Tips for Slurry Wall Structural Design

Construction techniques and site-specific subsurface conditions affect slurry wall performance more than structural details or code interpretations.

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Reinforced concrete diaphragm walls, commonly known as slurry walls, have been used in Europe since the 1950s and have gained increased popularity more recently in the United States, particularly in major urban projects, including Boston and Chicago.

In the Boston area alone, involvement in the design and construction of slurry walls has been extensive. Besides their use on the Central Artery/Third Harbor Tunnel project and many transit subway projects, several underground structures have been constructed using slurry walls, including segments of the Red Line tunnel extension from Harvard to Alewife stations, Rowes Wharf, 75 State Street, 150 Summer Street, Post Office Square garage, North Station garage and Beth Israel garage (under construction). In all of these projects, the slurry wall acted as the temporary as well as the permanent structural wall. The walls were designed to resist lateral earth/water pressures, earthquake forces and, at the same time, had to be load bearing by supporting column loads from air-rights developments and the below-ground floor reactions. Slurry walls are capable of behaving structurally as temporary and permanent earth/water retention systems, as well as behaving as load-bearing elements (LBEs) with the ability to provide a finished concrete surface. These multiple characteristics of slurry walls have resulted in many advantages over conventional foundation design, particularly when used in congested built-up areas.

Simply put, the construction of a slurry wall panel consists of:

- Excavating a trench approximately two to three feet wide by six to 25 feet long with a grab bucket down to till or rock;
- Filling the trench with a liquid slurry bentonite (primarily clay mineral) as the excavation proceeds;
- Lifting a reinforcing cage (that is assembled at the site) with a crane and placing it in the completed trench under slurry; and,
- Lowering tremie pipes to pump concrete from the bottom up, displacing the slurry liquid as concreting takes place.

Panels can be cast either alternately or successively, and the joints between adjacent panels are intended to be watertight.

Since a slurry wall is typically buried in soil for at least half its face and for the entire service life of the structure (a perfect soil-structure interaction problem), its design is greatly dependent on the method of analysis used and requires close communication between structural and geotechnical engineers. Therefore, a brief description of these analysis methods used in design, which have been well presented by Kerr and Tamaro,[1,2] is warranted.

Methods of Analysis

There are four primary methods of analysis for slurry walls:

- Equivalent beam on rigid support (rigid method);
- Beam on elastic foundation (Winkler method);
- Finite element method (FEM); and,
- Limit analysis.

Equivalent Beam on Rigid Support (Rigid Method). In this method, the point of zero pressure is computed below the excavation cut (passive pressure equals active pressure) and acts as a fictitious support. As the excavation proceeds, the wall spans as an elastic beam between rigid supports (provided by bracing members or floor slabs) with the lowest support at the point of zero pressure below the subgrade. While this method may be conservative, it does provide an easy structural solution. However, the computed wall deflections from the elastic analysis do not correlate to predicted ground movements

Beam on Elastic Foundation (Winkler Method). Winkler analysis models the passive side of the wall as a series of "springs" based on the modulus of the subgrade reaction of the soil. Data on the elastic properties of the soil are required in order to obtain the soil modulus. The analysis is conducted using general structural computer software in order to determine moments and shears. This method seems to yield smaller forces than the rigid method and also gives a better indication of wall movement.

Finite Element Method (FEM). This method requires somewhat advanced computer software and modeling procedures. It also requires a more thorough knowledge of the soil properties that are modeled in the program and with the construction sequences simulated in the finite element analysis. Its greatest benefit is better prediction of soil movement. Its use for structural design is limited and reliance on other methods is necessary.

Limit Analysis (Plastic). Limit, or plastic, analysis is not typical for foundations or below grade structures. The advantages of this method need to be explored more, but without further study at this stage its economical advantages, if any, must be carefully analyzed.

While the rigid method is well known and has been widely used in the past, the Winkler method has been rapidly gaining popularity. The rigid method offers a quick and easy method of analysis and is certainly worthwhile to consider as a first choice or as a preliminary analysis tool due to its simplicity. The Winkler method offers a more attractive solution in that it yields a more realistic representation of wall behavior and allows complete modeling of the wall based on soil springs as well as brace or floor springs (K-values based on stiffness and volume change), and could be used next as a check for design. Selection of the analysis method should be an engineering judgment that should be exercised on a case-by-case basis, taking into account the degree of conservatism desired.

Certainly, FEM is a powerful analysis tool and offers many advantages during construction/excavation as a means of monitoring wall movements and comparing them with predicted ones. It is recommended that FEM be used as a supplement in instances where adjacent construction may be sensitive to wall movements and better prediction of these movements is required.

Foundation work and underground structures are meant to have considerably longer life than superstructures and, as such, should be analyzed and designed more conservatively since their maintenance, accessibility and repair is much more limited. Given the various unknowns inherent in soil properties, as well as actual versus assumed loads and other such factors, a cautious approach to elastic analysis should be exercised. Regardless of the method that is employed, it is essential that close coordination and communication take place early in the design process between geotechnical and structural engineers, and that a consensus and understanding be reached. This process must be followed by coordination meetings with the contractor in order to avoid problems during construction. However, the contractor, excavator, inspector, rebar supplier, ironworker and slurry wall contractor should not all be expected to coordinate with each other on a specific shop drawing item. Therefore, the engineer should be the mediating influence between all parties involved.

Slurry Wall Penetration

The depth of wall penetration below subgrade or dredge level is a stability problem. In most cases, it is controlled when the excavation is at its final stage and just prior to installing the lowest slab or bracing member. It is assumed, for economical reasons or soil conditions, that the penetration depth below this level is insufficient to produce restraint at the bottom of the wall (analogy to the free earth support method). Since the wall is free to rotate about its lower end, a factor of safety with respect to passive resistance must be used in the design. A value of 1.50 is recommended as a minimum.

The required penetration, d, is determined by equating to zero the sum of the moments on the active and passive sides about the brace point or floor level above (see Figure 1). This condition yields a cubic equation in which d is easily solved by substituting trial values. The embedment is often arbitrarily increased by 20 percent (an additional two feet for every 10 feet of embedment) to guard against excess excavation or the presence of pockets of weak soil.

Some engineers logically may want to take advantage of the additional bending resistance provided by the wall at the brace elevation and/or the moment capacity of the brace point at the connection to the wall. This approach is certainly an engineering judgment that should be used on a case-by-case basis, depending on the degree of protection provided by the other systems utilized. It is a function of several factors including the overall degree of conservatism in selecting soil parameters, method of analysis, accuracy of loads and groundwater pressure, existing adjacent foundations, etc. The same rationale also should be used in determining the magnitude and applicability of the safety factors that must be incorporated in the design.

Another type of analysis that may be of interest is to consider the effect of base friction on the wall penetration, taking into account the coefficient of friction between the concrete and the rock or bearing material, and then making a determination on whether to rely on it or not.

State-of-the-Art Technology

A few of the projects located in the Boston area, examined from a design criteria standpoint, illustrate some of the current methodology used in the design of slurry walls.

North Station Underground Garage. This six-level underground garage is located at the site of the new Shawmut Center sports arena behind the existing Boston Garden building. Construction is nearly complete, and consists of three-foot thick reinforced concrete slurry walls, 60 to 80 feet deep, embedded in bedrock and with wall panels varying in length from 8.5 to 25 feet.

Due to the large footprint of the garage, the scheduled arena construction above it and the extremely tight physical constraints of the site that is surrounded by the existing Boston Garden building, the Orange Line subway tunnel, the Federal Administration Building, and a Central Artery ramp structure, the top-down method of construction was chosen. Under this method, the slurry walls were built first all around the site, acting as a water cut-off as well as the permanent wall of the structure. Interior steel columns were then installed from grade using a slurry trench technique. The base was filled with tremie concrete, forming supporting caissons or LBEs. The ground floor level was then built around the columns that supported it, and the completed slab braced the slurry wall while excavation was carried out in a mining-like operation, thus eliminating the need for conventional internal bracing or tiebacks. This step was repeated for the lower levels.

Since the slurry wall served as both the temporary and permanent excavation support element for the garage, it was analyzed for all stages of construction, and its design and detailing were comprehensively included in the contract documents.

During the construction process, active lateral pressures were assumed and a triangular pressure distribution was used. The passive pressure was computed at various excavation levels and was developed based on the point of

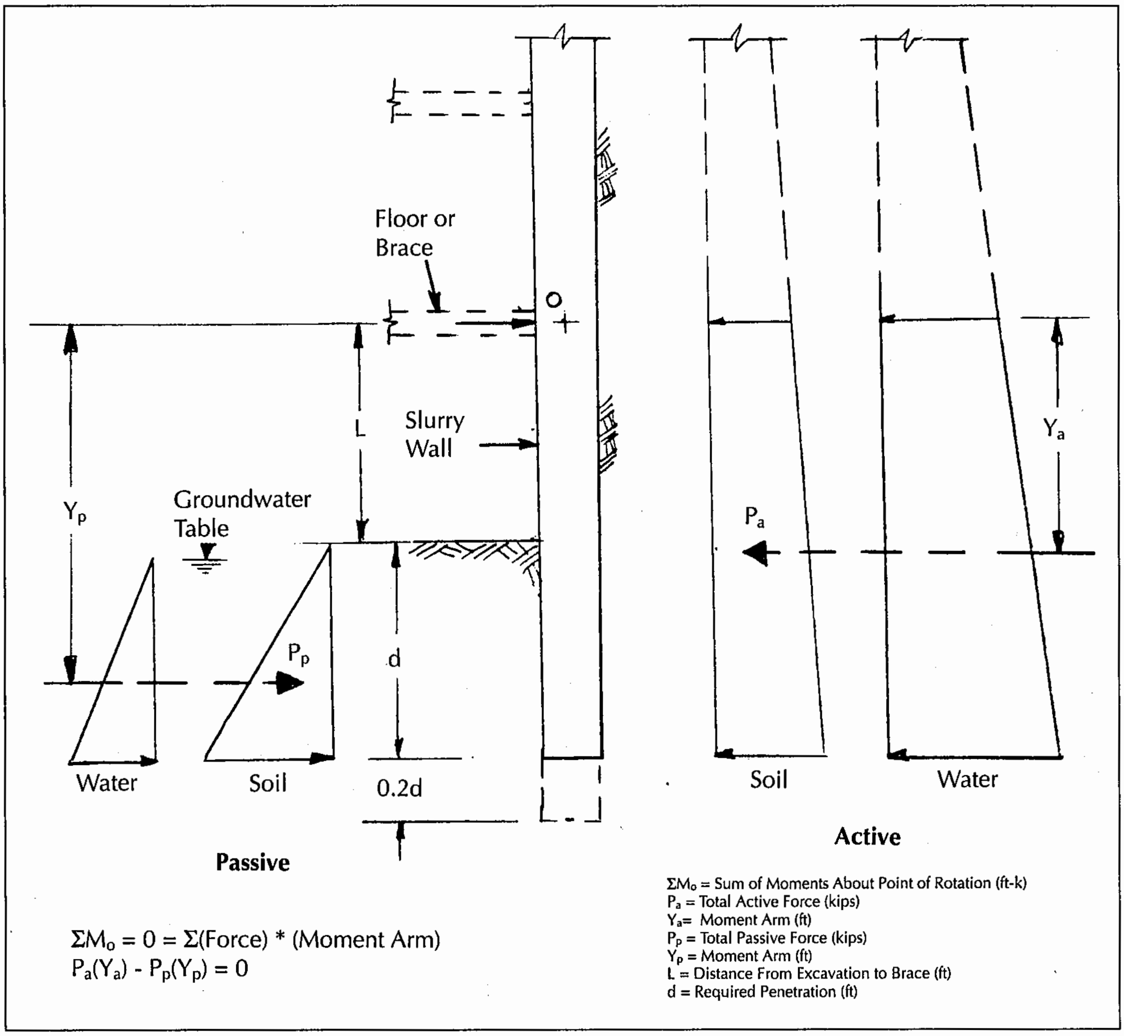

FIGURE 1. Slurry wall penetration.

"zero pressure" needed to compute the reaction below the excavation level (rigid method). This analysis was later checked using the Winkler method by inputting the soil on the passive side as a series of spring constants based on the soil modulus of the subgrade reaction. A finite element analysis was also performed and the wall was analyzed assuming the predicted slurry wall deformations. The calculated bending moments obtained using FEM were lower than the ones from the other analyses.

As construction neared completion, prior to casting the lowest base slab (six levels down), active lateral pressures were also assumed but based on a rectangular distribution. This analysis was used to determine the slurry wall penetration and stability that was obtained by equating to zero the active and passive moments about the level above.

At rest, lateral pressures (triangular distribution) were used for the design of the slurry wall in order to evaluate long-term (permanent) loading.

In all of the above stages, the slurry wall was designed for soil, groundwater and surcharge pressures, and in the long-term stage for seismic pressures. A factor of safety of 1.50 was applied to passive soil pressures for all stages. To obtain a comprehensive bending moment envelope along the height of the wall, the support reactions provided by the floor slabs at each stage were considered to be rigid supports (zero deformation) as well as flexible springs, taking into account the elastic shortening of the slab as well as volume change deformations resulting from creep, shrinkage and temperature variations.

Computations were performed using the ultimate strength design method (the American Concrete Institute's Building Code Requirements for Reinforced Concrete [ACI 318]).[3] The basis for using the 1.50 safety factor that was applied to the passive pressure in all stages was to minimize wall movement next to adjacent structures and limit cracking (since the wall is the temporary and permanent support, and the stresses in the wall were greater during the excavation stages than in the long-term stage).

Post Office Square Underground Garage. The seven-level Post Office Square garage in the heart of downtown Boston utilized three-foot thick slurry walls approximately 90 feet deep, and the top-down construction method. Slurry wall analysis and design were somewhat similar to the North Station project. The contract drawings took into account the slurry wall reinforcement and detailing for permanent loading. The reinforcement was checked and modified as required for staged excavation during the construction phase. The project was successfully completed in 1990, and has won several design awards.

Rowes Wharf. Located along the Boston Harbor, this wharf structure employed 2.5-foot thick slurry walls on the waterside and landside that were about 70 feet deep. The up-down construction method was used for this project.

In all of the above projects, an extensive construction monitoring program was instituted that included observation wells, inclinometers and settlement monitoring. This program was necessary in order to monitor potential wall movements as construction excavation progressed, and to check against calculated wall deflections and structural behavior.

Theory Versus Practice

What does all this mean to structural design? Engineers fall back to the ACI 318 concrete building code and design a slurry wall for flexure and shear as if the requirements were no different from those for walls cast above ground. The major difference with slurry wall construction is that it is a deep strip foundation built somewhat in the blind without the convenience inherent with exposed construction (in terms of placing and curing concrete, the control of form dimensions and the placing of reinforcement). For these reasons, a more conservative approach to design and detailing is warranted, including making allowances for anticipated construction tolerances and keeping in mind that the finished product does not always turn out as portrayed by the neat straight lines on the contract drawings (see Figure 2).

While flexural and shear design have a great affect on the eventual performance and appearance of the slurry wall, crack control and wall/soil deflection/displacement must also be considered in a typical design.

Flexural Design

As an illustration of recommended design procedures, consider the example of a three-foot thick wall by one-foot wide strip. Typically, a continuous slurry wall is reinforced in two directions and spans vertically between supports (which consist of bracing members or concrete slabs). Therefore, only minimum horizontal reinforcement is required in accordance with ACI 318-89. Horizontal reinforcement, in this instance, can be calculated thus:

$$
\begin{aligned}
(A_{sh})_{min} &= 0.0025 \times b \times h \\
&= 0.0025 \times 12 \times 36 = 1.08\ in^2/ft \div 2\ each\ face \\
&= 0.54\ in^2/ft
\end{aligned}
$$

where:

b = Width of compression face of wall (in)

h = Thickness of wall (in)

A_{sh} = Area of horizontal steel (in^2)

In this case, use a #7 horizontal bar with 12-inch spacing on each face (0.60 in^2/ft).

With the soldier-pile-tremie-concrete (SPTC) slurry wall system that is being used on parts of the Central Artery/Third Harbor Tunnel project, reinforcement may be omitted on the provision that the flexural, shear, compression and bearing stresses are less than the concrete cracking strength and the permissible stresses allowed by ACI 318.1 (Building Code Requirements for Structural Plain Concrete). In this system, the wall spans horizontally a rela-

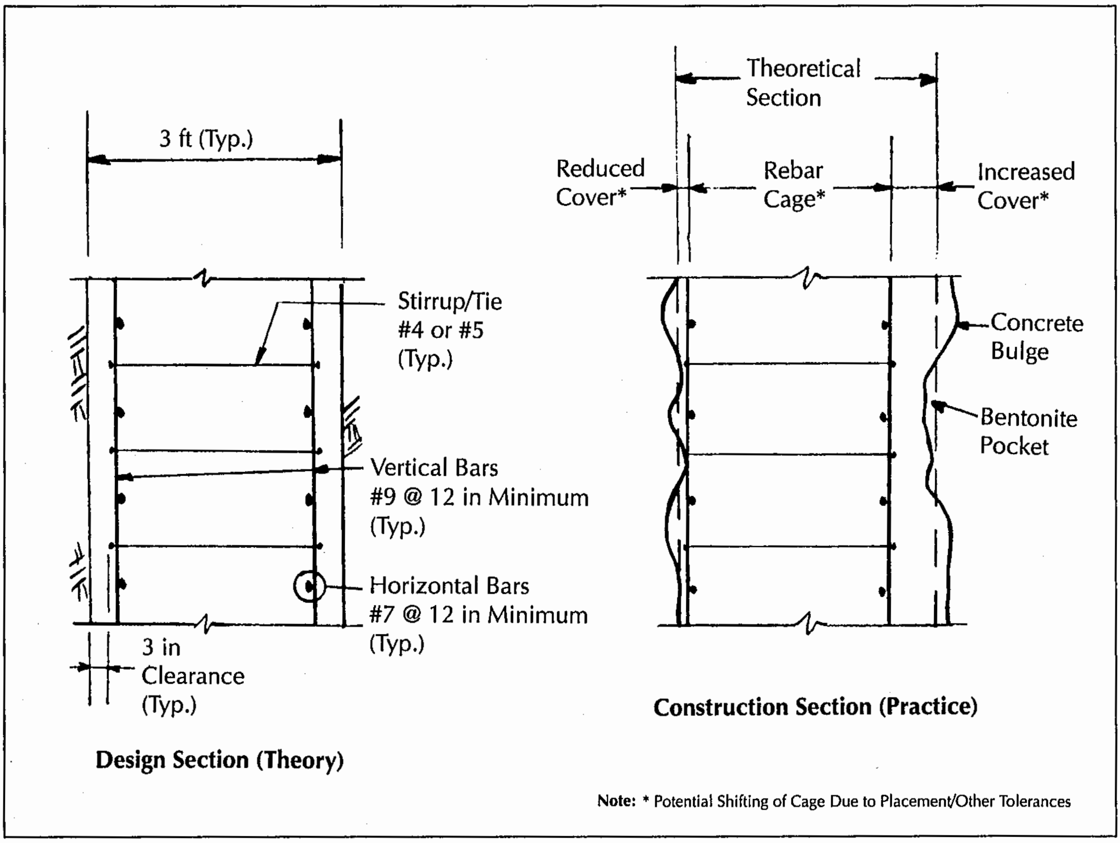

FIGURE 2. Reinforced concrete slurry wall section.

tively short distance (six feet) with a span/depth ratio of less than 2. Thus, the wall behaves as a deep beam member, supported by vertical steel sections that are capable of providing continuous vertical support to resist the overall lateral pressures.

Vertical Reinforcement. As a general rule, use larger bars at larger spacing for ease of concreting and a stiffer cage. Do not use less than six-inch spacing, bundle if necessary and do not splice in congested areas. Consider using #9 and #11 bars at six- and 12-inch spacing and compute the ultimate moment capacity for the various bar sizes and spacing from ACI Commentary R9.1.1:

$$\phi M_n = 0.9\, A_s f_y\, (d - a/2)$$
$$a = A_s f_y / (0.85 f'_c b)$$

where:

ϕM_n = Design moment capacity (ft-k)
A_s = Area of tension reinforcement (in^2)
f_y = Specified yield strength of reinforcement (psi)
d = Effective depth (in)
a = Depth of equivalent rectangular stress block (in)
f'_c = Specified concrete compressive strength (psi)

Concrete strength is typically 4,000 pounds per square inch (psi). Concrete strength is greatly affected by any change in the water-cement ratio. This change could occur if the bentonite slurry is intermixed with the concrete, which would increase that ratio and theoretically reduce the concrete strength. However, a good slurry mix should prevent that.

On recent projects, there has been no reason to conduct test cores on slurry walls. Xanthakos indicated that core tests obtained from walls several weeks after concrete placement showed strengths higher by as much as 1,000 psi than the design strength.[4] It would be interesting to obtain test cores several months — or longer — after excavation has been completed and the slurry wall surface has been exposed.

For concrete strength selection, it may be good practice to specify 4,500 psi or to design for 3,500 psi. This choice is a judgment call. It does not affect the amount of flexural reinforcement, but it may affect shear design and deflection. For a small incremental cost (if there is any at all), it is worthwhile doing, depending on the type of structure involved and until further tests indicate otherwise.

Grade 60 steel is the commonly used steel in reinforced concrete practice. Therefore, the yield strength of reinforcement, f_y, is usually 60,000 psi.

The effective depth of the slurry wall requires some attention since it greatly affects the amount of reinforcing required. ACI 318-89, Section 7.7.1(a), prescribes that "concrete cast against and permanently exposed to earth" have a minimum concrete cover of three inches needed for reinforcement. Furthermore, a tolerance of one inch in placing the steel is prescribed by ACI 117-70 (Tolerances), Section 2.2 (Reinforcement Placement), which is intended for formed walls.[5]

In addition, the out-of-plumb tolerance of the wall should be considered and typically should not exceed one inch from floor to floor. Finally, bulges and cavities occur typically as a result of slurry wall construction, resulting in the loss of cross section and, at times, exposed reinforcement.

Taking all the above factors into account, slurry wall effective depth can be computed as:

$$d = 36 - 3(cover) - 1(rebar\ tolerance) - 1\ (other\ tolerances) - 1/2(\#4\ stirrup) - d_b/2(rebar)$$
$$= 36 - 5.5 - d_b/2$$

For #9 bars with a bar diameter, d_b, equal to 1.13, the effective depth equals 29.9 inches. For #11 bars with d_b equal to 1.41, the effective depth equals 29.8 inches. Therefore, a value of 29.5 inches for the effective depth is recommended. Some may argue that this value is too conservative and it very well may be. However, its adoption is an engineering judgment that should be used on a case-by-case basis and depends largely on the degree of protection required and the type of structure under study.

TABLE 1
Moment Capacities

Bar Size	Spacing (in)	A_s (in²)	a (in)	$\o M_n$ (ft-k)
#9	6	2.0	2.94	275
#11	6	3.12	4.59	417
#9	12	1.0	1.47	129
#11	12	1.56	2.29	199

The moment capacities can be computed as shown in Table 1.

One easy method of selecting the required reinforcing is to plot the maximum positive and negative moments at each stage of construction along the full height of the wall and selecting the appropriate reinforcement by comparing the applied moments and the moment capacities.

Minimum Vertical Reinforcement. This requirement may also be subject to debate. One can rightfully turn to ACI 318 Chapter 14 (on walls) and determine the minimum area of vertical reinforcement (in^2) from the following equation in Section 14.3.2:

$$(A_v)_{min} = 0.0015 \times 12 \times 36 = 0.65\ in^2 \div 2\ each\ face$$
$$= 0.33\ in^2/ft$$

In this case, the minimum vertical reinforcement would be #5 bars at 11-inch spacing or #6 bars at 16-inch spacing.

However, bearing in mind that the intent of ACI's requirements for walls is that they serve as primarily compression members, while a slurry wall is primarily a flexural member. Therefore, it is recommended to use ACI Section 10.5.2, which calls for providing reinforcement at least one-third greater than that *required by analysis*. What is required by analysis could be, as a minimum, the moment capacity of the wall as a plain concrete section, or cracking moment, determined from the modulus of rupture of concrete, f_r, as shown in Section 9.5.2.3:

$$M_{cr} = f_r\, I_g/y_t$$

where:
f_r = Modulus of rupture (psi) = $7.5\sqrt{f'c}$ = 474 psi

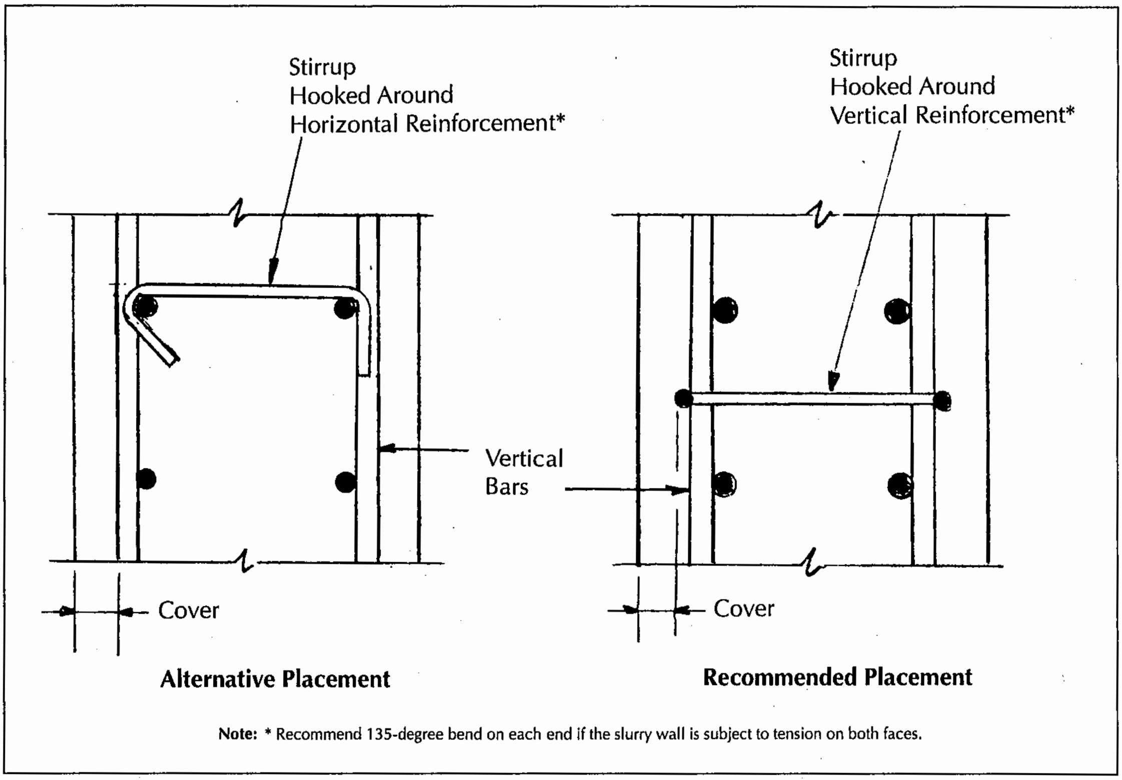

FIGURE 3. A section of the stirrup reinforcing detail.

I_g = Moment of inertia (in^4) = $bh^3/12$ = 46,656 in^4
y_t = Distance from neutral axis to extreme fiber in tension (in) = $h/2$ = 18 in

The wall's cracking moment is equal to 102.4 ft-k. The bonding resistance coefficient, K_u, is equal to (M_{cr}×12,000)/(bd^2) or (102.4× 12,000)/(12×29.5^2), which yields 118 psi. The ratio of tension reinforcement, ρ, is equal to A_s/bd, which yields a value of 0.0022 (from the ACI Design Handbook Table 2.2, Flexure). ρ_{min} is equal to 0.0022×(4/3), or 0.0029. The minimum area requiring tension reinforcement, $(A_s)_{min}$, is equal to 0.0029×12×29.5 or 1.03 in^2/ft, in which case use a #9 bar at 12-inch spacing (1.0 in^2/ft).

Therefore, if the applied moment is less or equal than 102.4 ft-k, provide minimum reinforcement as computed above along each face of the wall. It is also important to point out that in completing a project's detailed drawings, the total number of bars provided in the wall should be based on the overall panel length. This requirement is because of the large concrete cover to the vertical bars from the ends of the panel, which can be as much as nine inches and is dictated by the size of the end stops.

On the other hand, some may consider ACI Section 10.5.3 more appropriate for a slurry wall since it is essentially a one-way structural slab of uniform thickness. This assumption is true from the standpoint of the width to depth ratio. However, a slab is typically much thinner than a three-foot slurry wall section. Using Section 10.5.3, or $(A_s)_{min}$ = 0.0018×12×36 = 0.78 in^2, will result in a moment capacity of 101 ft-k, which is slightly smaller than the cracking strength of plain concrete (the 102.4 ft-k above). Therefore, it would not be effective or contribute to the wall's bending resistance. Moreover, a slab's weight is not normally subjected to a true permanent and uniform loading. That is the reason why the minimum reinforcement required for slabs is less than that for beams, since an overload as discussed in the ACI Commentary would be distributed laterally and a sudden failure would be less likely. A slurry wall, on the other hand, is permanently subjected to the uniform loading of soil and water (primary loads) and, hence, for design purposes it is recommended to use Section 10.5.2.

Shear Design

With regard to flexure, it is recommended to plot the shear force envelope, V_u, along the height of the wall and compare it with the shear capacities, $øV_n$. The ultimate shear capacity of the concrete is determined by ACI Section 11.3 as follows:

$$øV_c = (0.85 \times 2 \times \sqrt{4{,}000} \times 12 \times 29.5)/1{,}000 = 38 \text{ kips/ft}$$

If the shear force, V_u, is less than 38 kips/foot, then no shear reinforcement is required. If V_u is greater than 38 kips/foot, provide minimum stirrups to meet ACI Section 11.5.5.3. Use 18 inches for the horizontal spacing, b_w. For the recommended vertical spacing, S, first calculate the maximum vertical spacing, S_{max}, which is equal to $d/2$ (29.5/2 or 14.75 inches). Therefore, it is recommended to use 12-inch vertical spacing.

The minimum vertical reinforcement required is determined by:

$$(A_v)_{min} = 50 b_w S/f_y = (50 \times 18 \times 12)/60{,}000 = 0.18 \text{ in}^2\text{/ft}$$

In this case, use a #4 single leg stirrup at 18-inch horizontal and 12-inch vertical spacing. Shear capacity for this configuration is calculated as:

$$V_s = A_v f_y d/S = (0.2 \times 60 \times 29.5)/12 = 29 \text{ kips/ft}$$
$$øV_n = øV_c + øV_s = 38 + (0.85 \times 29) = 63 \text{ kips/ft}$$

If the shear force is less than or equal to 38 kips/ft, then no shear reinforcement is required. If the shear force is greater than 38 kips/ft and less than or equal to 63 kips/ft, use a #4 single leg stirrup at 18-inch horizontal and 12-inch vertical spacing. Shear capacities for different stirrup spacings and sizes can be developed in a similar manner.

In contrast to flexural design, the shear design in this case is quite straightforward. However, some disagreement lies in the detailing and placing of the stirrups. The issue is whether to hook the shear reinforcement around the vertical bars (recommended) or around the horizontal bars (see Figure 3). The recommendation is based on the interpretation of the following ACI code sections and additional comments, and is intended to promote further discussion.

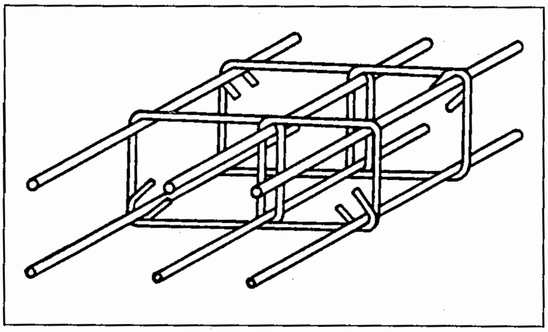

FIGURE 4. Slab stirrups.

ACI 318-89 requirements for anchorage (Sections 12.13 and R12.13: Development of Web Reinforcement) state that web reinforcement or stirrups shall be carried as close to the compression and tension surfaces of a member as cover requirements and the proximity of other reinforcement will permit because, near ultimate load, the flexural tension cracks penetrate deeply. Section 12.13.2 states, in part, that #5 bar stirrups (as well as smaller stirrups) should be anchored with a standard hook around *longitudinal reinforcement*. The vertical bars in the slurry wall are the flexural reinforcement and act as the longitudinal reinforcement. Therefore, the stirrups should be hooked around this reinforcement. It should also be noted that ACI anchorage requirements for stirrups have been developed primarily for slabs and beams in which the longitudinal (or flexural) reinforcement is enclosed by the stirrups. A wall subjected primarily to bending is, for design purposes, a vertical slab. The ACI code also addresses this requirement in the following sections:

> Section 7.11.2 states that "lateral reinforcement for flexural framing members subject to stress reversals or to torsion at supports shall consist of closed ties, closed stirrups, or spirals extending around the flexural reinforcement." This requirement is applicable for earthquake design where the dynamic or cyclic reversal of stress occurs. It is intended for building or superstructure

design in order to guard against excessive cracking caused by tension and to reverse shear developing on the bottom of beams at the supports. Even though stress reversal in slurry wall construction is more gradual (since it is typically associated with staged excavation), these reversals of stresses do cause reverse tension in the wall at different support points and between supports. Since the short-term loading (during excavation) seems to always control design and flexural tension cracks penetrate deeply near ultimate load, hooking the stirrups around the reinforcement is recommended.

Commentary on Section R11.12.3 (Special Provisions for Slabs and Footings) states that "research has shown that shear reinforcement consisting of bars or wires can be used in slabs provided that it is well anchored." The anchorage detail used in the tests is shown in Figure 4.

Section 14.3.6 requires that vertical reinforcement be enclosed by lateral ties if the vertical reinforcement area is greater than 0.01 times the gross concrete area, and where vertical reinforcement is required as compression reinforcement. This situation is particularly applicable if the slurry wall is supporting large column loads.

Conclusion

In summary, flexural and shear design for slurry walls must be approached not only from structural integrity and code requirement viewpoints, but must also take into account — during the design and detailing of the contract documents — the construction practices in the area and the increased tolerances associated with this type of construction. The recommendations presented here are intended to facilitate structural design, bring awareness to the differences between design and construction, and provide a more durable concrete wall that will exhibit fewer cracks and leaks, and help reduce any long-term maintenance that may be required. The ultimate performance of the slurry wall is more significantly affected by construction techniques and site-specific subsurface conditions than it is by structural details and code interpretations.

NOTE — *The author's extensive involvement in the North Station project rendered that description in the section on State-of-the-Art Technology to be more elaborate than the other noteworthy projects.*

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REFERENCES

1. Kerr, C.W., & Tamaro, J.G., "Diaphragm Walls—Update on Design and Performance," in *Design and Performance of Earth Retaining Structures,* Conference Proceedings of Geotechnical Engineering Division ASCE, Ithaca, June 18-21, 1990.

2. Tamaro, J.G., "Slurry Wall Design and Construction," in *Design and Performance of Earth Retaining Structures,* Conference Proceedings of Geotechnical Engineering Division ASCE, Ithaca, June 18-21, 1990.

3. American Concrete Institute Committee 318, "Building Code Requirements for Reinforced Concrete," ACI 318-89 (Revised 1992) and "Commentary-ACI 318R-89" (Revised 1992).

4. Xanthakos, P.P., *Slurry Walls,* McGraw-Hill, 1979.

5. American Concrete Institute Committee 117, "Standard Specifications for Tolerances for Concrete Construction and Materials," ACI 117-90, 1990.