Highway Interchange Constructed With Slurry Walls

E. Itzig Heine
Frank A. McDonough
Surinder Singh

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E. Itzig Heine, P.E.
President, Alpha Corporation, McLean, Virginia

Frank A. McDonough, P.E.
Executive Vice President, Alpha Corporation, McLean, Virginia

Surinder Singh, P.E.
Vice President, Alpha Corporation, McLean, Virginia

SYNOPSIS

The use of slurry walls to support 40 feet high embankments and bridge structures at Techwood-Spring Connector, part of Williams Street Interchange in Atlanta, is described. Advantages of using slurry walls in lieu of conventional walls are discussed.

PROJECT DESCRIPTION

As a part of upgrading and expanding the Atlanta Interstate System, the Williams Street Interchange required grade separation via numerous ramps and bridges. The Techwood-Spring Connector, a depressed section of the highway, is a part of this Interchange that runs beneath Williams Street and Alexander Street. A partial layout of the complicated ramp system is shown in Figure 1. The grade difference between pavement at Techwood Spring Connector and bridges above varied to a maximum of 40 feet, and required massive retaining walls for a length of 720 feet on both sides of the street. In addition, massive abutments were provided to support bridge decks at Williams Street and Alexander Street.

The bid package included conventional reinforced concrete walls supported on continuous footings as the basic design for the embankment walls but gave an option of slurry walls designed and installed by the Contractor. Because of the excessive height of the embankment, the proposed footings for the conventional retaining walls were wide and extended far behind under the backfill. The construction of these walls would have required either temporary support system such as sheeting, or massive open cut excavation and backfill. In addition, this option necessitated highway detours during construction in this busy section of downtown Atlanta. Both open cut excavation and flexible sheeting system could result in lowering of the water table in the general vicinity of the excavation area, and possible settlement of the adjacent structures; the design of the excavation support system would have to be rigid enough to prevent this. At the time of the bid a cost comparison was made between the conventional retaining walls and slurry walls. Slurry walls were found to be preferable for the following reasons:

1. Since slurry walls acted both as temporary and permanent support of the embankments, the need for an auxiliary excavation support system was eliminated.

2. Major savings were realized by minimizing the amount of excavation and backfill.

3. Since slurry walls are rigid and practically impervious, movements associated with ground water lowering and soil disturbance due to excavation were reduced to negligible amounts.

4. Construction of the slurry walls allowed traffic to be maintained on all adjacent streets without major detours. Since bulk excavation between the walls was carried out after the construction of the bridge decks was completed, no costly temporary structural framing and traffic decking were required.

5. It was possible to construct poured-in-place reinforced concrete bridge girders as well as tee struts on temporary subgrade by excavating in stages. There was no need for costly shoring and scaffolding.

6. Construction easement was much less for the slurry wall option than for the conventional wall and footing.

7. Savings in time were realized by starting construction of bridge work as soon as the slurry walls were poured.

The slurry walls on either side of the proposed Techwood-Spring Connector, as shown in Figure 1, were 720 feet long each and varied in height from 30 to 60 feet including embedment below subgrade. The maximum panel length was limited to 16 feet. The nominal thickness of walls was set at 2'-8". Except near each end of the walls, a single level of struts was used to provide lateral support to the walls, at a clear height of 16'-9" above grade. Continuous poured-in-place reinforced concrete walers were keyed into the slurry wall panels to distribute earth
FIG. 1 - LOCATION PLAN

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loadings to the struts. Continuous cap beams, poured over the top of the slurry walls, were used to distribute the loads and support the upper portion of the retaining walls and parapet walls. The two skewed bridge structures at Williams and Alexander Streets were directly supported on the cap beams.

SUBSOIL CONDITIONS

A number of borings were taken along the proposed alignment of the Connector to determine the soil characteristics for design and construction of the slurry walls. The soils in general were micaceous sandy silt deposits with occasional lenses of clay, increasing in density, rather erratically with depth until rock surface was reached. A general profile of the soils and rock is shown in Figure 2. The predominant soil stratum above the proposed subgrade consisted of medium dense micaceous sandy silt deposits, varying in depth from 10 to 50 feet; the penetration resistance values were in the range of 11 and 30 blows per foot. Underlying this stratum were dense micaceous silty sand deposits with N-values ranging from 30 to 60. This stratum was rather erratic in nature as the thickness varied from negligible to approximately 20 feet. This stratum in general extended below the final pavement level. The next significant stratum overlying the rock surface was very dense micaceous sandy silt layer ranging in thickness from 5 to 30 feet. The standard penetration values were very high in this stratum, generally exceeding 60 blows. It was proposed to set the bottom of the slurry walls in this very dense sandy silt material except where the rock surface was higher. Where rock was encountered, bottom of walls was extended 1.0 foot minimum into sound rock.

The following soil parameters were used in the analysis and design of slurry walls:

A. Soils with \( N < 20 \):
   - Unit Weight = 110 pcf
   - Angle of Internal Friction = 14°
   - Cohesion = 500 psf
   - At-Rest Pressure Coefficient = 0.6

B. Soils with \( 20 < N < 40 \):
   - Unit weight = 110 pcf
   - Angle of Internal Friction = 25°
   - Cohesion = 300 psf
   - At-Rest Pressure Coefficient = 0.5

C. Soils with \( 40 < N < 60 \):
   - Unit weight = 115 pcf
   - Angle of Internal Friction = 34°
   - Cohesion = 200 psf
   - At-Rest Pressure Coefficient = 0.5

D. Soils with \( N > 60 \):
   - Unit Weight = 120 pcf
   - Angle of Internal Friction = 40°
   - Cohesion = 200 psf
   - At-Rest Pressure Coefficient = 0.45

The allowable bearing pressure at the bottom of the wall in the very dense sandy silt layer was assumed to be 7 kips per square foot. The water table was generally 20 feet above the subgrade.

SLURRY WALL DESIGN AND CONSTRUCTION

Figure 3 shows a general profile of the slurry wall with the bottom of wall as originally proposed in the Bid Documents, as originally designed and as actually installed. The limits of subgrade, location of strut level and parapet walls are also shown. The subgrade shown is the final pavement level. One level of struts was used to provide lateral stability where the height of the wall exceeded 14 feet; portions of the walls at each end were cantilevered above the subgrade. The as-proposed bottom of the wall was used for bid purposes whereas as-designed limit was governed by structural analysis and stability considerations.

Figure 4 shows a typical section through the wall and a general section at the bridge. A continuous cap beam was provided at the top of the wall to support bridge deck beams and distribute the vertical loads evenly on the walls. Bridge deck beams were anchored to the cap beams at one end only with an expansion joint on the other end so that no axial load was transmitted to the deck by the slurry walls. A continuous 3 inches deep blockout, with steel dowels, was provided at the strut level to anchor continuous waller to the walls. Tee struts, spaced 16 feet on centers maximum and spanning 77 feet between the walls, were provided at a minimum clear height of 16'-9" above grade. The wall was finally faced with precast concrete panels to give it a finish and texture compatible with the formed concrete surfaces of the adjacent walls at the Interchange.

Computer analysis was employed to check wall stability at different stages of construction. A general stability analysis of the wall at different locations was performed for short term loading condition and for long term loading using at-rest earth pressure coefficients based on the soil parameters described earlier. Active earth pressure coefficient was used for short term loading analysis. Depending upon the height of the wall above subgrade, portions of the wall were analyzed as a free cantilever above subgrade or as a member with one level of support at the strut level. Since there was no physical strut provided at the subgrade level, wall analysis was carried out by considering only passive pressure from below the subgrade. For design considerations effective subgrade level was assumed to be 4 feet below the final pavement level to allow for possible future excavation for installation of utilities and other structures below the pavement. Triangular pressure diagrams were used for both active and passive pressures acting on the wall. The embedment of wall below the subgrade was determined by em-

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FIG. 2 - SOIL PROFILES
FIG. 4 - WALL SECTIONS

DECK

CONC. GIRDER

SUBGRADE FOR POURING OF BRIDGE DECK

CAP BEAM

PARAPET & CAP BEAM

DOWELS

WALER

CONC. STRUT

WALER

PRECAST FASCIA PANEL

SLURRY WALL

PAVEMENT

10'-6" MIN.

2'-6"

EMBEDMENT

BOTTOM OF WALL

TYP. SECTION

SECTION AT BRIDGE

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ploying a minimum factor of safety of 2.0 applied to passive pressure. The general stability analysis also included investigation of deep seated failure for long term loading condition. Computed deflections of wall sections were limited to 1/2 inch under maximum loading condition. The strength design of wall sections was done in accordance with the Ultimate Strength Design provisions of the ACI-318 Building Code (1977).

The installation of the slurry wall panels took place at a fairly rapid rate though difficulty was encountered for trenching in dense stratum at lower depths. In the upper loose stratum some loss of ground took place that resulted in bulges that had to be chiselled away to accommodate precast panels. The bulges were attributed partly to the need to keep the trenches open for a long period to permit inspection and verification of the rock-like material. Some of the bulges were caused by the removal of small boulders and breaking up of rock.

After the walls were installed continuous cap-beams were poured on top of the walls. General excavation was done in stages to facilitate construction of bridge deck and poured-in-place concrete struts without using shoring and scaffolding. As shown in Figure 4, first stage excavation was done to underside of bridge deck beams. After the deck was poured and cured, excavation was carried to the bottom of the struts. Continuous walers and struts were then formed and poured before further excavation was advanced to the subgrade level. Finally, drainage structures were installed, curbs to support pre-cast panels poured, and pavement constructed to the final grade. Figures 5 and 6 show views of completed slurry walls, bridge structure, tee struts and walers prior to installation of pre-cast panels.

SUMMARY AND CONCLUSIONS

A case study of the use of slurry walls as embankment walls and bridge abutments at Techwood-Spring Connector, an integral part of Williams Street interchange in downtown Atlanta, has been presented. The use of slurry walls for this depressed section of the Interstate Highway was specified as an alternate to the conventional concrete retaining walls in the Bid Documents. However, after evaluation of all factors involved, the use of slurry walls was found to be more cost-effective. By pouring the walls ahead of bulk excavation for the depressed roadway, it was possible to construct the bridge deck structures and struts used for lateral stability of the walls, from subgrade; thus eliminating the need for shoring and scaffolding. The need for a temporary excavation support system for the construction of conventional embankment walls was eliminated.

Depending upon the height of embankment to be supported, the walls were either cantilevered above subgrade or provided with a single level of poured-in-place concrete struts spanning 77 feet across between continuous concrete wa-

Slurry wall excavation proceeded with ease in the upper soil strata but considerable difficulty was encountered in establishing the trench bottom in the lower very dense material. Although the design was based on adequate embedment in dense material, controversy arose because of the lack of a practical means of verifying the foundation elevation with as-proposed wall bottom given in the Bid Documents. The bid item was based on an assumed bottom of wall profile in rock-like material, and inspectors often insisted on trenching deeper even though the design toe had been achieved at higher elevation in relatively easily-excavated but suitable material. This resulted in much chiselling of the deeper rock-like material that necessitated leaving the trench open for long periods of time. The trench sides were thereby subjected to more exposure to unravelling caused by bucket removal and this resulted in some over-excavation and concrete bulges.

The recorded wall movements were within the Contract specified limits and were in agreement with the computed values.

The slurry walls in this case provided a unique solution for the support of 40 feet high embankments and two skewed bridges along Techwood-Spring Connector at Williams Street Interchange in Atlanta, Georgia. Based on the experience gained on this project, it is recommended that the slurry trench method be confined to soil excavation where possible as this is far more timely and cost-effective. In the case where a deeper toe in the rock-like material is specified, it is suggested that inspection criteria be developed based on the use of slurry trench tools, i.e. bucket and chisels, for an accepted means of verifying the suitability of the material. In this way excessive chiselling could be avoided in most cases and the trench would not have to be left open for a long period of time.

REFERENCES

AMERICAN CONCRETE INSTITUTE (1977), Building Code Requirements for Reinforced Concrete, ACI 318-77, Detroit, Michigan.

FIG. 5 - COMPLETED WALL

FIG. 6 - STRUTS AND BRIDGE GIRDERS