the depth averaged, or shallow water, equations of flow which are valid if vertical accelerations can be neglected (i.e., the pressure can be assumed to be hydrostatic). This is valid for most floodplains, and channels that are not too steep (i.e., generally less than 10% grade) including flow around bridge piers provided the bridge deck is not impinged.

[6] Nested meshes were made to have the same vertical extent as the larger 2-D meshes to avoid issues at the interface.

results indicate that these decks may be impacted at the flow rates evaluated. These bridges are well upstream of the Project area.

Table 1 Mesh dimension, size, and number of cells

Location	Dimension	Size (x,y,z)	Number of Cells
External terrain	2-D	20 x 20 ft	250,000
Channel	2-D	10 x 10 ft	1,000,000
SPRR	2-D*	2 x 2 ft	22,000
Firestone	2-D*	2 x 2 ft	37,000
UPRR LAR	3-D	2 x 2 x 2 ft	3,740,000
710	2-D	2 x 2 ft	67,000
Imperial	3-D	1.5 x 2 x 2 ft	9,457,000
UPRR RH	3-D	1.5 x 1.5 x 1.5 ft	2,738,000
TOTAL			17,435,000

* Meshes were 2-D since bridge decks were not included. At the flow rates evaluated the SPRR and Firestone bridge decks may be impacted by water. This is not included in the model since these bridges are far upstream of the Project area.

The locations of the nested meshes for the UPRR LAR, I-710, Imperial Highway, and UPRR RH bridges are indicated in Figure 3. Bridge decks were included in the model for the UPRR LAR, UPRR RH, and Imperial Highway crossings since the flow impinges these bridges at the design peak flow rates considered (USACE 1999). The meshes used to resolve these bridges in 3-D use millions of computational cells (Table 1). By contrast, the I-710 deck is raised well above the water surface and is not needed to be modeled. The bridge piers can be modeled in 2-D using only 67,000 cells.

The mesh around the Imperial Bridge extends further upstream and downstream of the bridge than the other meshes (Figure 3) to enable the same mesh to be used to evaluate the Blue Park features, including low flow channel modifications and a new pedestrian crossing (see Blue Park section). To partially offset this larger mesh length elongated cells were used (i.e., with dimensions of 1.5 ft in the x-direction across the channel, 2 ft in the y-direction along the channel, and 2 ft in the vertical z-direction).

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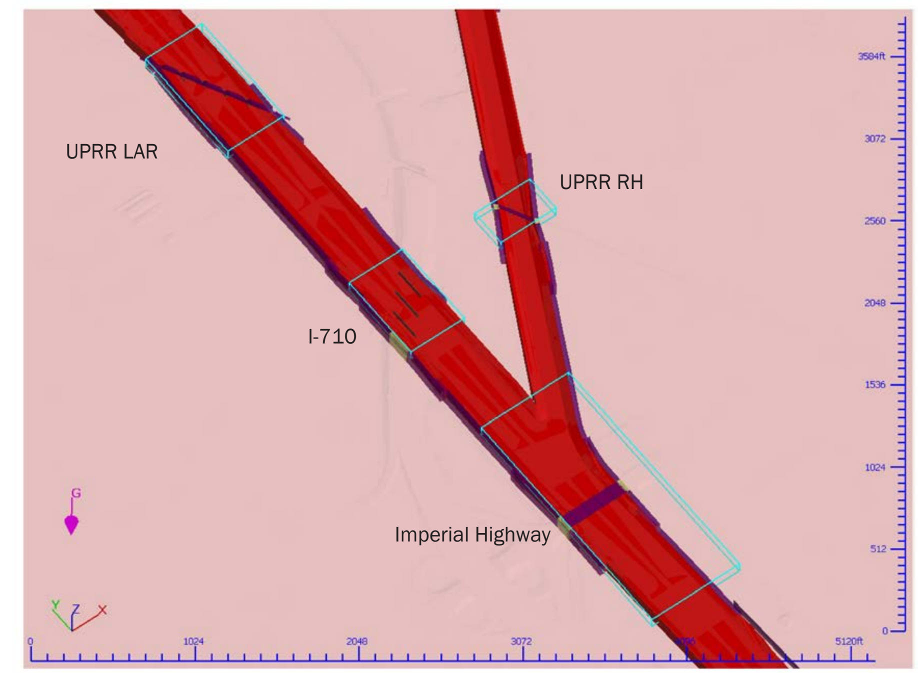

Figure 3. The CFD model used a 10-ft 2D mesh throughout the channel with nested meshes to resolve the more complex flow around bridge piers and decks. These nested meshes used cell sizes ranging from 1.5 ft to 2 ft in size. The nested mesh around the I-710 Freeway crossing was 2D, since the bridge deck is above the water surface elevation. The meshes around the UPRR LAR, UPRR RH, and the Imperial Highway crossings were all 3D.

The UPRR crossings over the LAR and RH cross at oblique angles (Figure 3) and are noted to sit below the parapet walls. This is illustrated for the UPRR LAR crossing close-up views in Figure 4. The figure shows the representation of the bridge piers and bridge deck geometries. The original bridge piers were built prior to the channelization of the river and do not align with the general flow direction and have been retrofitted with noses and tails that do align with the flow (Figure 4, right). The skewed piers may produce complex flow patterns, disturbances, and sideways forces on the piers. Figure 4 (right) also indicates the level of detail that is obtained with the 2 ft computational mesh.

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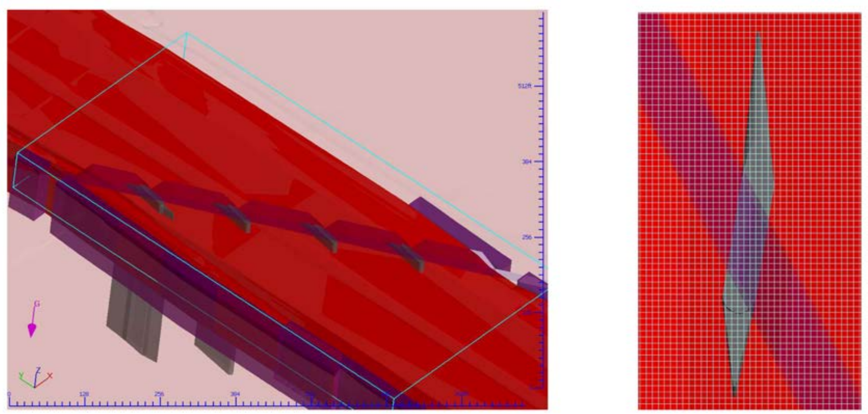

Figure 4. Close-up views of the UPRR crossing over the LAR. The bridge deck sits below the parapet walls and crosses the river at an angle (left) that invalidates the assumptions used in 1D HEC-RAS modeling. Close-up of the western-most pier (right) shows detail of the mesh used in the modeling and illustrates that the piers are not aligned with flow direction.

Roughness

The geometries in the 3-D CFD model are defined with a roughness height, k, to represent whether they are relatively smooth concrete (i.e., for channel bottom and some smoothed channel sides) or rougher grouted stone (i.e., for most channel sides). Design memoranda typically use values for roughness in terms of Manning's n, which is more conventional for traditional channel hydraulic calculations. Values of n = 0.014 for concrete and n = 0.020 for grouted stone are provided for the conditions in the Project area (USACE 1999). To convert these n-values to physical k-values the equations used to calculate the bottom drag coefficient for the depth-averaged equations (i.e., 2-D CFD model) from both n and k per the Flow-3D User Manual (Flow Science 2019, Section 11.3.21) were used to plot the drag coefficient as functions of flow depth (Figure 5). Based on these comparisons at a typical flow depth of approximately 15 ft appropriate values for k were determined as summarized in Table 2. A roughness value of 2 ft, representing a typical value for overland flow, was used for the external terrain. This value may want to be refined and made spatially variable if flooding adjacent to the channel is to be further evaluated.

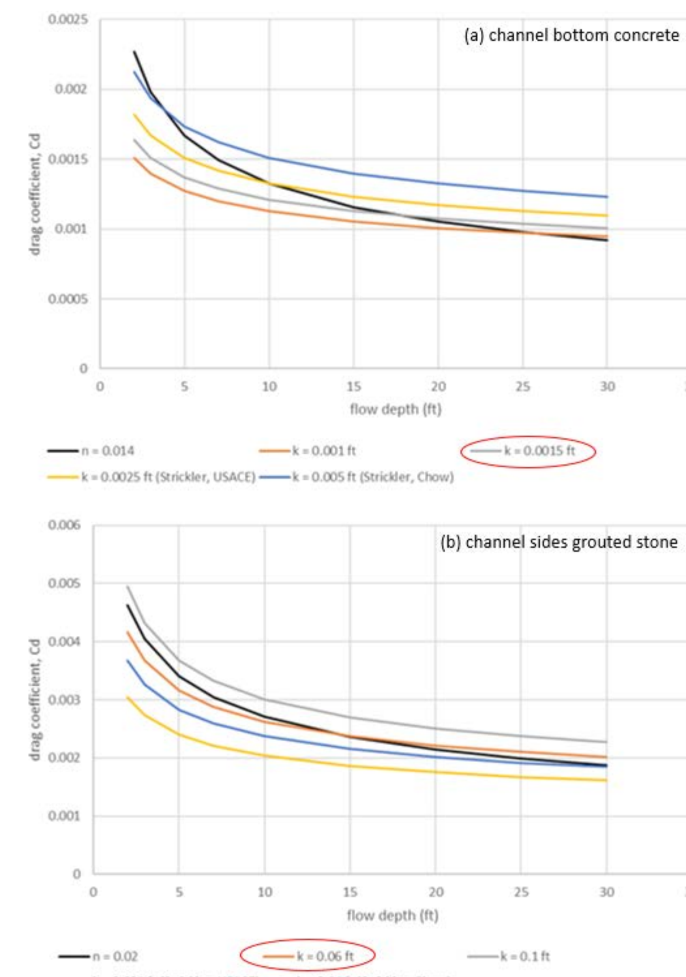

Figure 5. Plots of drag coefficient versus flow depth for (a) channel bottom concrete (n = 0.014) and (b) channel sides grouted stone (n = 0.020) and a range of different roughness, k, values. Based on comparisons for a typical depth of ~15 ft it was determined that values of k = 0.0015 ft for concrete (left) and k = 0.06 ft for grouted stone (right) were reasonable.

Table 2. Roughness used in CFD model

Surface	Manning's n (USACE 1999)	Roughness, k
Concrete	0.014	0.0015 ft
Grouted stone	0.020	0.06 ft
External terrain		2.0 ft

The implementation of the different roughnesses is illustrated in Figure 6. The entire channel bottom is concrete, while most of the channel sides are grouted stone. Some portions of the channel sides are smoother concrete, including in the RH channel at and downstream of the UPRR crossing that was smoothed for hydraulic benefit (USACE 1999) and at other crossings where bike path underpasses were built. The different regions of concrete versus grouted stone sides were determined from review of design memoranda, viewing in Google Earth, and field confirmations.

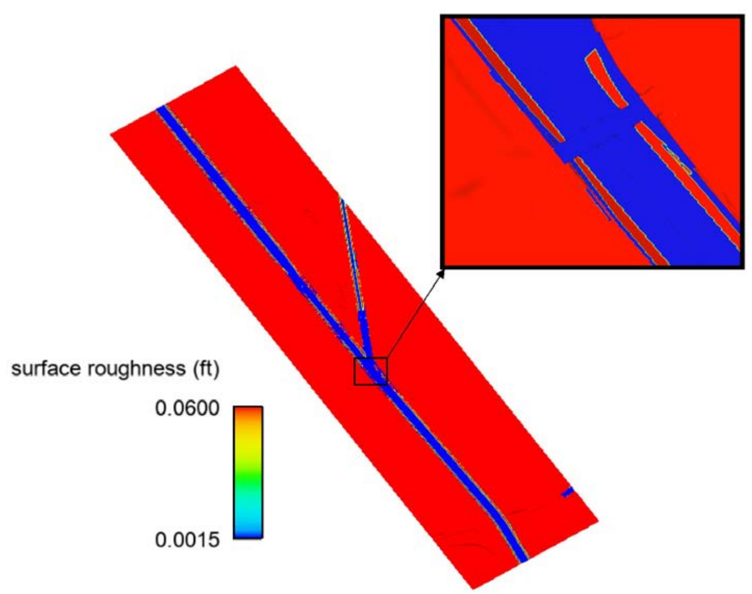

Figure 6. Surface roughness used in the CFD model included k = 0.0015 ft for concrete, k = 0.06 ft for grouted stone channel sides, and k = 2.0 ft for surrounding terrain.

Flowrates and Boundary Conditions

The flow rates used in the CFD model are presented in Table 3 and include two design peak flow conditions representing large flood events to assess flood carrying capacity and a typical dry weather flow rate to further assess the low-flow channel modifications. The two design peak flow conditions are based on maximum deliverable discharge (MDQ) analyses and approximately represent peak flows for a 0.2% (500-year) event (USACE 1999). The timing of the peak flows in the LAR and RH are different due to upstream controls (e.g., Sepulveda and Hansen Dams in the upstream LAR watershed, and the Whittier Narrows Dam on the RH), and the two flow scenarios representing each peak were selected (Table 3), which is consistent with the USACE physical modeling approach used in the 1990's (USACE 1999). Both peak flow conditions result in a total combined flow of 184,000 cfs below the confluence.

Table 3 Flowrates used in the CFD model

Scenario	Q_{LAR} (cfs)	Q_{RH} (cfs)
LAR design peak flow	140,000	44,000
RH design peak flow	131,100	52,900
Blue Park low flow	100	0

The inflow rates were specified as volume flow rate inflows at the upstream boundaries in the model. At the downstream boundary the fluid elevation was specified at 90 ft. Varying this elevation did not have a substantial effect on the simulation within the region of interest.

Initialization and Runtimes

Simulations were initialized by first running a simplified model without bridge geometries and without the nested meshes. The flow rates were instantaneously applied at the boundaries which resulted in waves traveling down the channels as they filled up. These waves reflected off the downstream boundary and resulted in some channel overtopping. These transient waves are not reflective of real-world conditions, and some remnants from the overtopping may be notable in the final simulations. These initialization simulations were run to 2,000 seconds to allow the transient waves to subside, which took approximately 3.5 hours of computer time.

The water depths and velocities from the initialization simulation were used for initial conditions in the final simulations. The bridge geometries and associated meshes were essentially added instantaneously to the simulation, which resulted in additional transients. These final simulations were run to 60 seconds to allow transients to subside, which took 27 to 40 hours of computer time per simulation.

Results and Verification

Results from the baseline model simulations are presented in Figure 7 and Figure 8 for LAR and RH design peak flow conditions, respectively. Each figure shows views of modeled free surface elevation, flow depth, depth averaged velocity, and Froude number. Results generally look similar, but with additional overtopping along the RH channel for the RH design peak. However, this should be investigated with a model that includes the Garfield and Southern Avenue bridges and additional resolution in the regions of interest. The model predicts overtopping of the LAR channel south of the confluence at the overflow weir location near Hollydale Park, which is consistent with physical modeling studies (USACE 1999). The flow rate over the weir is discussed shortly.

Close-up views of rendered water surfaces for the UPRR LAR, UPRR RH, and Imperial Highway bridges are presented in Figure 9 and provide an indication of the level of detail that the CFD model can provide. The results indicate that the bridge decks are impinged by water, which is consistent with physical modeling studies conducted in the 1990's (USACE 1999).

The simulated free surface elevation is plotted as a function of distance along the LAR channel for the LAR design peak flow scenario in Figure 10. Results of the physical model results from the 1990's (USACE 1999, Table B-7) are also indicated on the plot. The results from the CFD model are taken along the channel centerline and represent instantaneous values (at t = 60 seconds), whereas the physical model results are averaged over time and multiple locations across the channel. Despite the different sampling methods the results indicate generally good agreement with the model able to replicate the steep drop in elevation down the 'chute' upstream of the UPRR bridge, the relatively 'level' elevation between the UPRR bridge and upstream of Imperial Highway, the drop immediately upstream of Imperial Highway, and the general elevations near the left bank overflow weir region. The complex 105 Interchange support structures were outside the region of interest and were not included in the model and likely resulted in the free surface elevation being lower than physically modeled in that region.

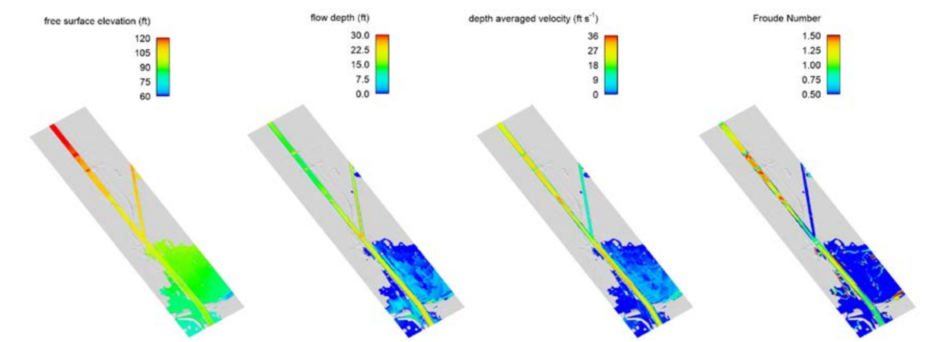

Figure 7. Baseline model results for the LAR design peak flow conditions (QLAR=140,000 cfs, QRH=44,000 cfs), showing free surface elevation, flow depth, depth averaged velocity, and Froude number. Results illustrate depths ranging from about 15 feet to 25 feet and velocities in excess of 30 feet per second (fps) in the LAR channel that is mostly supercritical (Froude number > 1). Flows in the RH channel are slower and subcritical (Froude < 1) and generally deeper. There is some overtopping predicted at isolated locations along the RH channel, although these should be further evaluated with a higher resolution model. The model predicts overtopping of the LAR channel south of the confluence at the overflow weir location near Hollydale Park, which is consistent with physical modeling studies (USACE 1999).

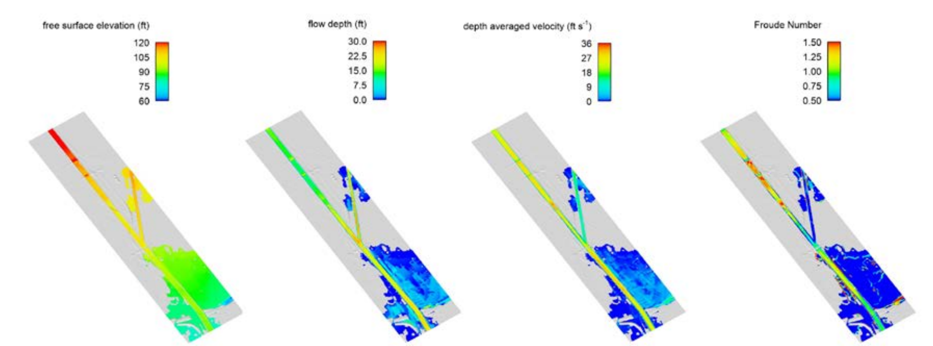

Figure 8. Baseline model results for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs), showing free surface elevation, flow depth, depth averaged velocity, and Froude number. Results illustrate similar trends to the LAR peak conditions (Figure 7), but with notably more overtopping along the RH channel.

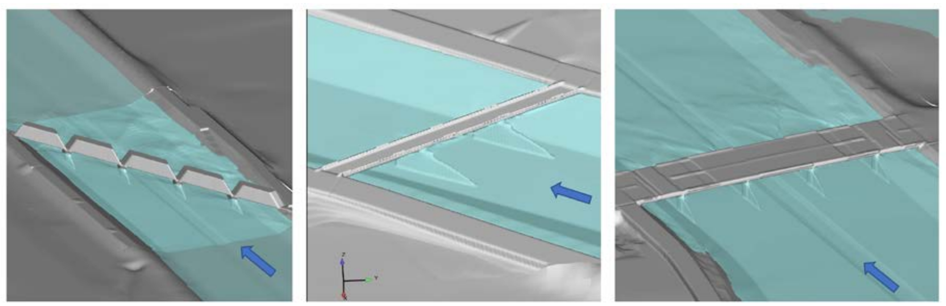

Figure 9. Close-up of baseline model results for design peak flow conditions for UPRR crossing of LAR at LAR peak flow (left), UPRR crossing of RH at RH peak flow (center), and Imperial Highway at RH peak flow (right). The CFD results indicate that these bridge decks are impinged at these flow rates, which is consistent with physical modeling studies conducted in the 1990's (USACE 1999).

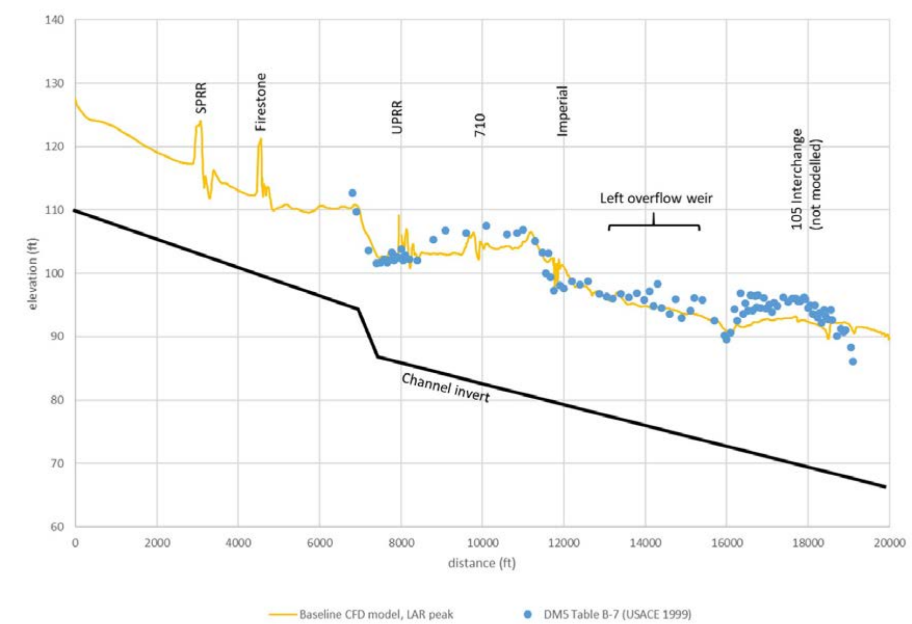

Figure 10. Comparison of baseline CFD model results with physical model results from the 1990's (USACE 1999). The baseline CFD results represent an instantaneous free surface elevation of the LAR design peak flow taken along the channel centerline at t = 60 s; whereas, the physical model results are averaged over time and width across the channel. The CFD model is able to replicate the steep drop in elevation down the 'chute' upstream of the UPRR bridge, the relatively 'level' elevation between the UPRR bridge and upstream of Imperial Highway, the drop immediately upstream of Imperial Highway, and the general elevations near the left overflow weir region.

The model provides estimates of the flow rate over the weir as a function of time as presented in Figure 11. The model results agree well with the average value determined in the physical modeling studies (USACE 1999). The fluctuations of a few hundred cfs about the mean value in the model results may indicate longer simulation time is needed to reach steady-state, or that the flow in this region is inherently unsteady (e.g., due to travelling waves). Future investigations may consider longer run-times and using a model using a higher resolution and possibly 3-D mesh and refined built geometry to better represent the top and backside of the weir.

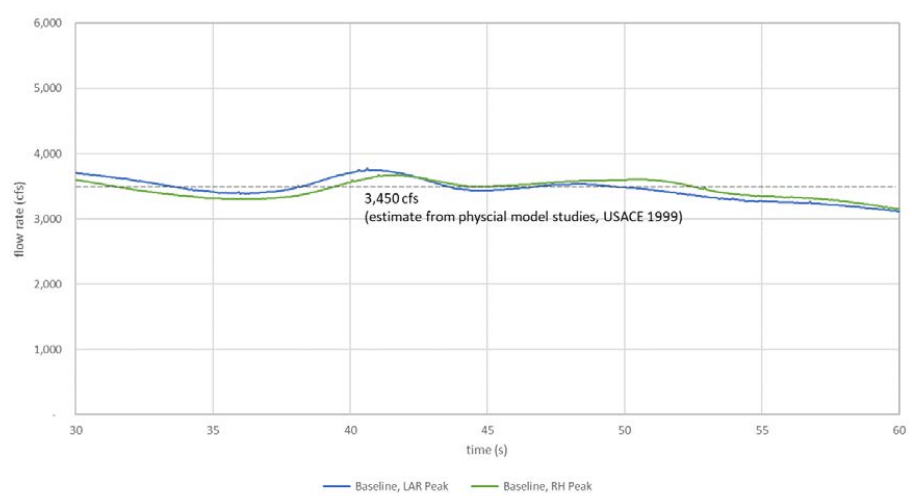

Figure 11. Modeled flow rates over the weir on the East Bank as a function of time. The time-axis starts at 30 seconds to remove the initial transients that occur in the simulation. The model results are presented for the LAR and RH design peak flow conditions, both of which have a combined flow downstream of the confluence of 184,000 cfs. Model results agree well with the average value determined in physical modeling studies (USACE 1999).

The baseline model results presented herein provide verification of the CFD model suitable for assessing feasibility of concepts. A more formal validation may be required prior to full design and for permitting purposes. This may require additional refinements of geometries and meshes in critical regions and more detailed comparisons to the 1990's physical model studies, potentially re-scaling the model to the same scale as the physical model to remove differences caused by scale effects.

PLATFORM PARKS

Model Changes

Changes were made to the baseline models to represent LAR and RH Platform Parks. In both cases, only Option A, which requires full replacement of the existing UPRR crossings, was evaluated. The geometries for the existing UPRR bridges (i.e., piers and decks) across the LAR and RH were removed and new STL geometries were added to represent the 3-ft thick support walls, including pier noses, as illustrated in Figure 12. The support walls in the RH were thickened to 5 ft at the UPRR crossing to support the rail load, including the additional future Metro West Santa Ana Branch (WSAB). An additional three short walls/piers were added to the LAR at the UPRR crossing to shorten the spans for the rail. The roughness of the channel side walls in the vicinity of the platforms was changed to represent smooth concrete (k = 0.0015 ft), rather than grouted stone (k = 0.006 ft), to reduce the hydraulic resistance.

New 3-D[7] meshes were defined to fully resolve the walls. The LAR mesh included the I-710 piers and comprised of 1.5 x 3 x 3 ft cells. The elongated cells with 2:1 length to width ratio can be used due to the relatively uniform flow direction within the channel that is aligned with the cells. The RH mesh runs at an angle to the channel and therefore a uniform mesh comprising 2 x 2 x 2 ft cells was used. To reduce computational demands the Imperial Highway bridge deck (downstream of the confluence) was removed and the flow around the piers were solved using a 2-D mesh. This is not anticipated to affect the LAR and RH platforms that are upstream of the confluence.

[7] In theory 2-D meshes should be able to correctly resolve the flow field around and between the walls. However, the 2-D routines would not smoothly represent the geometry of the walls, resulting in 'stair-steps'. Using 3-D meshes corrected this issue due to the FAVOR algorithm that Flow-3D uses (Flow Science 2019).

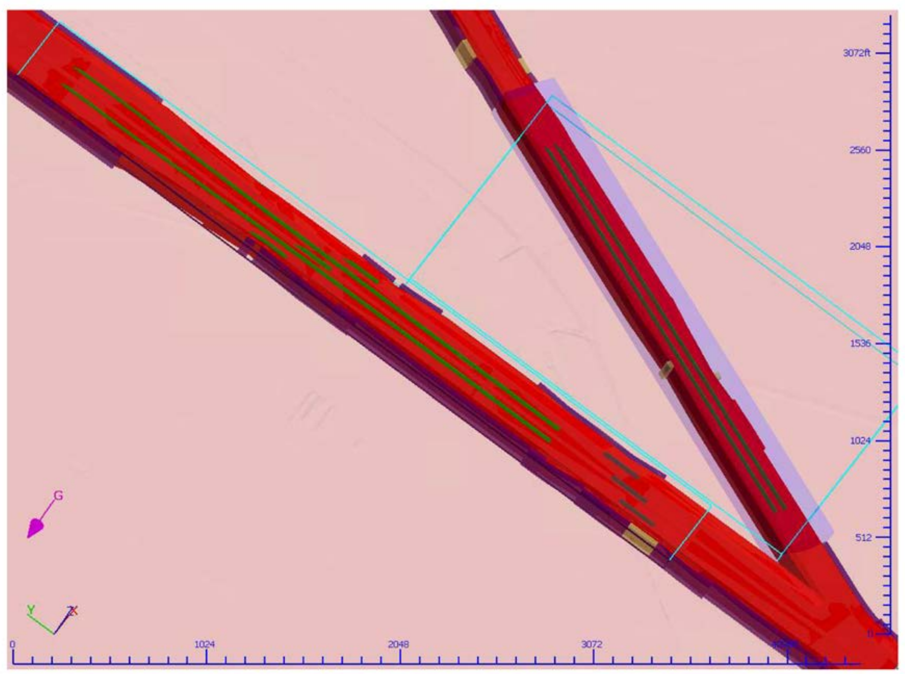

Figure 12. Platform support walls (green geometries) were added to the model to support the LAR and RH platform parks. Three additional shorter support walls were included in the LAR to support the weight of the future combined UPRR/Metro WSAB rail bridge. Each support wall included a nose geometry based on standard pier nose design. Existing bridge decks for the UPRR crossings were not included in the model, based on the assumption that they will be raised and built into the platform deck. The platform deck was not modeled since they are designed to be clear of the water. Additional 3D mesh blocks were defined to resolve the platform walls. The LAR mesh was extended downstream to also include the 710 Freeway piers and was comprised of 1.5 x 3 x 3 ft cells (resulting in approximately 12 million cells). The RH mesh was comprised of 2 x 2 x 2 ft cells (resulting in approximately 40 million cells, although many of these cells were outside the channel and dry).

The walls in the RH are straight and were aligned with the two existing UPRR RH bridge piers, which resulted in relatively even flow area. By contrast the walls in the LAR converge due to the narrowing of the LAR channel as it transitions down the 'chute' upstream of the existing UPRR bridge. Initial wall positions were estimated based on maintaining similar contraction ratios in all three bays. Initial simulations indicated that this resulted in a larger increase in free surface elevation in the center bay than the other bays. An adjustment to the wall positions reduced the elevation in the center bay but resulted in higher elevations in the East bay. Rather than continue to iterate manually on the wall positions, the initialization simulation without any bridge or wall geometries was analyzed. Two

streamlines were created starting at the upstream extent of where the walls were to be positioned and equally spaced across the channel. These streamlines represent the location where the water would flow without any obstacles within the channel and therefore using them as guides to define wall locations will result in minimal disturbance to the flow and relatively equal free surface elevations in the three bays. The final wall positions were asymmetrical, with the east wall continuing to converge slightly[8] from the bottom of the 'chute' to the downstream end.

Model results for each of the LAR and RH Platform Parks are discussed below.

[8] The wall position converges towards the channel centerline at a rate of about 1% (i.e., by 20 ft over ~2,000 ft length).

LAR Platform Park Results

The LAR Platform Park was analyzed for the LAR design peak conditions (the critical condition for the LAR). Results are presented for free surface elevation (Figure 13), flow depth (Figure 14), and depth averaged flow velocity (Figure 15). Each figure plots the baseline model results (left panel) and the LAR Platform Park results (right panel) to enable comparisons to be made. Model results indicate generally similar free surface elevations, depths, and velocities for the LAR Platform Park model as for the baseline model, and importantly the flow is predicted to remain below the top of the support walls (i.e., below the proposed platform) and within the channel.

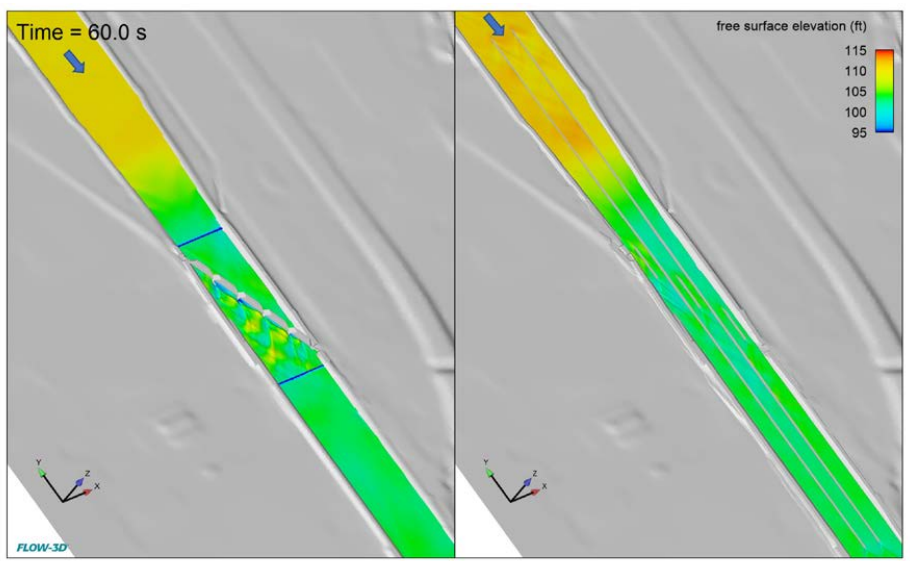

Figure 13. Modeled free surface elevation for baseline (left) and LAR Platform Park support walls (right) for the LAR design peak flow conditions (QLAR=140,000 cfs, QRH=44,000 cfs). Model results indicate similar free surface elevations for both conditions with the flow being contained in the channel. The position of the platform walls was designed and optimized using a 'streamline design' approach to achieve relatively equal free surface elevations within each of the three bays.

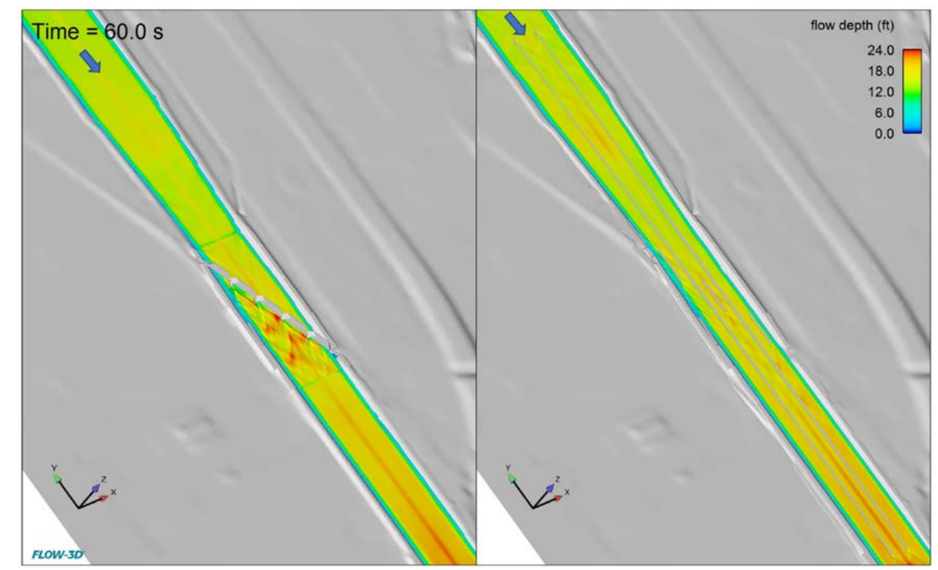

Figure 14. Modeled flow depths for baseline (left) and LAR Platform Park support walls (right) for the LAR design peak flow conditions (QLAR=140,000 cfs, QRH=44,000 cfs). Model results indicate similar depths for both conditions with the flow being contained in the channel. The position of the platform walls was designed and optimized to achieve relatively equal depths within each of the three bays.

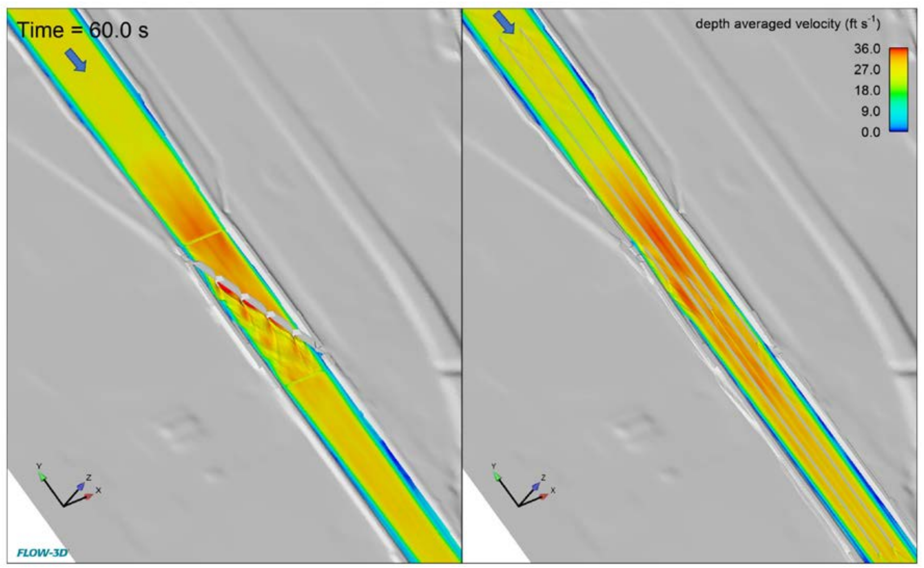

Figure 15. Modeled velocity for baseline (left) and LAR Platform Park support walls (right) for the LAR design peak flow conditions (QLAR=140,000 cfs, QRH=44,000 cfs).

The removal of the existing UPRR bridge piers and deck results in generally smoother flow, as illustrated in Figure 16. The existing bridge piers result in waves with an amplitude of approximately 10 ft (i.e., elevations of troughs at ~100 ft and crests of ~110 ft), while the proposed platform with support walls result in lower amplitude waves (e.g., cross wave formation on the shorter support walls/piers).

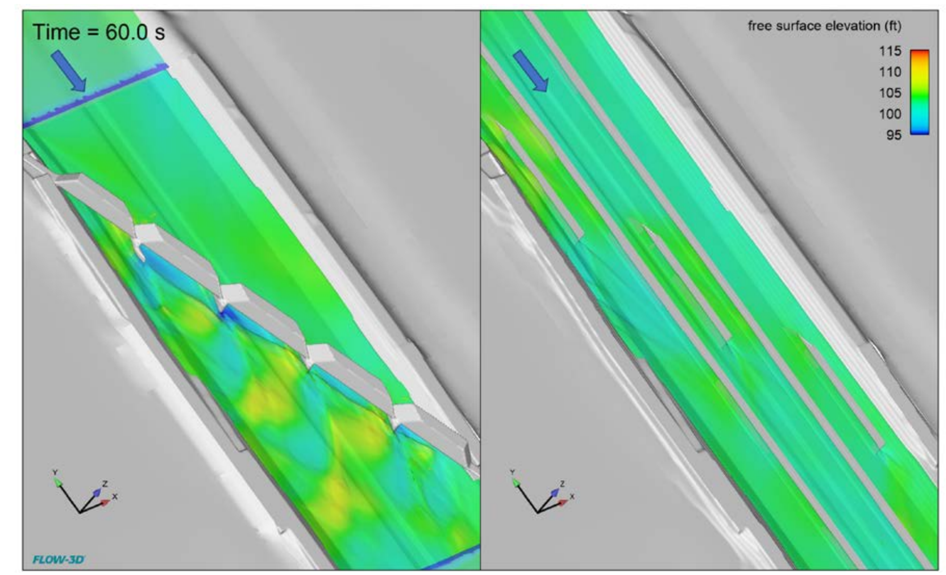

Figure 16. Close-up of modeled free surface elevation for baseline (left) and LAR Platform Park support walls (right) for the LAR design peak flow conditions (QLAR=140,000 cfs, QRH=44,000 cfs). Model results indicate similar free surface elevations for both conditions with the flow being contained in the channel. Removing the existing UPRR bridge results in generally improved flow conditions with lower amplitude waves.

The free surface elevation differences are more clearly illustrated in Figure 17 that plots modeled free surface elevation near the East and West banks as functions of distance along the channel. A localized increase near the West bank for the LAR Platform Park model is notable at a distance of approximately 1,600 ft. The localized increase of approximately 2 ft is of similar size to the waves in the baseline model and stays within the channel. The increase could likely be removed or reduced through modification to the channel bank (e.g., modification of bike path) and additional optimization of the wall positions (e.g., the additional support wall/pier near the West bank).

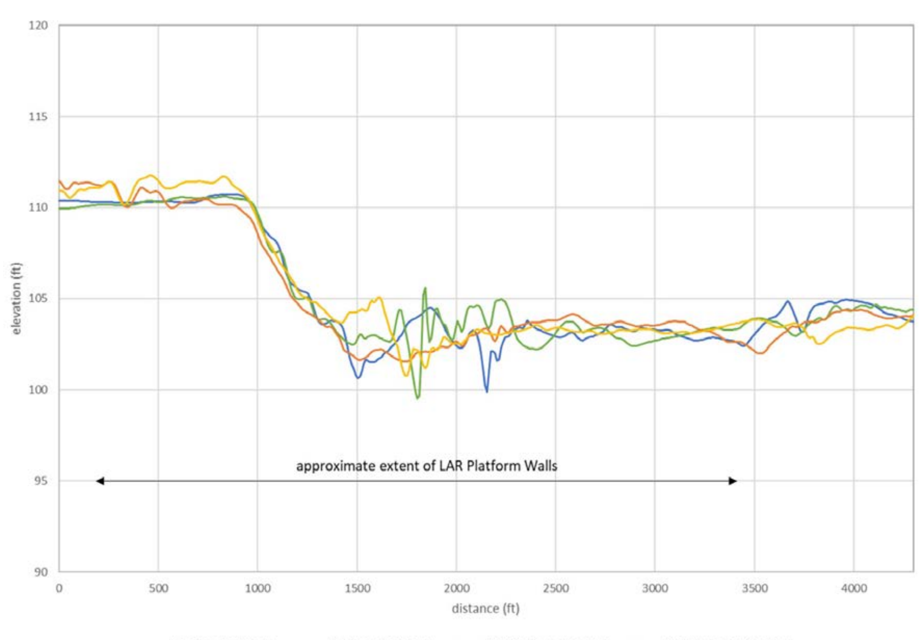

Figure 17. Free surface elevation as a function of distance near the East and West banks for the baseline model and the LAR Platform Park model for the LAR design peak flow conditions. Results indicate generally similar free surface elevations with the exception of the sharp depressions in the baseline model caused by the bridge piers (distance ~1,800 ft on the West bank ~2,200 ft on the East bank) and a localized increase near the West bank (distance ~1,600 ft) for the LAR Platform Park model. The localized increase of approximately 2 ft stays within the channel and could likely be removed or reduced through modification to the channel bank (e.g., modification of bike path) and additional optimization of the wall positions (e.g., the additional support wall near the West bank).

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The LAR Platform Park support walls were extended to approximately 300 ft upstream of the I-710 crossing, based upon a rule-of-thumb developed using 1-D model results (Geosyntec 2019). The CFD modeling confirmed that this was an adequate distance to avoid interference with the flow around the I-710 piers, as illustrated in Figure 18.

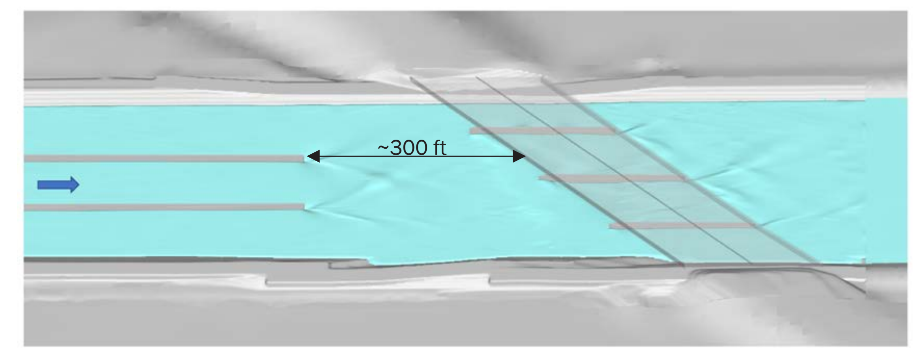

Figure 18. Close-up of the LAR Platform Park model results indicating the distance between the I-710 crossing.

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RH Platform Park Results

The RH Platform Park was analyzed for the RH design peak conditions (the critical condition for the RH). Results are presented for free surface elevation (Figure 19), flow depth (Figure 20), and depth averaged flow velocity (Figure 21). Each figure plots the baseline model results (left panel) and the RH Platform Park results (right panel) to enable comparisons to be made. Model results indicate generally similar free surface elevations, depths, and velocities for the RH Platform Park model as for the baseline model, and importantly the flow is predicted to remain below the top of the support walls (i.e., below the proposed platform). The overtopping that occurs upstream of the platform is of similar extent as for the baseline model.

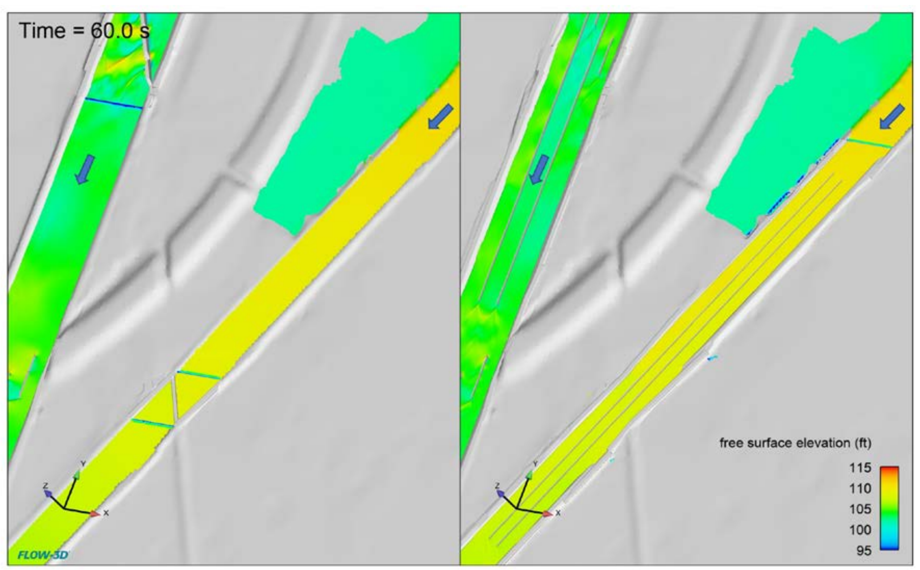

Figure 19. Modeled free surface elevation for baseline (left) and RH Platform Park support walls (right) for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs). Model results indicate similar free surface elevations for both conditions. The platform walls were positioned to align smoothly with the existing UPRR Bridge piers and include local thickening to support the rail load.

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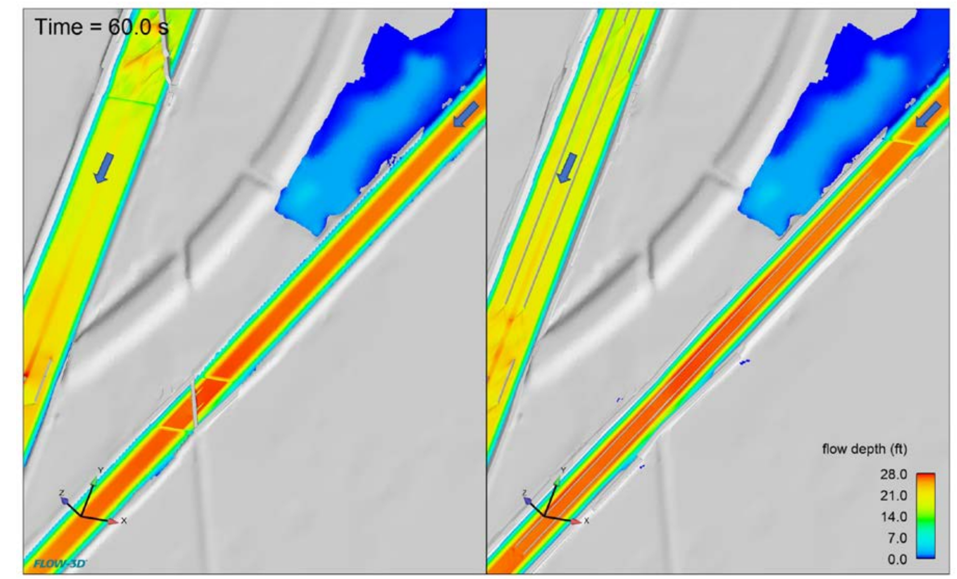

Figure 20. Modeled flow depth for baseline (left) and RH Platform Park support walls (right) for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs). Model results indicate similar flow depths for both conditions.

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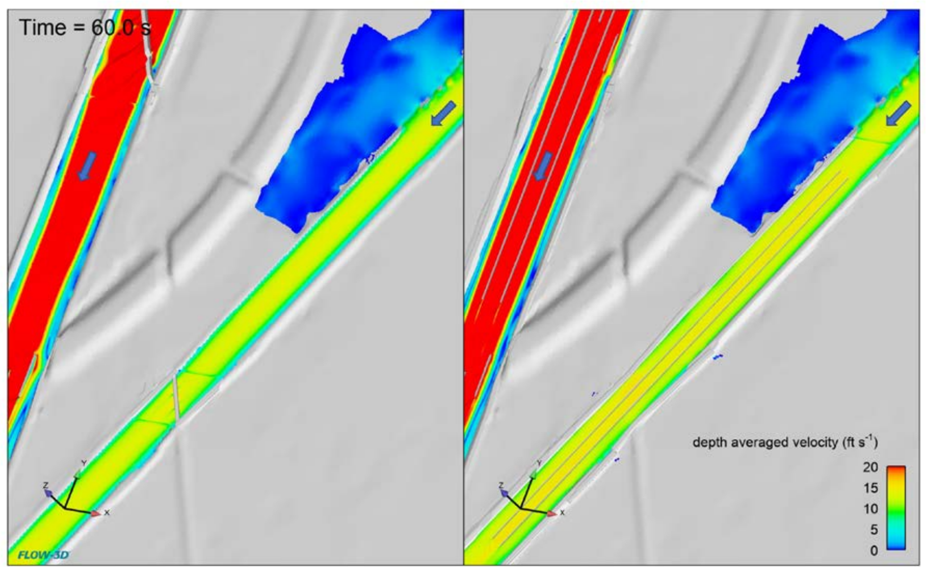

Figure 21. Modeled velocities for baseline (left) and RH Platform Park support walls (right) for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs). Model results indicate similar velocities for both conditions. Velocity in the RH channel is lower than the velocity in the LAR channel.

A close-up view of the baseline model UPRR bridge crossing and the same region with the crossing replaced by platform support walls is presented in Figure 22. Model results indicate similar free surface elevations for both conditions. The impingement of the flow on the existing UPRR bridge deck is illustrated. This impingement can be prevented with the platform park approach if the UPRR crossing is raised and integrated into the platform.

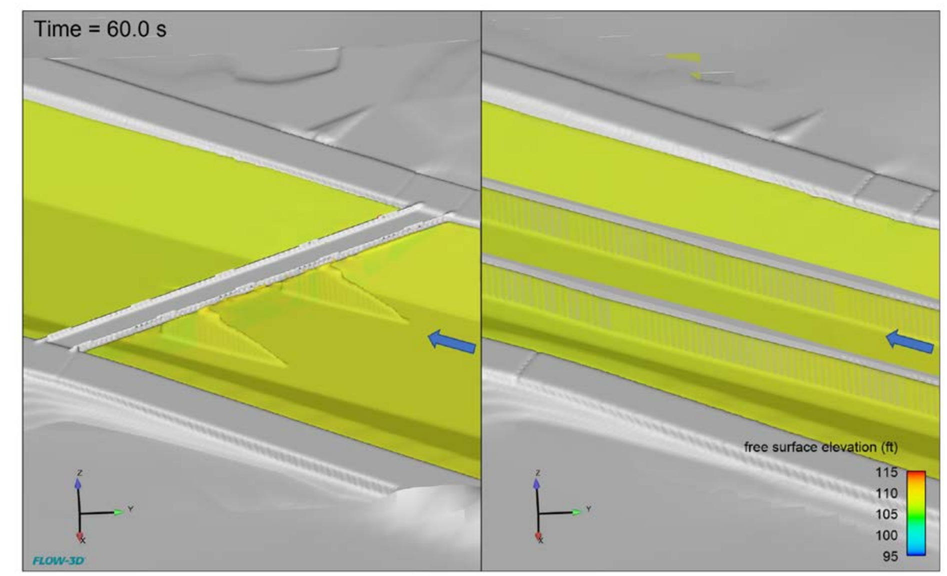

Figure 22. Modeled free surface elevations for baseline (left) and RH Platform Park support walls (right) for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs). Model results indicate similar free surface elevations for both conditions. The impingement of the flow on the UPRR bridge deck is illustrated (left). This impingement can be prevented with the platform park approach if the UPRR crossing is raised and integrated into the platform (right).

The modeled free surface elevations are plotted as a function of distance near the East and West banks in Figure 23. Results show generally similar free surface elevations upstream of the platforms, indicating minimal effect at upstream locations. Upstream and downstream of the existing UPRR bridge location, the RH Platform Park model results indicate some variation in free surface elevation, with elevations being approximately 1 ft higher on the West bank and 1 ft lower on the East bank. These local variations are likely caused by the general contraction in the channel geometry around the existing UPRR bridge and could be alleviated through iterative design of the wall locations, or through reshaping of the channel if the existing UPRR bridge is removed and built into the RH Platform Park.

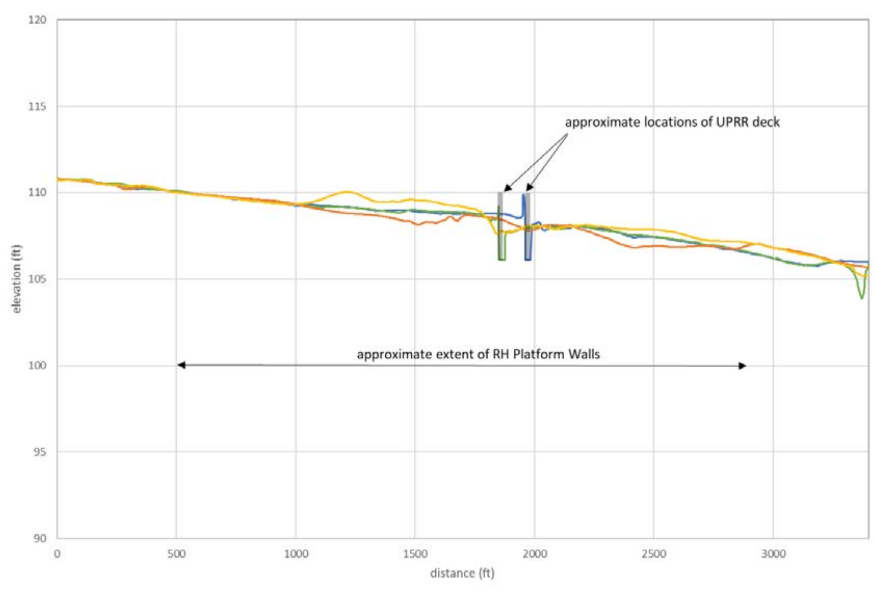

Figure 23. Free surface elevation as a function of distance near the East and West banks for the baseline model and the RH Platform Park model for RH design peak flow conditions. Results show generally similar free surface elevations upstream of the platforms, indicating minimal effect at upstream locations. The baseline model has sharp depressions and waves caused by the UPRR bridge deck. Upstream and downstream of the UPRR bridge location, the RH Platform Park model results indicate some variation in free surface elevation, with elevations being approximately 1 ft higher on the West bank and 1 ft lower on the East bank. These local variations are likely caused by the general contraction in the channel geometry around the existing UPRR bridge and could be alleviated through iterative design of the wall locations, or through reshaping of the channel if the existing UPRR bridge is removed and built into the RH Platform Park.

BLUE PARK AND SELA BRIDGE PARK

Proposed Features and Design Considerations

The proposed Blue Park will be located near the confluence and consists of several features, including terraces and ramps, the Braid Lab, and the low flow crossing and diversion channel. The proposed SELA Bridge Park will be immediately downstream of the low flow crossing. Due to the proximity of these features a single model was developed to assess the proposed modifications together. This model did not include the changes used to assess the LAR and RH Platform Parks.

For some of the features, several design iterations were performed before finding approaches that work. These iterations and an overview of the design approach are discussed below. Model results are then provided for the final modeled geometry.

Terraces and Ramps

Terrace geometries were designed to be as streamlined as possible, avoiding abrupt protrusions into the flow, and generally removing rather than adding to the channel. Initial modeling evaluated terraces on the East bank downstream of the SELA Bridge Park but indicated potential to cause localized waves and increased free surface elevations that may result in increased flow over the overflow weir. However, the mesh adjacent to the weir used 10-ft cells and was 2-D which may have magnified these effects. Additionally, the crest and backside of the weir was based upon LiDAR data, rather than geometries developed from as-built plans. A refined model, which could require a survey of existing conditions, may be required to better assess terraces in this location. Instead, the CFD model presented below includes a partial terrace in this location, with steps above the design water surface elevation.

Evaluation of the CFD model results indicated slower moving water from the RH inflow along the East bank, upstream of Imperial Highway bridge. The extra hydraulic resistance caused by the steps has a lower effect on slower moving water, so this location was identified as a preferred option and was included in the final CFD model. Prior to the CFD modeling the recommendations were to avoid changes in this region (i.e., at the confluence) due to the complexity of flow (Geosyntec 2019). This change in findings illustrates the benefit of performing CFD modeling.

A ramp was included within the terrace to provide ADA access. The initial model had the ramp facing upstream (due to better connectivity to the proposed Water Education Center) but the CFD results indicated that this could cause run-up along the ramp and increased impingement of water on the Imperial Highway bridge. The terrace and ramp were re-designed with the ramp facing downstream (per standard recommendations for supercritical channels) and re-evaluated in the CFD model. Results are presented shortly.

Braid Lab

The Braid Lab was a modification to the low flow channel upstream of Imperial Highway bridge where the channel splits into five braids as part of a transition of the low flow channel from the center to closer to the West bank. The initial design had vertical sides within the braids, but these were later flattened to provide less hydraulic resistance for the peak design flow conditions. Results of the CFD model are presented shortly.

Low Flow Crossing and Diversion

The Low Flow Crossing was positioned downstream of Imperial Highway bridge and upstream of the SELA Bridge Park, and was designed to enable a simple, low profile (i.e., low hydraulic resistance), bridge to cross the channel for pedestrian and maintenance vehicle access. Initial design of a channel contraction, or 'throat', to enable a simple steel 'plate' bridge to span the channel was conducted using 1-D HEC-RAS to evaluate low flow conditions. The goal was to contain the dry weather flows within the channel and to keep the low flow channel less than 30 inches deep (to avoid requirement for a handrail). Structural limitations for the 'plate' bridge required lengths of less than approximately 10 to 12 ft. Both single channels and twin channels were evaluated. Results of the analyses are presented in Table 4.

Table 4. 1-D HEC-RAS results for evaluating channel contractions

Configuration	Dry weather low flow	Throat width	Overtopping?
Single channel	120 cfs	10 ft	Limited
Single channel	120 cfs	12 ft	No
Single channel	100 cfs	10 ft	No
Twin channels	120 cfs	10 ft	No
Twin channels	120 cfs	6 ft	No

Based upon the analyses and structural trade-offs a single channel with a 10 ft throat width was implemented into the CFD model for testing at design peak flow conditions. Additionally, a side diversion channel was added upstream of the low flow crossing to enable an intake to be placed to provide water to be pumped to the adjacent Imperial Wetlands. Results of the CFD model are presented below.

Low Flow Check Dams

Check dams in the low flow channel were conceived to trap sediment and provide habitat between storm events and consisted of different degrees of blocking of the low flow channel. The designs were not analyzed for low flow conditions, where the goal would be to maintain the flow within the channel, but they were assessed for design peak flow rates in an initial CFD model. The check dams were implemented downstream of the SELA Bridge Park, adjacent to the overflow weir and in a portion of the channel where flow is supercritical. Initial model results indicated potential to cause localized waves and increased free surface elevations that may result in increased flow over the overflow weir. However, the mesh used for the check dams was 2-D which may have magnified these effects. A refined model and 3-D mesh may be required to better assess check dams in this location. Additional detailed studies of hydrodynamic forms for the check dams could also reduce the effects of the dams.

Generally, check dams would be better suited to portions of the channel with excess freeboard and with subcritical flow (to reduce wave effects). Check dams were not included in the final model.

SELA Bridge Park Piers

The SELA Bridge Park piers were previously evaluated using 1-D HEC-RAS modeling that indicated slender piers 3 ft to 4 ft wide and 30 ft long positioned approximately 500 ft to 600 ft downstream of Imperial Highway bridge would likely work (Geosyntec 2019). Initial CFD modeling indicated that longer piers may be feasible. Ultimately two piers, each 3ft wide and 60 ft long, together with standard shaped pier noses were implemented into the final model. Results are presented shortly.

Model Changes

Changes were made to the baseline models to represent the Blue Park and SELA Bridge Park features. A new channel geometry was built in Rhino by OLIN and Gehry Partners to represent the terrace, ramp, Braid Lab, low flow crossing and diversion. This was exported to STL and replaced the existing geometry in the baseline model. Additional STL geometries were built to represent the SELA Bridge piers, and these were implemented into the model. These features are illustrated in Figure 24.

The Imperial Highway grid used for the baseline model was initially set-up to be large enough to include these changes, but after additional iterations on the terrace position (i.e., moving it upstream of Imperial Highway) it was required to further extend the mesh in the upstream direction (Figure 24). An additional higher resolution mesh was also added around the terrace to better resolve the steps (Figure 24). The resolution of the Imperial Highway mesh did not resolve the underside of the 'plate' bridge crossing over the low flow channel, and therefore these simulations consider the low flow crossing to be blocked (e.g., by debris) which is a conservative assumption.

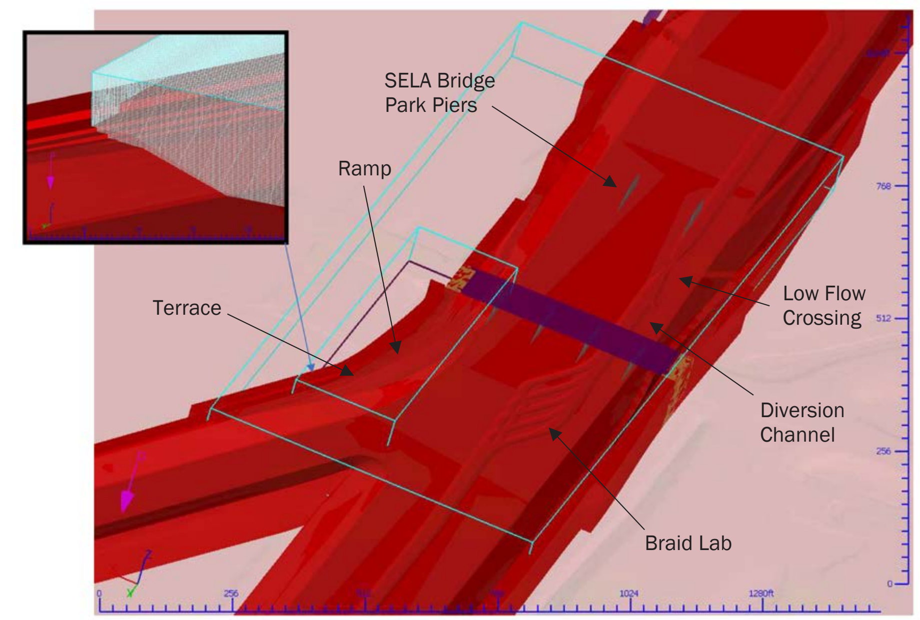

Figure 24. Channel geometries were rebuilt to include a terrace and ramp on the East bank of the LAR upstream of Imperial Highway and extending into the RH, low flow channel modifications including the Braid Lab, a low flow crossing, and a side diversion channel. Additional geometries were added to define the two new SELA Bridge Park piers. The Imperial Bridge refined computational mesh was extended further upstream to include the full terrace extent and maintained at a resolution of 1.5 x 2 x 2 ft (resulting in approximately 16 million cells). An additional nested mesh (see inset) was added around the terrace to better resolve the steps and used a resolution of 0.75 x 2 x 1 ft (approximately 8 million cells).

Design Peak Flow Results

The model was run for both the LAR design peak and RH design peak flows (Table 3). Both scenarios have the same flow of 184,000 cfs downstream of the confluence and indicated generally similar results. However, the RH design peak flows result in higher velocities near the terrace, and, therefore, the terrace has a larger effect on RH design peak flow scenario resulting in slightly higher free surface elevations than the LAR design peak flow scenario. Therefore, results are presented for the RH design peak flow, which is the critical driver.

Results are presented for free surface elevation (Figure 25 and Figure 26), flow depth (Figure 27), and depth averaged flow velocity (Figure 28). Each figure plots the baseline model results (left panel) and the Blue Park / SELA Bridge Park results (right panel) to enable comparisons to be made. Model results indicate generally similar free surface elevations, depths, and velocities for the Blue Park / SELA Bridge Park model as for the baseline model, and importantly the flow is predicted to remain within the channel, except at the designated overflow weir.

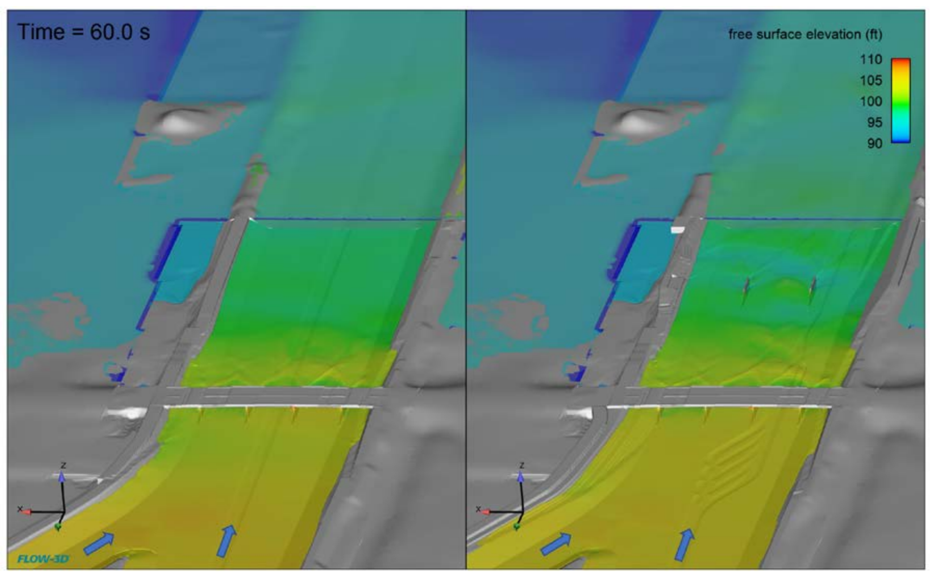

Figure 25. Modeled free surface elevation for baseline (left) and the Blue Park modifications (right) for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs). The LAR design peak flow condition generally produced similar results, except for near the terraces where the RH design peak flow condition was found to be the critical driver in terms of slightly increasing the free surface elevation upstream of the Imperial Highway Bridge. The model results are shown with some transparency to enable the details of the geometry to be visualized. Further conclusions on the free surface elevations are drawn in Figure 26 and Figure 29.

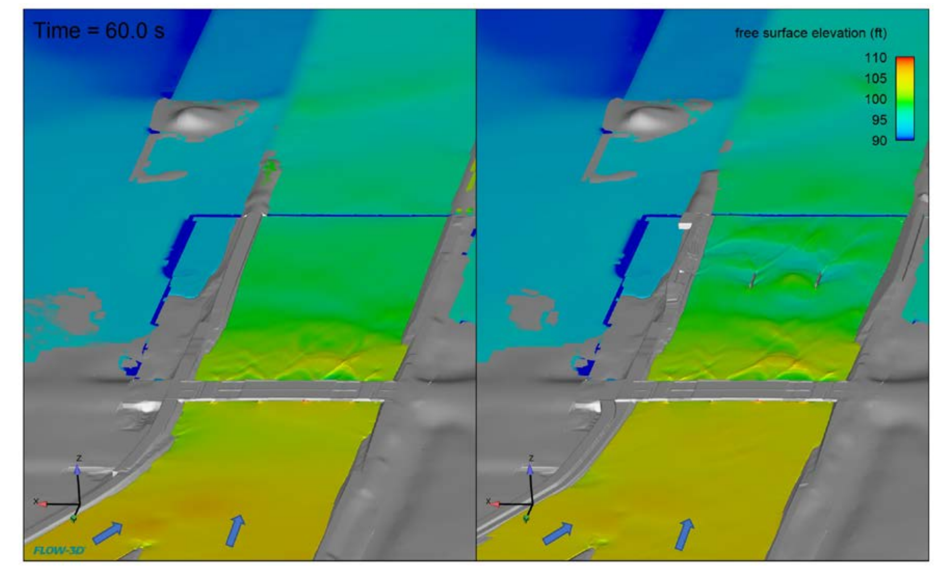

Figure 26. Same modeled conditions as Figure 25 with no transparency to enable clearer evaluation of free surface elevations. Model results are generally similar with a few notable differences: the Blue Park model (right) has higher free surface elevation adjacent to the terrace and upstream of Imperial Highway Bridge than the baseline model (left), the SELA Bridge Park piers result in some localized changes in free surface elevation, including small increases upstream of the piers and decreases downstream of the piers (these are further evaluated in Figure 29). The Braid Lab, low flow crossing, and diversion channel had minimal impact. Both models predicted similar elevations and flow over the weir on the East Bank, but it is noted that this was modeled with a relatively coarse (10 ft) and 2-D computational mesh.

Closer inspection of the free surface elevations in Figure 26 illustrate some subtle but notable differences between the baseline model and the Blue Park model. The Blue Park model has higher free surface elevation adjacent to the terrace and upstream of Imperial Highway Bridge than the baseline model which is likely due to the additional hydraulic resistance (i.e., wetted perimeter) of the terraces. The SELA Bridge Park piers also result in some localized changes in free surface elevation, including small increases upstream of the piers and decreases downstream of the piers. The Braid Lab, low flow crossing, and diversion channel had minimal impact.

TECHNICAL REPORTS: HYDRAULICS REPORT

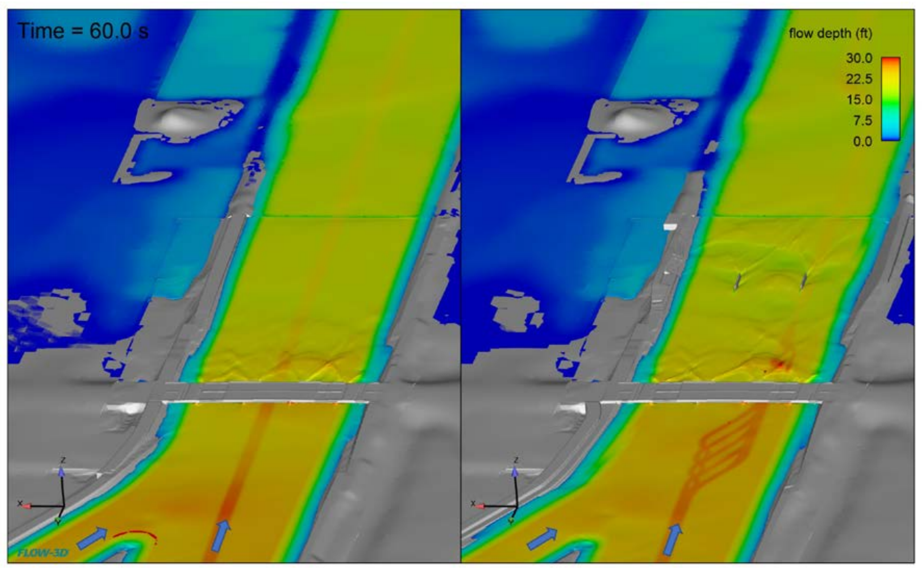

Figure 27. Modeled flow depth for baseline (left) and the Blue Park modifications (right) for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs). The Braid Lab and low flow crossing are apparent in the Blue Park model results as regions with higher depths. Flow depths are otherwise generally similar, except near the terrace upstream of Imperial Highway Bridge and locally around and downstream of the SELA Bridge Park piers.

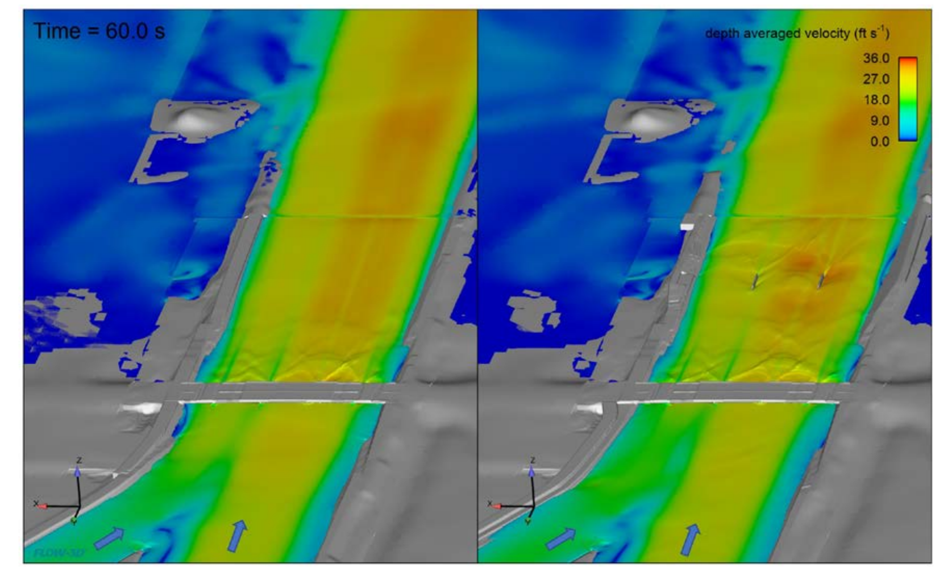

Figure 28. Modeled velocity for baseline (left) and the Blue Park modifications (right) for the RH design peak flow conditions (QLAR=131,100 cfs, QRH=52,900 cfs). Flow velocities are generally similar, except near the terrace upstream of Imperial Highway Bridge where the terraces result in slower (and deeper) flow and locally around and downstream of the SELA Bridge Park piers that cause faster (and shallower) flow.

The free surface elevations are further evaluated in Figure 29 that plots modeled free surface elevation near the East and West banks as functions of distance along the channel. The locations of Imperial Highway bridge, proposed SELA Bridge Park, and top of wall (or bank) including the overflow weir are annotated on the plot to aid in the interpretation. The proposed Blue Park modifications result in slightly higher free surface elevations upstream of and under Imperial Highway, particularly on the East bank. Figure 29 does not indicate impingement at this specific location, but closer examination of the model results indicates some locations near the East bank may experience intermittent times where water reaches the underside of the bridge deck. However, the general degree of bridge impingement is similar for the Blue Park model as it is for the baseline model. The free surface elevations are slightly increased upstream of the proposed SELA Bridge Park and may be mitigated (if necessary) by raising the parapet walls. The free surface elevation downstream of the SELA Bridge is generally lower due to water accelerating between the bridge piers. The free surface elevations are slightly higher at the upstream end of the East Bank overflow weir and generally similar further downstream.

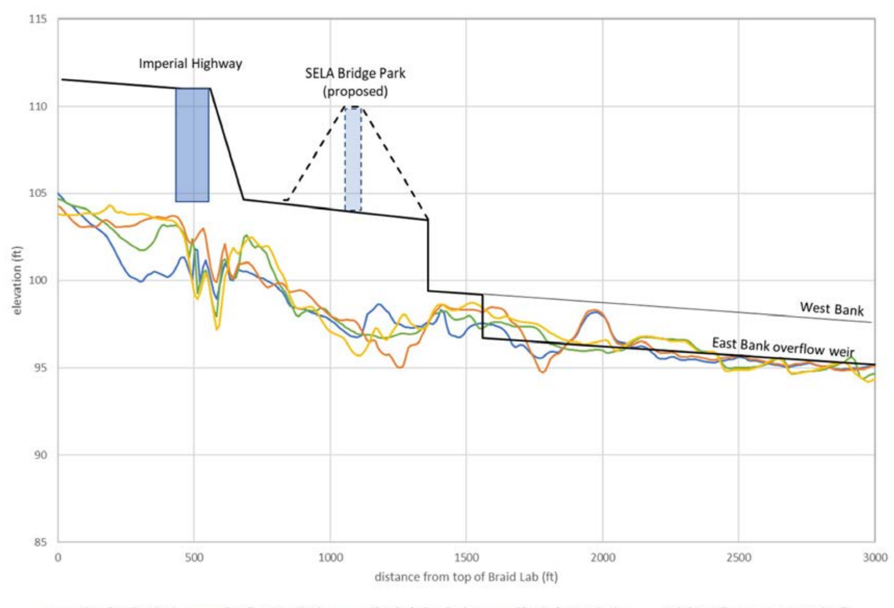

Figure 29. Free surface elevation as a function of distance near the East and West banks for the baseline model and Blue Park model for RH design peak flow conditions. Results indicate higher free surface elevations upstream of and under Imperial Highway, particularly on the East bank. The plot does not indicate impingement at this specific location, but closer examination of the model results indicates some locations near the East bank may experience intermittent times where water reaches the underside of the bridge deck. However, the general degree of bridge impingement is similar for the Blue Park model as it is for the baseline model. The free surface elevations are slightly increased upstream of the proposed SELA Bridge Park and may be mitigated (if necessary) by raising the parapet walls. The free surface elevation downstream of the SELA Bridge is generally lower due to water accelerating between the bridge piers. The free surface elevations are slightly higher at the upstream end of the East Bank overflow weir, and generally similar further downstream.

The modeled flow rates over the weir on the East Bank are presented in Figure 30. The Blue Park models result in lower average flow rates than the baseline models but have more temporal variability. The results indicate that the proposed Blue Park modifications are likely feasible, but longer duration simulations with a more detailed model may be required to better evaluate the temporal variability.

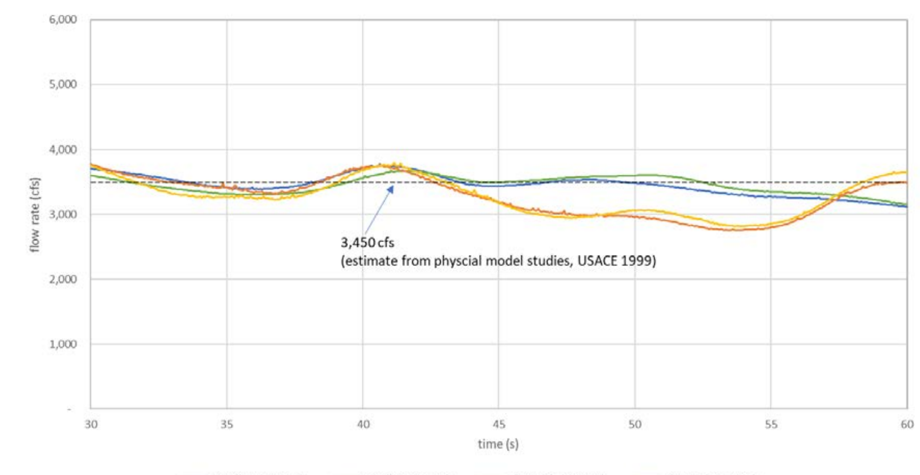

Figure 30. Modeled flow rates over the weir on the East Bank as a function of time. The time-axis starts at 30 seconds to remove the initial transients that occur in the simulation. Results are provided for the baseline model and the Blue Park model for both the LAR and RH design peak flow conditions. The Blue Park models result in lower average flow rates than the baseline models but have more temporal variability. Longer duration simulations with more detailed model geometry and mesh may be required to fully evaluate these conditions.

The flow phenomena summarized and described here are generally dynamic and unsteady and should be further evaluated in future phases using refined geometries and meshes (e.g., for the overflow weir) and unsteady analyses (e.g., temporal averaging of results).

BLUE PARK LOW FLOW CONDITION

To evaluate low flow conditions, the CFD model was shortened to only include the mesh block used to resolve the Blue Park geometries as illustrated in Figure 31. The inflow (QLAR = 100 cfs and QRH = 0 cfs) and outflow boundary conditions were applied directly to the mesh block. The grid size was decreased to 1 x 1 x 1 ft and further refined to a vertical size of 0.25 ft in the lower part of the channel. This resulted in almost 100 million cells, but most of these were empty. An additional smaller mesh was defined at the low flow crossing to resolve the flat 'plate' bridge crossing the channel (Figure 31). This mesh used a cell size of 0.5 ft in the horizontal direction, and cells ranging from 0.25 ft to 1 ft in the vertical direction and comprised approximately 0.5 million cells.

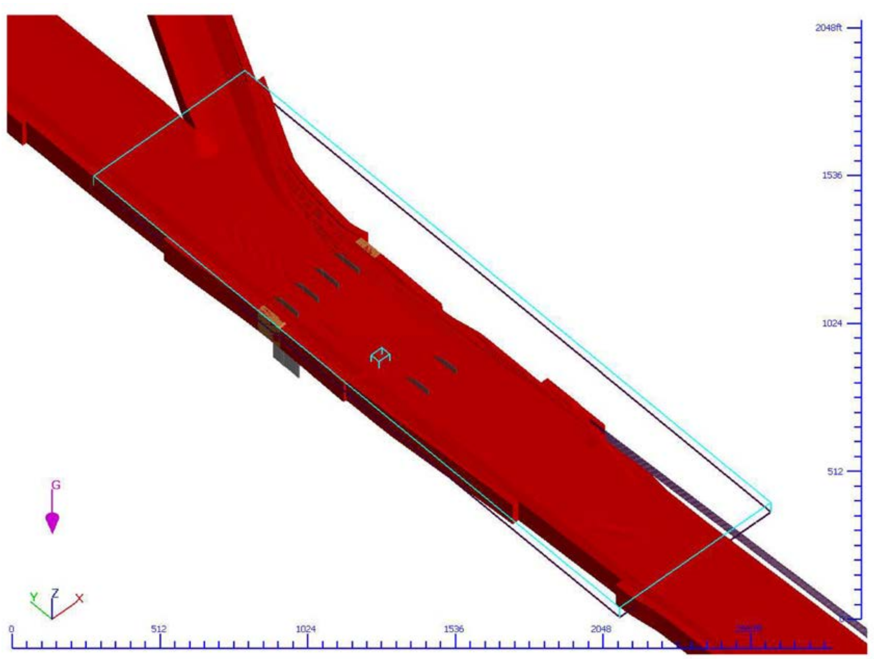

Figure 31. Shortened CFD model used to evaluate low flow conditions. The grid size was decreased to 1 x 1 x 1 ft and further refined to a vertical size of 0.25 ft in the lower part of the channel. This resulted in almost 100 million cells, but most of these were empty. An additional smaller mesh was defined at the low flow crossing to resolve the flat 'plate' bridge crossing the channel. This mesh used cell size of 0.5 ft in the horizontal direction, and cells ranging from 0.25 ft to 1 ft in the vertical direction and comprised approximately 0.5 million cells.

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Results of the model are presented in Figure 32. Flow depths in the low flow channel at 100 cfs of flow reach approximately 2 feet upstream of the low flow crossing. This is contained within the proposed low flow channel which maintains the current low flow geometry. Velocities through the Braid Lab and upstream of the low flow crossing will decrease from the current value in the low flow channel of approximately 6 fps to between approximately 1 and 4 fps in the Braid Lab and 3 to 4 fps upstream of the low flow crossing. For a short distance downstream of the low flow crossing the velocity reaches as high as 12 fps in localized spots and is generally in the range of 6 to 9 fps.

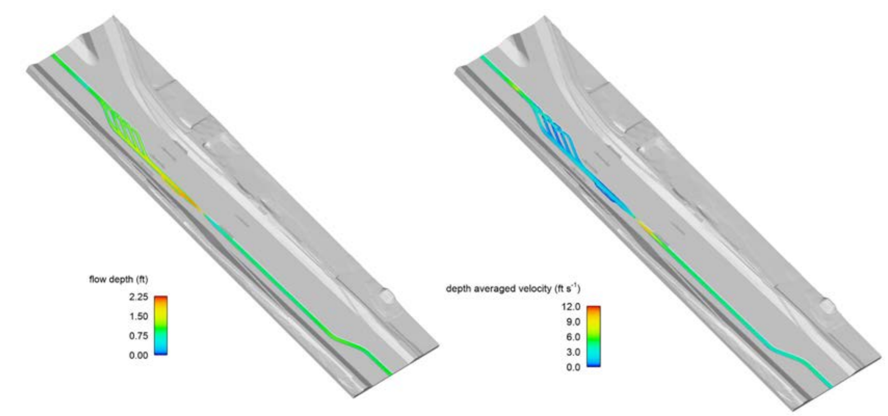

Figure 32. The model results indicated flow depths (left) in the low flow channel at 100 cfs of flow would reach approximately 2 feet upstream of the low flow crossing. This is contained within the low flow channel. Velocities (right) through the Braid Lab and upstream of the low flow crossing will decrease from the current value in the low flow channel of approximately 6 fps to between approximately 1 and 4 fps in the Braid Lab and 3 to 4 fps upstream of the low flow crossing. Downstream of the low flow crossing the velocity reaches as high as 12 fps in localized spots and is generally in the range of 6 to 9 fps.

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CONCLUSIONS

CFD modeling was performed to further evaluate feasibility of concepts and modifications within the proposed Rio Hondo Confluence Area Project. A baseline model was developed to represent current conditions and verified by comparisons to USACE physical modeling studies from the 1990's. Feasibility evaluations were largely based on comparisons of free surface elevations for design peak flow conditions of a modified model to the baseline model. Based on the evaluations the following project components are likely feasible: LAR Platform Park, RH Platform Park, Blue Park elements including Braid Lab, terrace and ramp, low flow channel crossing and side diversion channel, and the SELA Bridge Park. Check dams in the low flow channel downstream of the SELA Bridge Park indicated potential to cause localized waves and increased free surface elevations that may result in increased flow over the weir and were not included in the final model. Additional more refined modeling would be needed to re-evaluate check dams in these locations and/or to evaluate the feasibility in more suitable locations.

Findings for each are summarized below.

- LAR Platform Park
  - It was assumed that the existing UPRR bridge will be removed and rail crossings raised and integrated into the platform, which will have hydraulic benefits. Additional analyses are required to assess alternative option to maintain the existing UPRR bridge.
  - The platform park will be supported by two 3 ft thick walls running the entire platform length, and three additional support walls/piers to support the extra Metro WSAB load at the rail crossing.
  - The locations of the walls were refined by 'streamline analyses' using the CFD results to obtain relatively equal free surface elevations in each of the three bays. Additional refinements and/or local channel modifications may be required in future phases.
  - The channel side slopes were assumed to be smoothed to concrete to reduce hydraulic resistance. Additional analyses should be performed to assess if this is necessary.
  - The CFD results indicated that terminating the platform support walls 300 ft upstream of the I-710 bridge should be sufficient to avoid impacts to the I-710 bridge.

  - Hydraulic impacts of the UPRR rail crossing in its existing alignment require further analyses.
- RH Platform Park
  - It was assumed that the existing UPRR bridge will be removed and rail crossings raised and integrated into the platform, which will have hydraulic benefits. Additional analyses are required to assess an alternative option to maintain the existing UPRR bridge.
  - Platform park will be supported by two 3 ft thick walls running the entire platform length. These walls will be thickened locally to 5 ft to support the extra load at the rail crossing.
  - The locations of the walls were based on lining up with the existing two UPRR bridge piers. Additional refinements and/or local channel modifications may be required in future phases.
  - The channel side slopes were assumed to be smoothed to concrete to reduce hydraulic resistance. Additional analyses should be performed to assess if this is necessary.
  - The CFD results indicated that terminating the platform support walls 500 ft upstream of the confluence with the LAR should be sufficient to avoid disturbing flow at the confluence.
  - The upstream end of the platform should begin a safe distance from the Garfield Avenue bridge. This bridge was not included in the current modeling.
  - Hydraulic impacts of the UPRR rail crossing in its existing alignment require further analyses.
- Blue Park
  - The Braid Lab, low flow crossing, and side diversion channel have minimal impact on the free surface elevation of the design peak flow.

- A terrace and downstream facing ramp are likely feasible on the East bank of the LAR upstream of Imperial Highway bridge and extending a short distance into the RH channel.
  - This location was identified as a lower velocity region in initial CFD modeling.
  - The CFD model indicated a slight increase in free surface elevation in the vicinity of the terrace, but still well within the channel and if necessary, this could be mitigated by raising the walls slightly.
  - The Imperial Highway bridge is impinged by flow for this configuration, but the degree of impingement is similar to the baseline model.
  - The flow rate over the weir on the East Bank is slightly lower on average than the baseline model but has a higher temporal variability.
  - The flow phenomena and impingement are generally dynamic and unsteady and should be further evaluated in future phases using refined geometries and meshes (e.g., for the overflow weir) and unsteady analyses.
- SELA Bridge Park
  - Two slender piers, up to approximately 3 feet thick, and 60 feet long, with standard debris noses are feasible.
  - The free surface elevation upstream of the SELA bridge may increase slightly, but this can be mitigated by adding some small walls to meet freeboard requirements.

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USACE 2004. Stormwater Management Plan, Phase I, HEC-RAS Hydraulic Models, Rio Hondo Channel Reach 4 and Lower Los Angeles River Reaches 3b, 3a, and 2. July 2004.

USACE 2005. Los Angeles County Drainage Area Upper Los Angeles River and Tujunga Wash HEC-RAS Hydraulic Models, Final Report, July 2005.

END OF MEMORANDUM

TECHNICAL REPORTS

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PROJECT DESCRIPTION

The Los Angeles River – Rio Hondo Confluence Area Project (RHCAP) is located in Southeast Los Angeles County, California stretching between the cities of South Gate, Lynwood, and Downey. The overarching project consists of multiple project sites with a focus on multiple large platforms spanning the Los Angeles and Rio Hondo Rivers. These platforms serve the primary purpose of revitalizing the LA River: adding connectivity between neighborhoods on either side of the river and providing cultural and recreational space for nearby communities.

Working with OLIN, Gehry Partners, and Geosyntec, Magnusson Klemencic Associates, Inc. is providing design assist with regards to structural criteria and systems in the master plan and conceptual phase of this project.

There are twelve distinct project sites, of which five have structural components that MKA is providing guidance on. Figure 1 below includes a summary of the twelve project sites with the highlighted sites being those with structural elements.

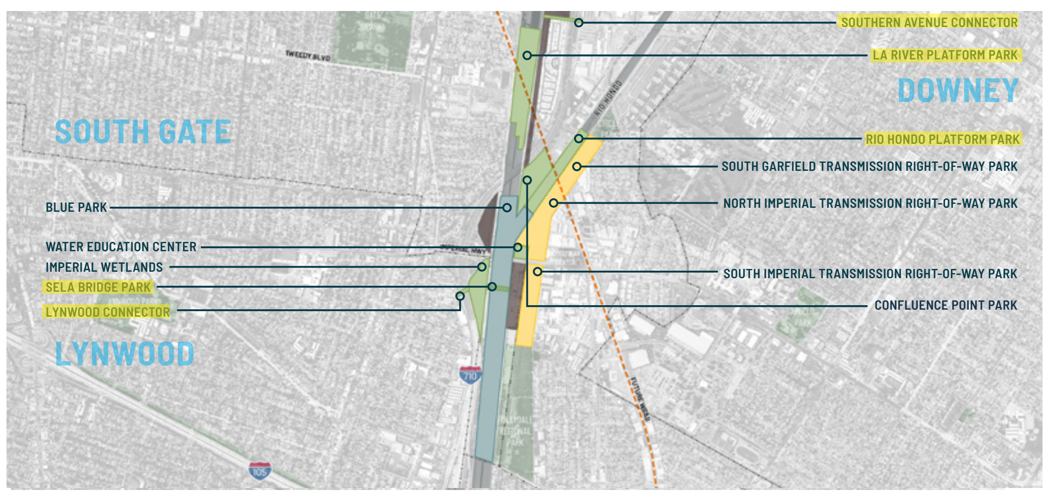

Figure 1. LA River RHCAP Project Sites

LA RIVER AND RIO HONDO PLATFORM PARKS

The two platform park sites are similar projects which involve the addition of structure to cap existing channels to create a combined total 33 acres of community park land. Covering a 2,650ft length of the Los Angeles River and a 2,330ft stretch of the Rio Hondo with cross-channel spans of approximately 450ft and 250ft respectively, these large-scale bridge structures are to support heavy landscaping loads while maintaining a minimal structural profile.

Conceptual design for both platform parks includes a slab and long span girder system supported by two in-river continuous concrete walls and parapet walls at existing levees on each side of the channel. For

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the slab-and-girder structure, various systems have been considered. These systems include the following:

- Precast pre-stressed concrete Deck Bulb Tee girders with top flange of girder forming slab, connected with a topping slab
- Precast pre-stressed concrete I-shaped girders supporting precast plank slab
- Built up steel plate girders supporting precast plank slab

Selection of the structural framing system depends on future studies addressing the cost effectiveness and constructability of each option, but the approximate girder depths studied for this report applies to all three systems. In addition to the girder depth, a precast plank slab would account for ~8"-12" of added structural depth while a topping slab would account for 4" of added structural depth. For three-span conditions, the center span girders may remain continuous over wall supports and extend up to 20ft past support as cantilevers. Exterior spans to be supported at end of cantilevers with bearing splice connections. This alternative to three simply supported spans reduces support complexities, results in more efficient girder design, and may allow for more repetition in girder lengths as channel width transitions over the length of the platform.

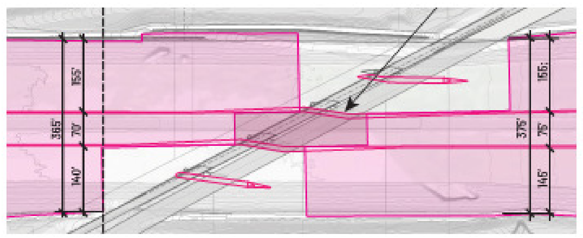
Another factor impacting girder depths are span lengths between in-river concrete wall and on-levee parapet wall supports. The placement of the walls is in turn determined by hydraulic requirements for river flow including considerations for existing and future rail crossings through both platforms. Currently, there are existing steel truss bridges for the Union Pacific Railroad (UPRR) that cross the two channels at a diagonal; the bridges are supported by four pier supports at the LA River, and two at the Rio Hondo. Additionally, a future extension of the Metro West Santa Ana Branch (WSAB) is to run parallel to the existing UPRR line with similar pier support requirements. As the presence of the existing bridges and piers have detrimental hydraulic and structural implications for the platform addition, the primary platform condition, Option A, includes the removal of the existing UPRR bridges and placing both rail lines atop the new platform structures.

An alternative platform condition, Option B, would maintain the existing UPRR bridges, provide clearances for the new WSAB line, and stop the platform on each side of the rail crossing. This alternative has not been fully assessed for hydraulic feasibility and has numerous non-optimal structural aspects. Option B requires platform support walls to align with existing UPRR bridge piers, resulting in much longer and unequal spans at LA River (See Figure 2).

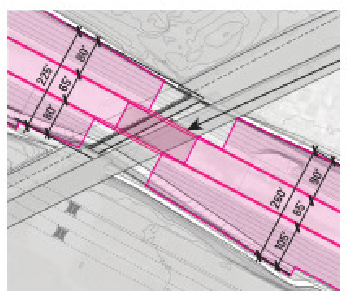
Option A - Mostly Equal Spans   Option B - Unequal Spans

Figure 2. LA River Platform North End Spans

It also reduces the platform areas near the rail crossings as the diagonal skew of the rail-channel intersection results in extremely long spans that are difficult to support at the rail-platform interface. Figure 3 shows platform setbacks required at both rivers due to the rail crossing.



Platform Setback at LA River



Platform Setback at Rio Hondo

Figure 3. Option B, Platform Setbacks for Rail Crossing

Due to the differences in platform width and supporting wall locations of the two options at each river, girder depth and spacing vary. Below is Table 1 which notes the maximum span lengths, approximate girder depths, and girder spacing for Options A and B for each platform. Figures 4-14 on the following pages provide additional details and information on the varying requirements for each condition.

Table 1. Platform Girder

Platform Condition	Max Span Length	Approximate Girder Depth	Girder Spacing
LA River, Option A	160 ft	6 ft	10 ft on center
LA River, Option B	190 ft	8 ft	8 ft on center
Rio Hondo, Option A	105 ft	5 ft	10 ft on center
Rio Hondo, Option B	105 ft	5 ft	10 ft on center

FIGURE 4. LA RIVER PLATFORM PARK | OPTION A | PLAN VIEW

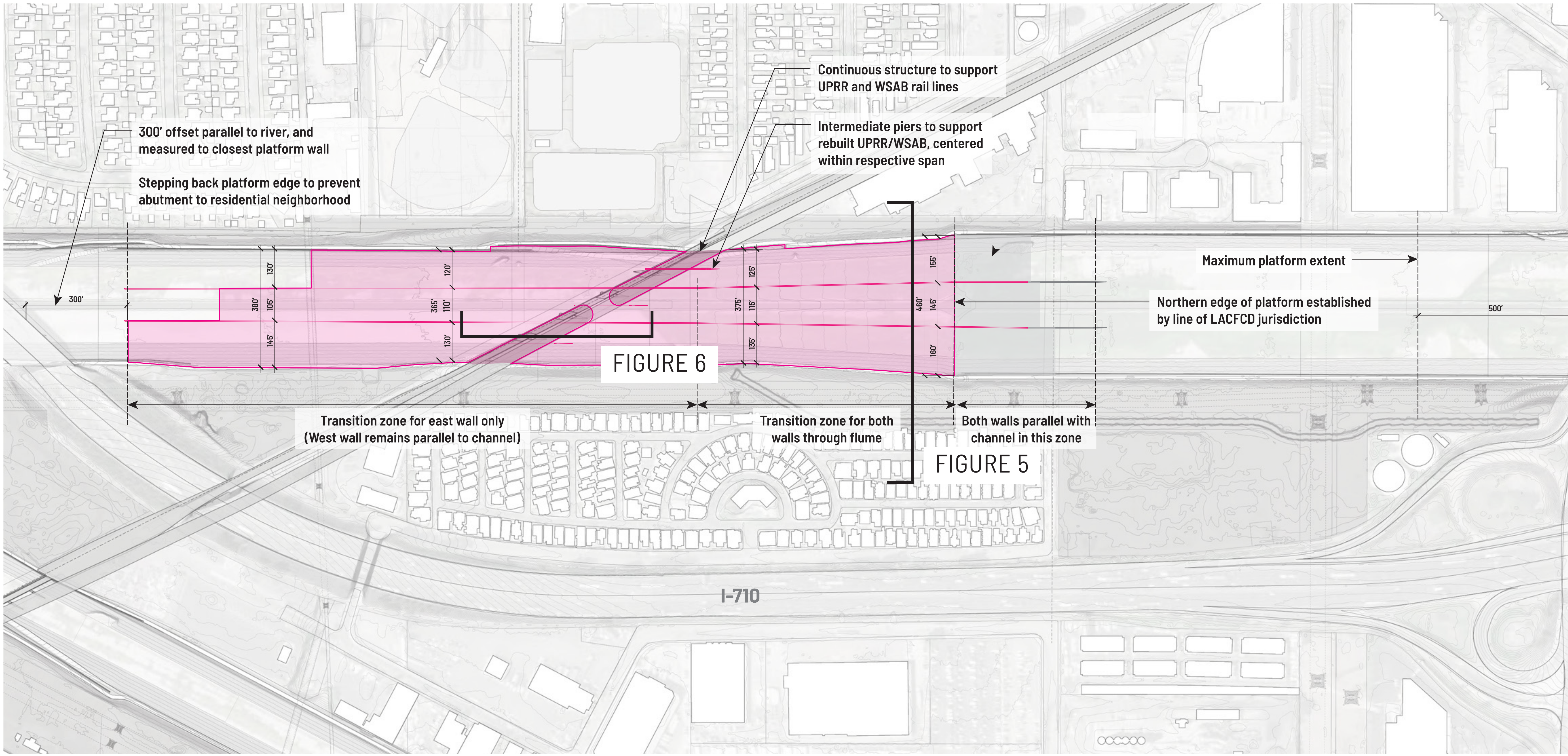


FIGURE 5. LA RIVER PLATFORM PARK | TYPICAL SECTION

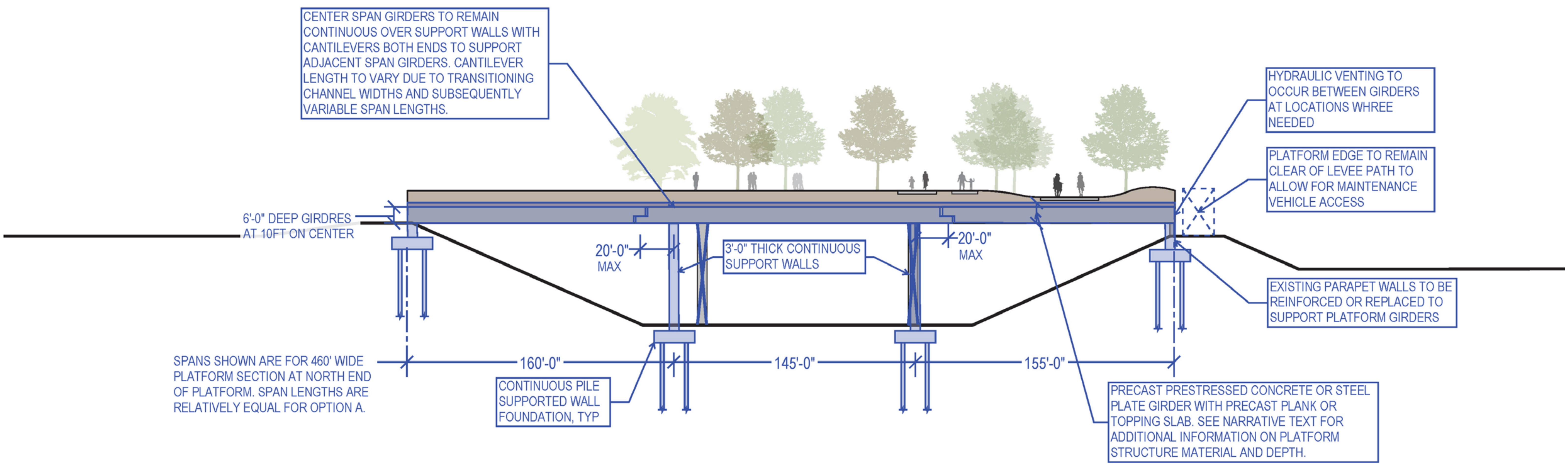


FIGURE 6. LA RIVER PLATFORM PARK | OPTION A | RAIL CROSSING SECTION

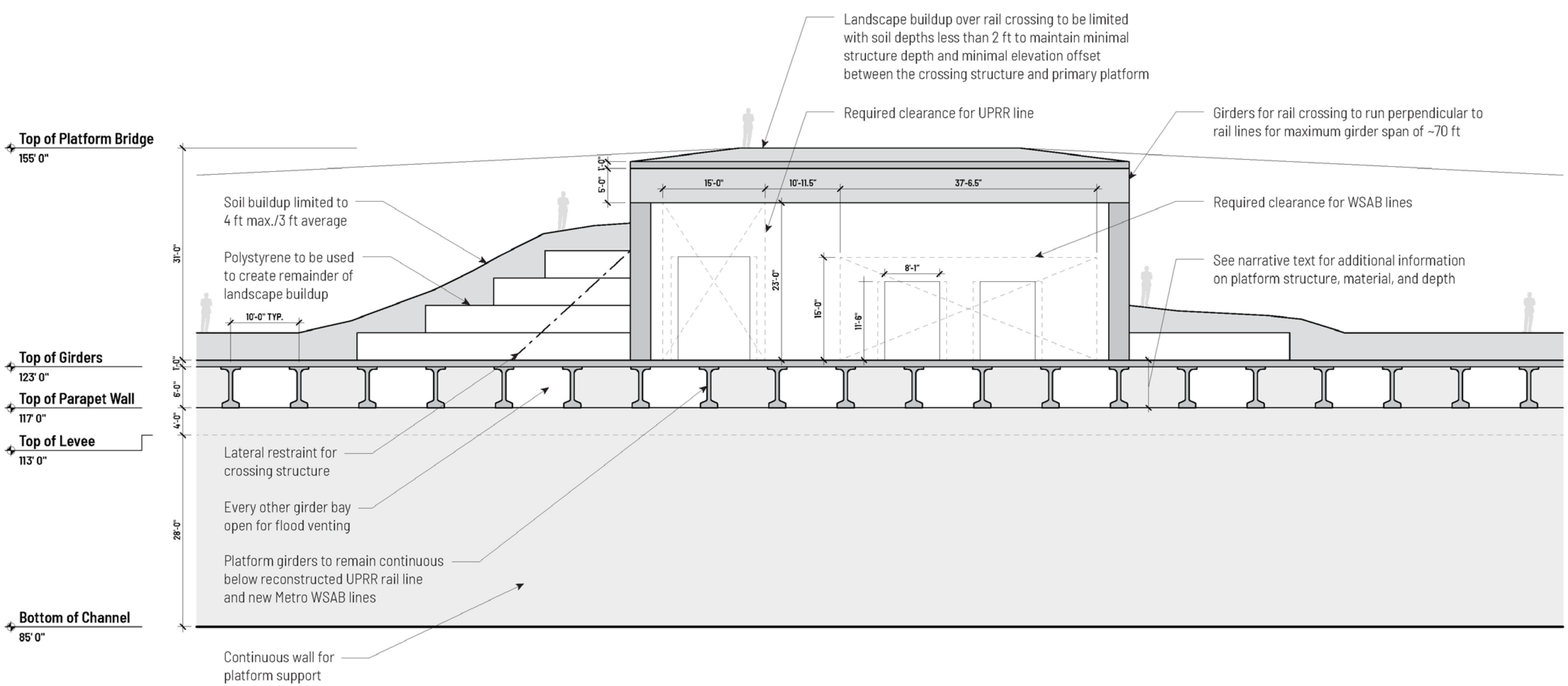
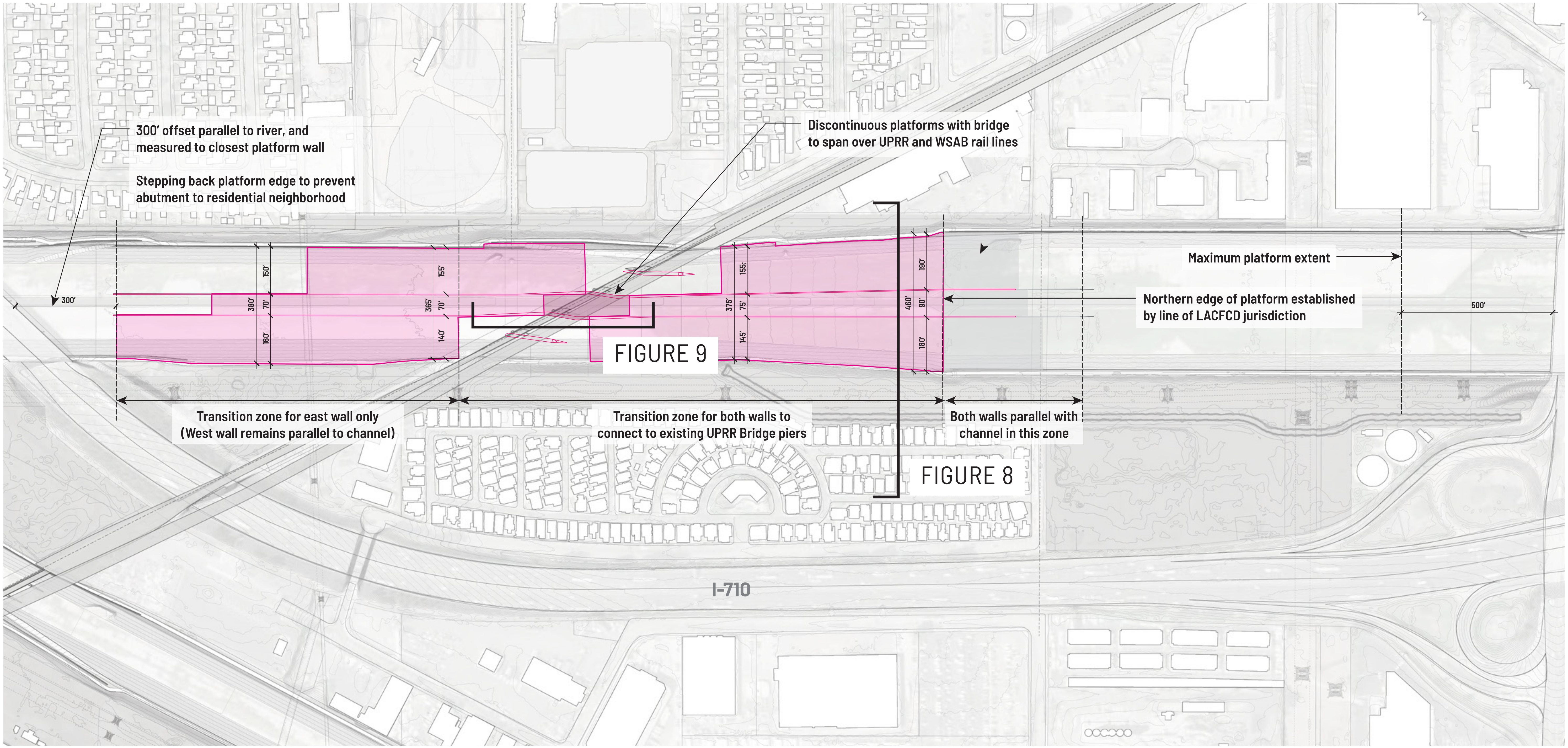


FIGURE 7. LA RIVER PLATFORM PARK | OPTION B | PLAN VIEW



TECHNICAL REPORTS: STRUCTURAL REPORT

FIGURE 8. LA RIVER PLATFORM PARK | OPTION B | TYPICAL SECTION

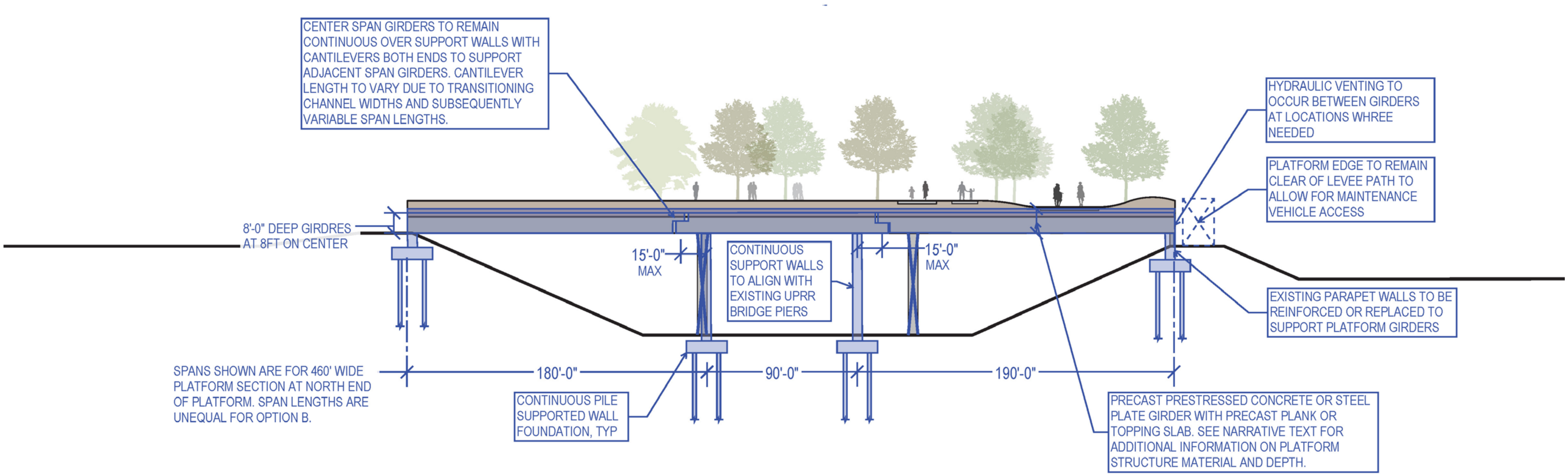


FIGURE 9. LA RIVER PLATFORM PARK | OPTION B | RAIL CROSSING SECTION

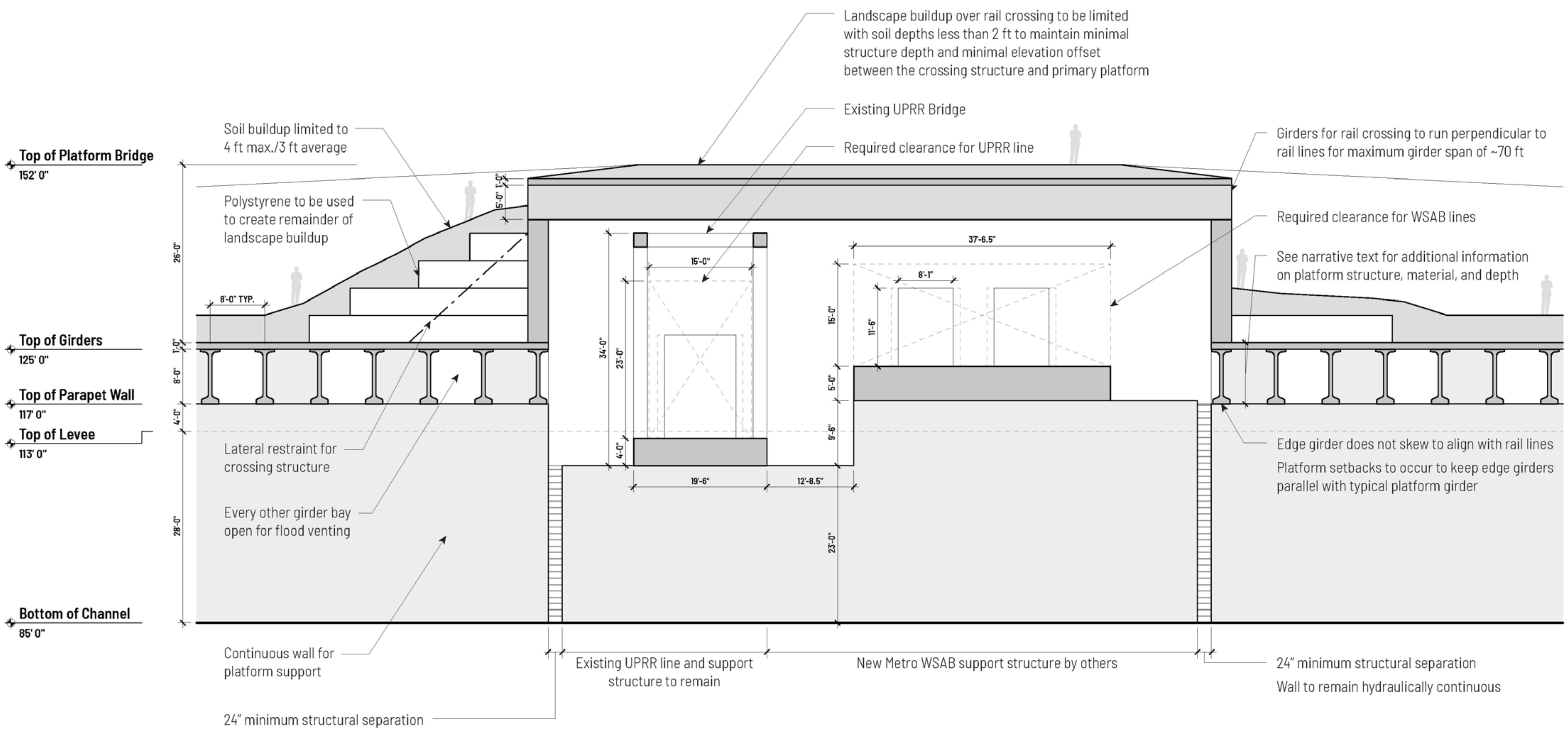


FIGURE 10. RIO HONDO PLATFORM PARK | OPTION A | PLAN VIEW

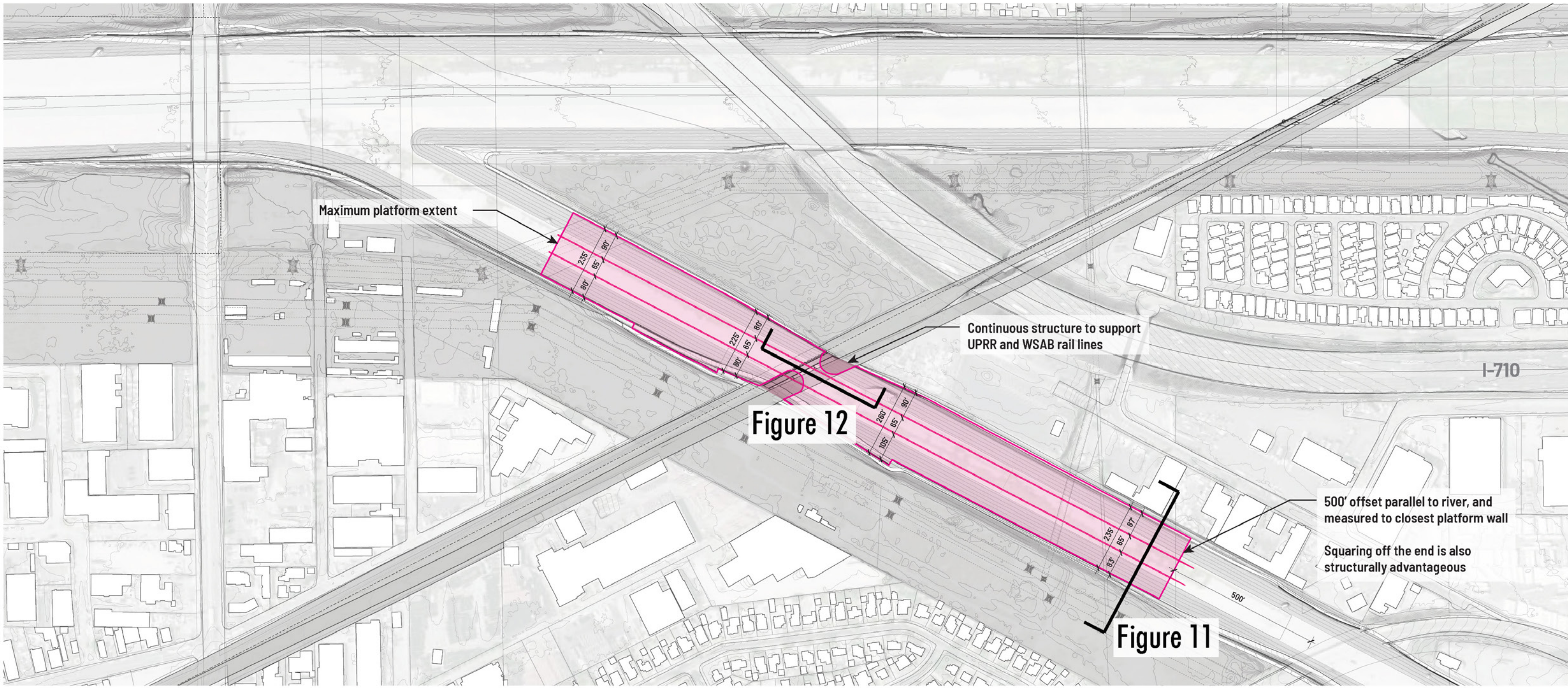


FIGURE 11. RIO HONDO PLATFORM PARK | OPTIONS A + B | TYPICAL SECTION

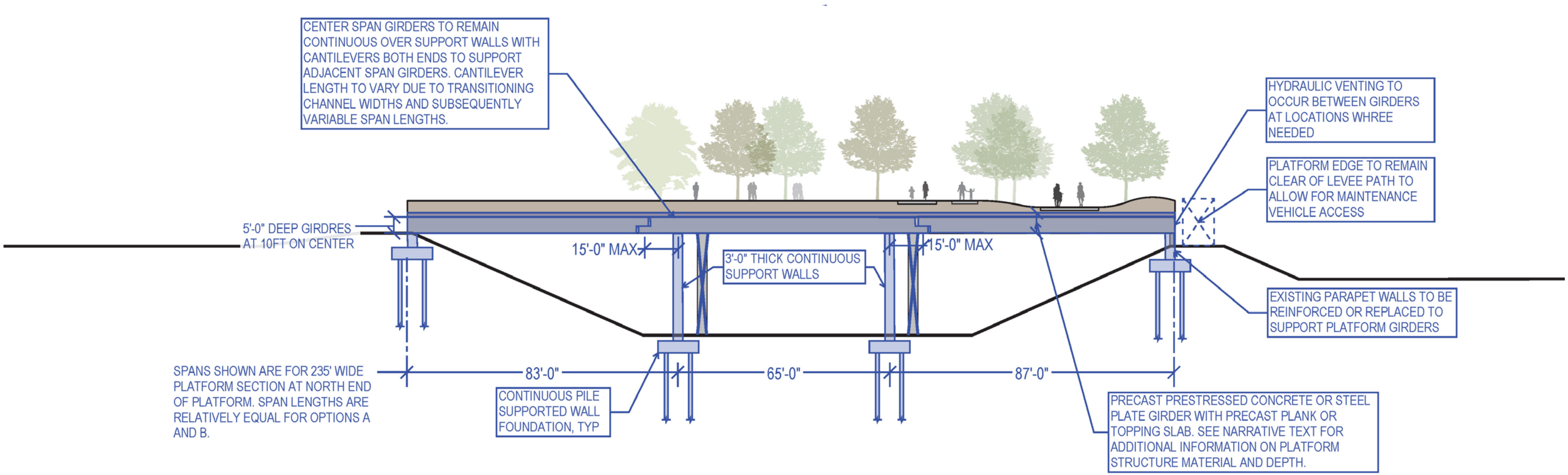


FIGURE 12. RIO HONDO PLATFORM PARK | OPTION A | RAIL CROSSING SECTION

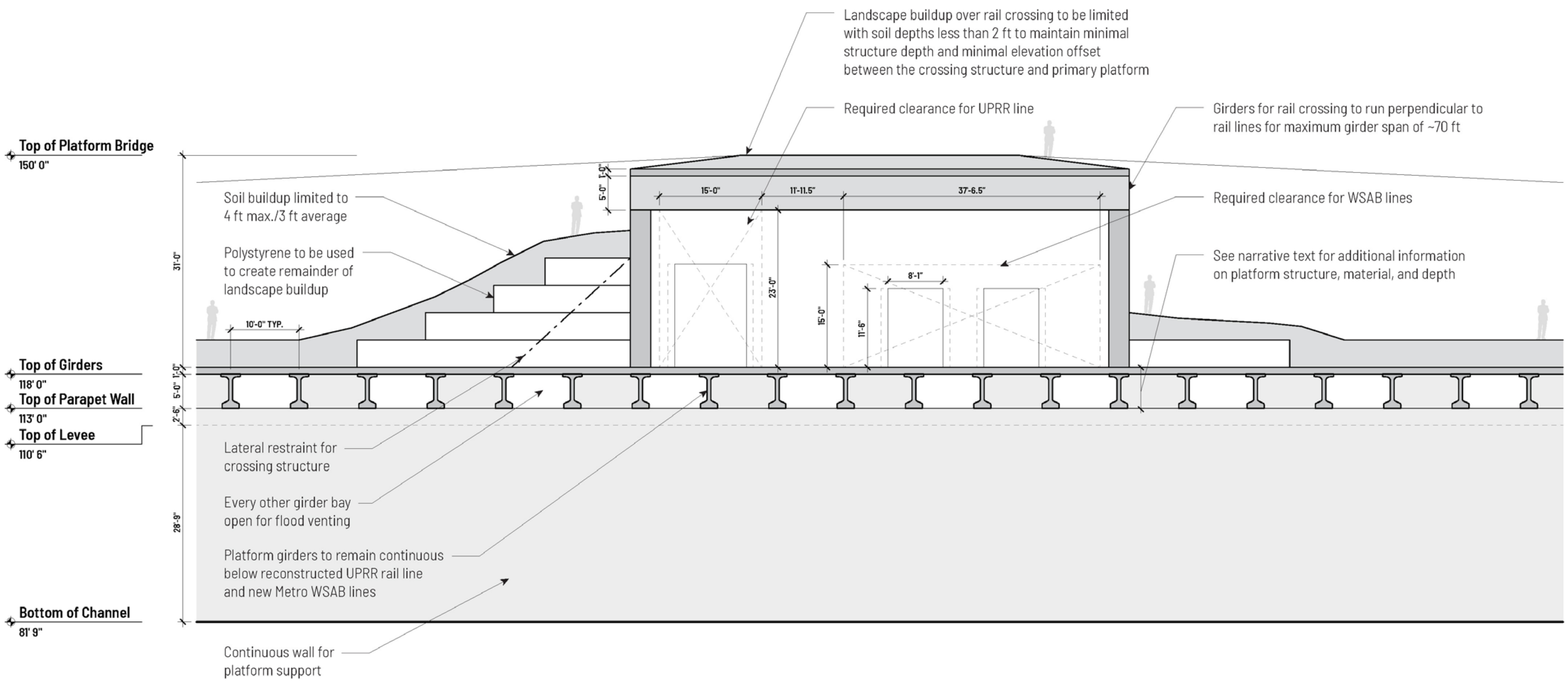


FIGURE 13. RIO HONDO PLATFORM PARK | OPTION B | PLAN VIEW

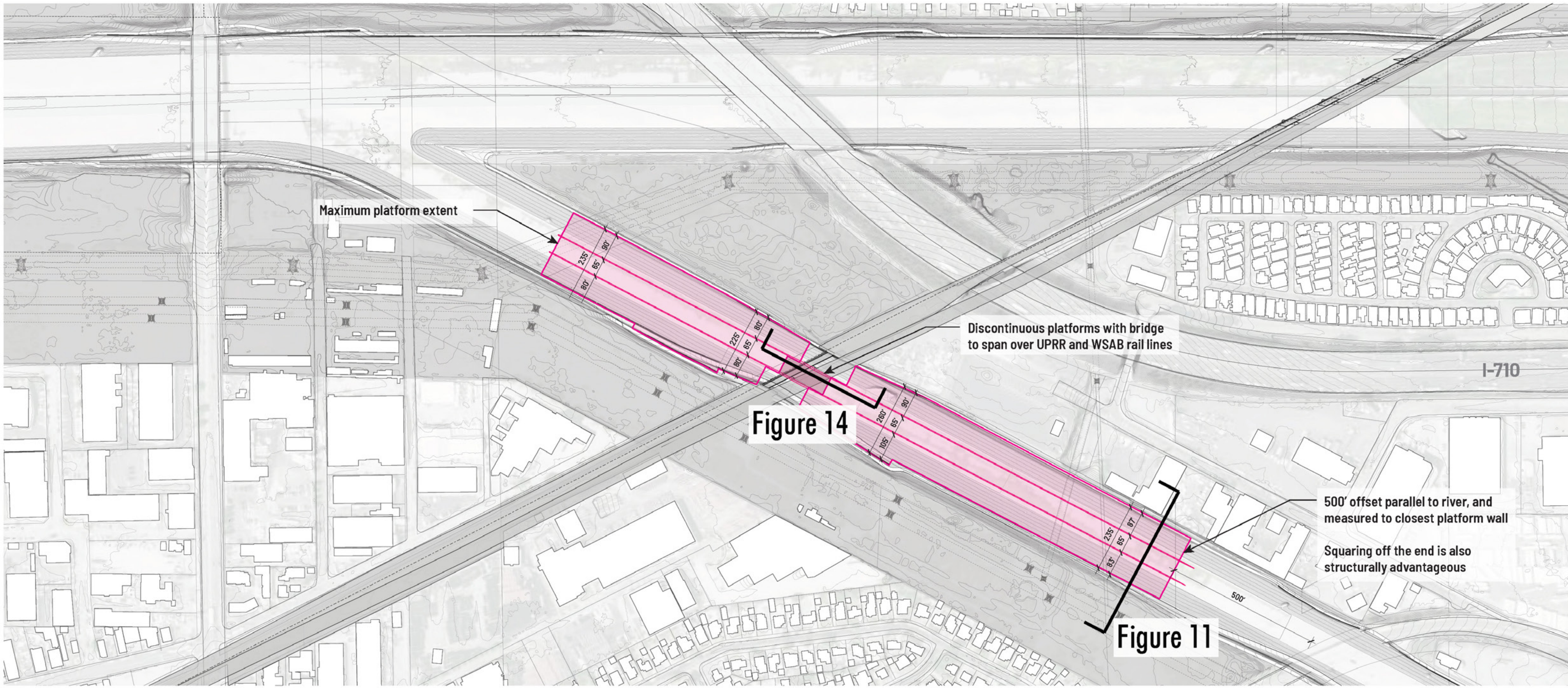


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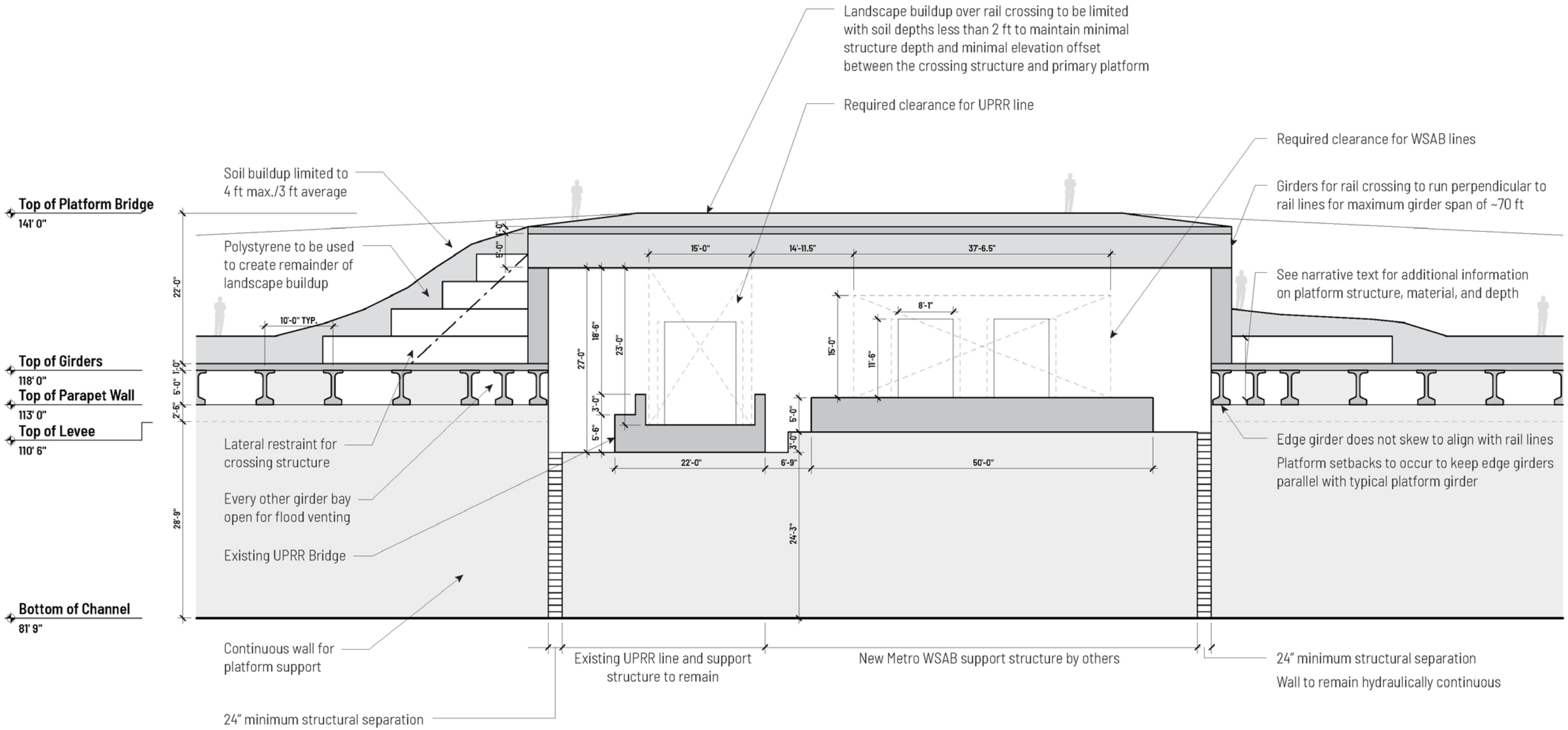
FIGURE 14. RIO HONDO PLATFORM PARK | OPTION B | RAIL CROSSING SECTION



SOUTHERN AVENUE AND LYNWOOD CONNECTORS

The Southern Avenue and Lynwood Connectors are pedestrian bridges that cross over interstate I-710 with intermediate supports at the highway median. At Southern Avenue, an additional span is provided to cross the intersection of Southern Ave and Frontage Rd E. Supported on steel framed elevated structures at the ends for access and concrete pile caps and columns at the center highway median, the spans crossing I-710 and roadway intersection consist of box truss bridges. Intermediate steel framing supporting a composite concrete slab will make up the walkway structure supported by a lower horizontal truss to limit the height required for pedestrians and cyclists to climb. Access ramps to bring pedestrians from grade up to bridge elevations to be steel framed. Secondary framing will be provided at the bridge pivot point to support signage or art installation for gateway design.

SELA BRIDGE PARK

The SELA Bridge Park must cross an overall 410ft span above the LA River. Despite its significant span, the bridge requires a shallow structural depth to closely align its ends with the elevation of the levees and LA River Trails. The bridge park is anticipated to include large sculptural installations above pier locations and trellis framing for lightweight planting support.

The bridge can be supported in several configurations. Options under exploration include various corbelled/cantilevered piers, precast concrete and steel materiality, as well as openings or "apertures" in the bridge deck to allow views below. The materiality being investigated include precast prestressed concrete girders considered for consistency with platform bridges and steel plate girders considered for flexibility for soil depressions and ease of attachment for trellis structures above. These schemes require customized structural solutions that must be optimized in the context of the desired shallow structural depth and design intent.

BUILDING CODES

The project is to be designed in accordance with the following building and material codes:

BUILDING CODE

- International Building Code, 2018 Edition (IBC 2018) with reference to American Society of Civil Engineers, *Minimum Design Loads for Buildings and Other Structures,* 2016 Edition (ASCE 7-16).
- CALTRANS, California Amendments to AASHTO LRFD Bridge Design Specifications (Sixth Edition)
- USACE EM 1110-2-2100, Stability Analysis of Concrete Structures
- USACE EM 1110-2-2104, Strength Design for Reinforced Concrete Structures

MATERIAL CODES

- Reinforced Concrete: American Concrete Institute, *Building Code Requirements for Structural Concrete and Commentary,* 2014 Edition (ACI 318-14).
- Structural Steel: American Institute of Steel Construction, *Specification for Structural Steel Buildings,* 2016 Edition (ANSI/AISC 360-16).

LOADING CRITERIA

The framing for each structure will have to be designed to support superimposed dead and live loads. Anticipated loads at the LA River and Rio Hondo Platform Parks include soil for landscaping, trees, water features, hardscape, site walls, seating, vehicles in isolated areas, public assembly and single-story recreation buildings. For the Southern Avenue and Lynwood Connector bridges, gravity loading is limited to pedestrians and cyclists with no vehicle traffic or landscaping anticipated. At the SELA Bridge Park, the current concept design includes the loading for light landscaping supported by trellises and pedestrian, not vehicular traffic.

A summary of the potential building-specific loading criteria follows. This loading meets or exceeds the requirements of the IBC and incorporates loading requirements specific to this project.

GRAVITY LOADING

The following loads are in addition to the self-weight of the structure. The minimum loading requirements have been taken from Table 4-1 of ASCE 7. Loads are given in pounds per square foot (psf).

Table 2. Gravity Loads

Use	Live Loading
Public Assembly	100 psf (not reduced)
Light Storage	125 psf (not reduced)
Mechanical/Electrical	125 psf (not reduced)
Roof	20 psf
Stores (Retail)	100 psf + partitions
Landscaping	240 psf to 480 psf (2ft to 4ft of soil)

SPECIAL LOADS

Sections of the platform parks are to provide support for rebuilt UPRR and new Metro WSAB rail lines in the preferred configuration, Option. In this case, the platform structure will need to be designed for (as yet to be determined) train loading for strength, deflection, and vibration criteria.

WIND DESIGN CRITERIA

Wind loading is not anticipated to control the design of the lateral force-resisting system for the platforms. Any new building structures on the bridges will be designed in accordance with the IBC and ASCE 7 requirements.

SEISMIC DESIGN

Seismic loads for the platforms will be resisted by the platform structures acting as horizontal diaphragms spanning between the support walls and/or piers located in the river and at the levees. Walls and piers will be designed to resist these loads in shear and flexure, transmitting shear, flexural and overturning forces into the pile caps and piles. Expansion joints provided for thermal expansion will have to be detailed to provide lateral restraint in one or both directions while also allowing thermal expansion.

Lateral seismic loads will be determined in accordance with the IBC, ASCE 7, and AASHTO requirements.

THERMAL EXPANSION

Given the length of the structure thermal expansion and contraction will need to be considered in the detailing of the girder supports.

MATERIALS

The material properties used for the design include the following:

Table 3. Structural Steel Properties

Member	Standard, Strength
Wide Flange Shapes	ASTM A992, F_y = 50 ksi
	ASTM A913, F_y = 50 ksi
Tube Sections	ASTM A500, Grade B, F_y = 46 ksi
Pipe Sections	ASTM A53, Type E or S, Grade B, F_y = 35 ksi
Angle and Channel Sections	ASTM A36, F_y = 36 ksi
Miscellaneous Plates and Connection Material	ASTM A572, F_y = 50 ksi
	ASTM A588, F_y = 50 ksi
High-Strength Bolts	
7/8" diameter and smaller	ASTM A325
1" diameter and larger	ASTM A490

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Table 4. Concrete Properties

Member	Strength*
Slab on Ground, Sidewalks, Curbs, Mechanical Pads	f'_c = 4.0 ksi
Basement Walls, Footings	f'_c = 5.0 ksi
Mat Foundation	f'_c = 6.0 ksi at 56 days
Composite Floor Slabs	f'_c = 4.0 ksi
Shear Walls	f'_c = 6.0 and 8.0 ksi at 56 days

*28-day strength, unless noted otherwise.

Table 5. Reinforcement and Post-Tensioning Properties

Standard	Strength
ASTM A615, Grade 60	f_y =60 ksi
1/2" diameter, 7-wire strand	f_{pu} = 270 ksi

Structural Technical Memo
Los Angeles River – Rio Hondo Confluence Area Project