# Design Process of Deep Soil Mixed Walls for Excavation Support

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**ABSTRACT**: *The use of deep soil mixing (DSM) technology for excavation support is growing in use across the world. In recent years, a number of projects in the United States have incorporated DSM for excavation walls. As deep soil mixing becomes a more economical alternative to traditional excavation support systems, determining which methods of design are most appropriate will become an important issue. Currently, standardized guidelines for design of DSM walls are not available. Case histories such as the Islais Creek Transport/Storage Project in San Francisco California can be used to illustrate the design process and steps that are unique to DSM excavation support. The focus of this paper is to present a case history to illustrate a design process for DSM walls.*

**KEYWORDS:** deep soil mixing, excavations, beam-column method, finite element method

**SITE LOCATION:** IJGCH-database.kmz (requires Google Earth)

## INTRODUCTION

Excavation support using deep soil mixing technology evolved from the early 1970's Japanese practice, in which single soil-cement columns were created to support excavations and act as cutoff walls. Deep soil mixing (DSM) for excavation support involves constructing a support wall by mixing in situ soils with a stabilizing agent. Some of the other common names used to describe this method are Cement Deep Soil Mixing (CDSM), Deep Mixing Method (DMM), or Soil Mix Wall (SMW) method (Porbaha, 1998). The mixed-in-place excavation support walls are well suited for urban areas with high groundwater levels since the placement of DSM columns causes little disturbance to the surroundings and generates low vibration and noise pollution. Numerous projects have incorporated deep mixing for excavation support and cutoff walls. One of the first major applications of DSM for excavation support in the United States was the Wet Weather Storage Basin for the East Bay Municipal Utility District (EDMUD) project in Oakland, California constructed in 1990 (Taki and Yang, 1991). One of the largest projects in the United States involving DSM technology is the Boston Central Artery and Tunnel (CA/T) project (O'Rourke and O'Donnell, 1997a and 1997b; O'Rourke et al., 1998, O'Rourke and McGinn, 2004).

The implementation of DSM technology into American excavation support design and construction practices is growing due to the various advantages it offers over traditional support systems; however, standardized design guidelines are not currently available. Although the design process for DSM walls is similar to traditional walls, there are steps that are unique when considering the design of the soil-cement. This paper presents a design flowchart and describes the steps recommended for design of DSM walls for excavation support. Aspects of simplified and computer-aided methods specific to DSM walls are also discussed. The Islais Creek Transport/Storage Project in San Francisco California is used to illustrate the design process for DSM walls.

## CONSTRUCTION OF DSM WALLS

Although there are a variety of DSM techniques, the most common method of deep soil mixing for application in

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excavations involves overlapping soil-cement columns that are either installed using a multi-auger rotary shaft or a drilling tool (Bruce et al., 1998). The construction requires specialized equipment for DSM and therefore contractors with specialized knowledge are recommended. The construction of DSM walls is typically faster than other traditional methods (structural diaphragm/slurry walls, sheet pile walls, soldier pile and lagging walls, secant/tangent pile wall and micro–pile walls) and generates fewer spoils than slurry (diaphragm) wall construction. Porbaha (1998) provides a comprehensive description of DSM construction and applications.

A stabilizing agent is mixed with the soil by multiple mixing blades to form the soil-cement wall. A variety of stabilizing agents can be used such as lime, fly ash, and cement; however, the most common is a slurry mixture of cement, water, and sometimes bentonite. The resulting deep mixed soil column is often referred to as soil-cement. Although the DSM specialty contractor often determines the mix design, it is important for the design engineer to understand factors contributing to the strength and permeability of the DSM column. The improved engineering properties of the stabilized soil are governed by a number of factors including soil type, slurry properties, mixing procedures and curing conditions (Yang, 2003). The unconfined compression strength of the soil-cement for an excavation support cutoff wall is usually greater than 700 kPa (100 psi) and hydraulic conductivity usually ranges from $10^{-5}$ to $10^{-6}$ cm/s ($4 \times 10^{-6}$ to $4 \times 10^{-7}$ in/s) (Taki and Yang, 1991).

Typically, steel reinforcement (usually wide flange H-beams or sheet piles) is placed in the soil-cement columns to resist bending. The typical arrangement of DSM excavation support walls is illustrated in Fig. 1. Steel reinforcement is installed in every other DSM column after mixing and the accessible face of each column is trimmed off once the excavation is complete. The wall system is supported with at least one level of struts or anchors and walers for horizontal support. It is common for DSM excavation systems to have multiple levels of support.

Quality assessment and performance monitoring procedures are recommended to meet specifications and ensure the continuity and homogeneity of the DSM wall. The experience and expertise of the project contractors play an important role in the quality of the resulting DSM walls (Porbaha, 2000).

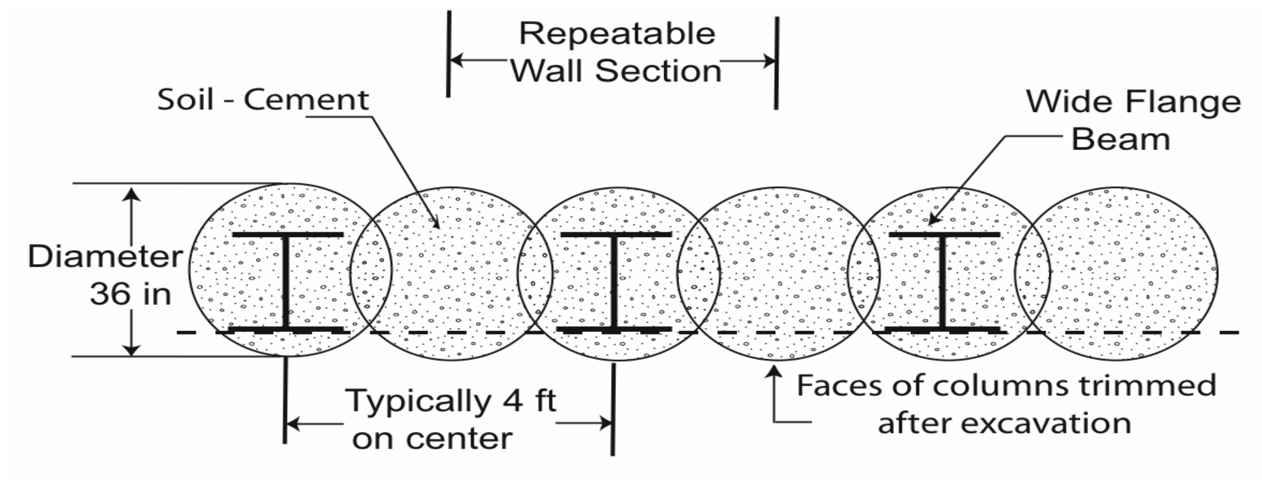



*Figure 1. Typical arrangement of DSM columns.*

## GENERAL DESIGN PROCESS FOR DSM WALLS

The goal for DSM walls is to stabilize open cuts and to minimize wall movements by restraining earth pressures. Their main purpose is to act as a permanent support system maintaining the stability of the excavation against lateral earth pressures while controlling the deformation and settlement of the surrounding structures (Porbaha, 2000). The design flowchart presented in Fig. 2 can be used to aid the engineer in the steps required for the design for DSM for excavation support (Rutherford, 2004).

The initial feasibility assessment (Step 1) of the application of DSM to a project is dependent on the site conditions and economics. Sites with ground settlement sensitivity, vibration sensitivity, high groundwater table, and/or soft soils are often good candidates for the use of DSM. Once the feasibility of DSM is determined, the functional and design criteria for the wall are established. The type of retaining wall system (Step 2) is decided based upon cost, site conditions, required wall height, speed of construction and other project specific requirements (Step 3).

A seepage analysis (Step 4) using either chart methods or computer software is carried out using initial wall geometry. Because DSM walls are often constructed in soft soils, stability against heave action is a major problem during construction (Porbaha, 2000). An external stability analysis (Step 5) is performed using classical methods such as method of slices, Bishop's method, Janbu's method and wedge method (Duncan and Wright, 2005).

All the analysis and design of DSM walls is carried out on the repeatable wall section (Step 6). The repeatable wall section is the smallest length of the wall which repeats itself over and over to form a complete wall. For DSM walls, the repeatable wall section is taken as the length of the wall between the midpoints on either side of a wide flange H-beam including the beam (Fig. 1). The vertical bearing capacity of the wall (Step 7) is determined by summing the forces acting on the soil-cement repeatable section. At the preliminary design stage, the length of wall over which the downdrag acts may be taken equal to the excavation height. More accurate analysis requires a computer aided analysis.

The wall components design issues are dependent upon the specific construction details of the wall. The geometry of the wall including the embedment and the support system are determined. The wall design can be performed using a variety of techniques alone or in combination: (1) the pressure diagram and simplified methods, (2) the beam-column method, (3) and the finite element method (FEM). Because these methods were developed with different goals, each exhibits different advantages and disadvantages which will be discussed in further detail in the following sections.

**Simplified Methods for Design of DSM Walls**

Many design guidelines recommend the use of the pressure diagram method for determining the loading on the wall components of excavation support systems (U.S. Department of the Navy, 1971; NAVFAC, 1982; U.S. Army Corps of Engineers, 1989; AASHTO, 1990; FHWA, 1998, 1999). The pressure diagram method consists of assuming an empirical pressure diagram (Terzaghi and Peck, 1967) or a theoretical active-passive pressure diagram (Canadian Geotechnical Society, 1985; Cheney, 1988) acting on the wall. The earth pressure diagrams for the design of single support/tieback walls, such as DSM walls, have been a topic of much debate. Some recommend that a triangular earth pressure diagram (as with cantilever walls) be used for single row anchored walls. Weatherby (1998) and Mueller et al. (1998) recommend that the same apparent earth pressure diagrams used to design walls with multiple support/tiebacks be used to design single support/tieback walls. The pressures are distributed to the anchors and the wall embedment either by using the tributary area method (Terzaghi and Peck, 1967), the hinge method (Lambe and Wolfskill, 1970) or using equilibrium considerations (Canadian Geotechnical Society, 1985; Cheney, 1988).

Once the wall pressures and support loads are estimated, the maximum bending moment is calculated at the point of zero shear and used to size the structural elements of the retaining system. Difficulties with the application of simplified methods arise due to the highly empirical nature of the design process and the assumptions required to simplify the soil stratigraphy and the structural system. Additionally, the pressure diagram method was not intended for the design of a stiff permanent wall and does not readily accommodate seepage analysis and unusual soil properties (Tamaro and Gould, 1992). A major limitation with the simplified methods is the lack of wall deflection and soil deformation predictions. These are especially important in the design of DSM walls, which are typically installed in soft soils where deformation estimations are vital.

The design of excavation support systems is evolving towards a deflection-based approach. Simplified methods have been developed, mostly based on case histories and parametric FEM analysis. Movements of in situ wall systems are related to the stiffness of the system. Briaud and Lim (1997, 1999) illustrate how anchor load magnitude directly influences deflection and bending moments of tieback walls. Using the proposed coefficient of apparent earth pressure (k) and relationships between k and the ratio of deflection at the top of the wall to the height of the excavation ($u_{top}/H$) or ratio of the mean deflection ($u_{mean}/H$), the engineer can select the anchor lock–off loads that will approximately generate a selected deflection (Fig. 3 and Fig. 4).

Another method of deformation control design is discussed by Clough and O'Rourke (1990). This semi-empirical method relates wall and soil mass deformations to system stiffness and base stability. Contours of maximum lateral deformation as a percentage of depth of excavation are represented in design charts for various soils. Deformations can be controlled within required limits by specifying design elements such as wall stiffness (EI), embedment depth of wall (D), and spacing of horizontal supports (h). The system stiffness (S) is determined from the wall stiffness and the spacing of the horizontal supports (Fig. 5) as

$$S = \frac{EI}{\gamma_w h^4} \quad (1)$$

where E is the modulus of elasticity of the wall material, I is the moment of inertia of a unit length of the wall, $\gamma_w$ is the unit weight of water, and h is the average spacing between supports. Calculations of the factor of safety against basal heave are used to determine an estimate for the maximum lateral deflection of the wall. Approximate profiles of lateral deformations and settlement behind the wall can be determined following guidelines presented by Clough et al. (1989).

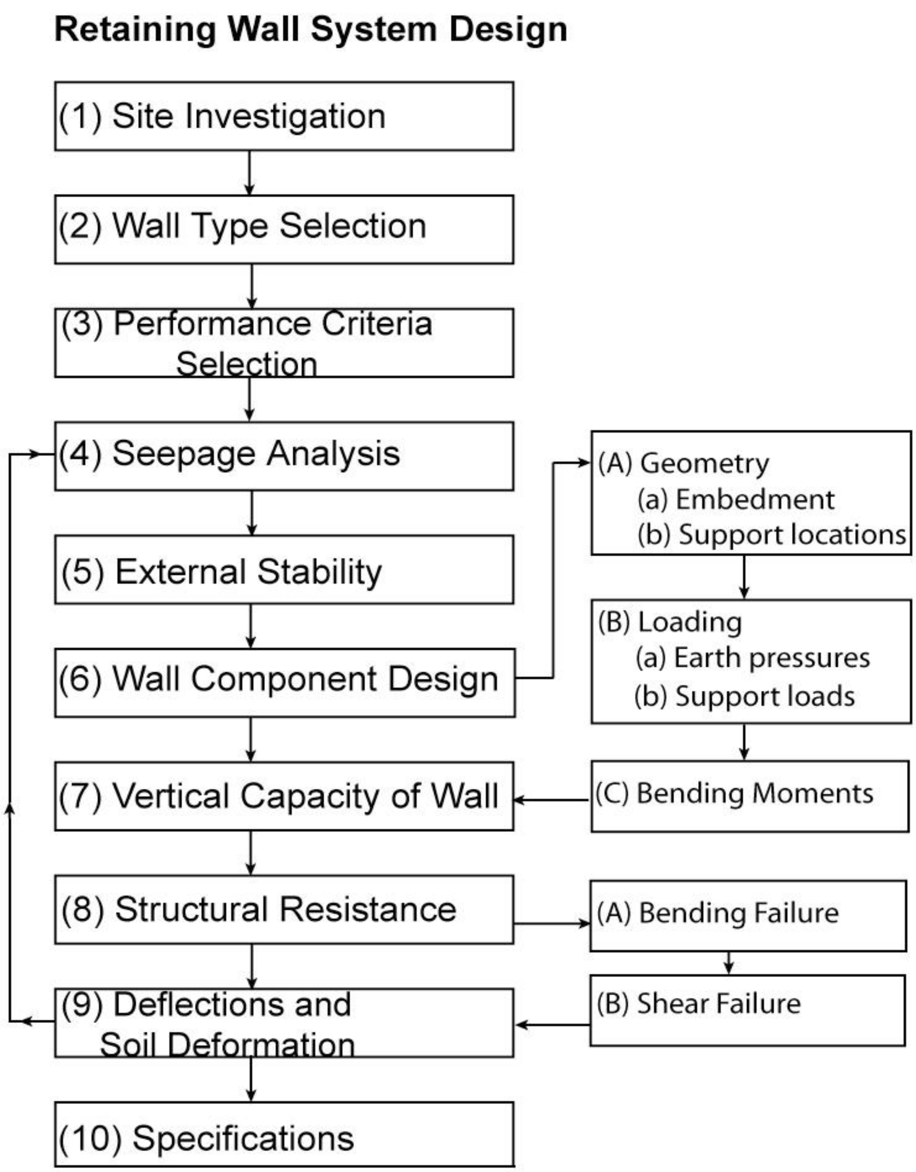



*Figure 2. DSM Wall Design Flowchart (After Rutherford, 2004).*

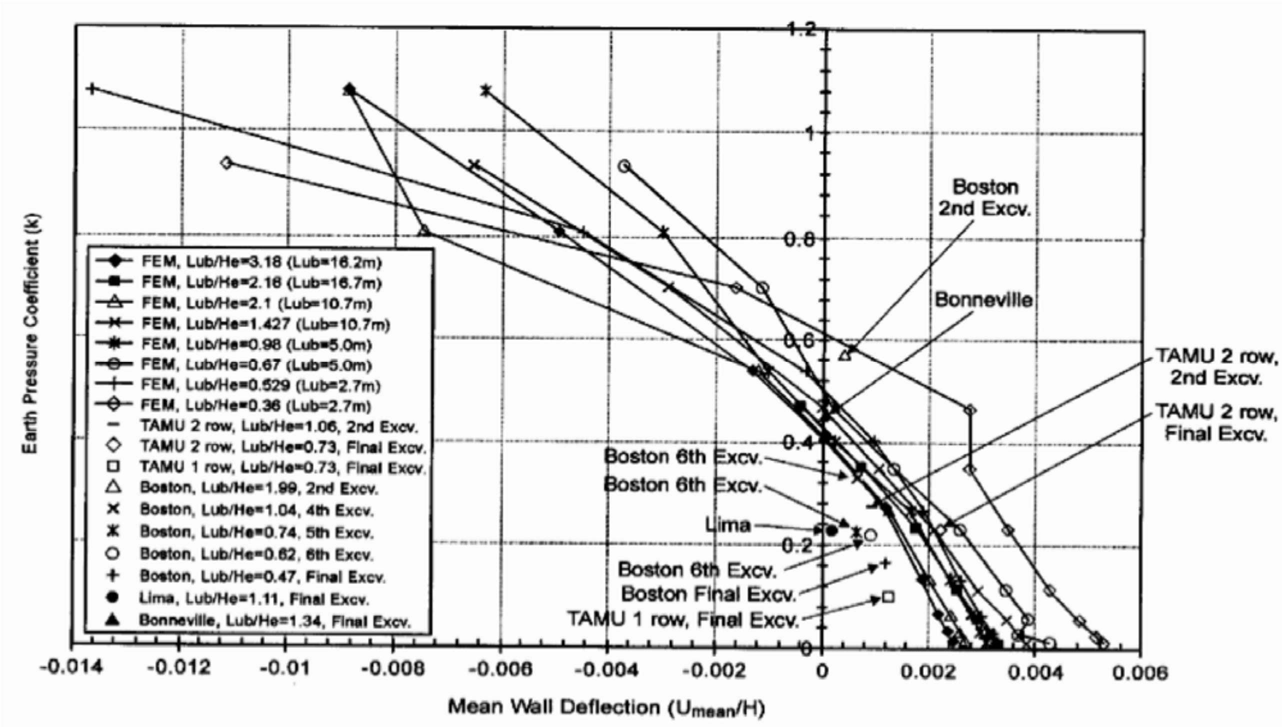



*Figure 3. Earth Pressure Coefficients Vs. Mean Deflection (after Briaud and Lim, 1999).*

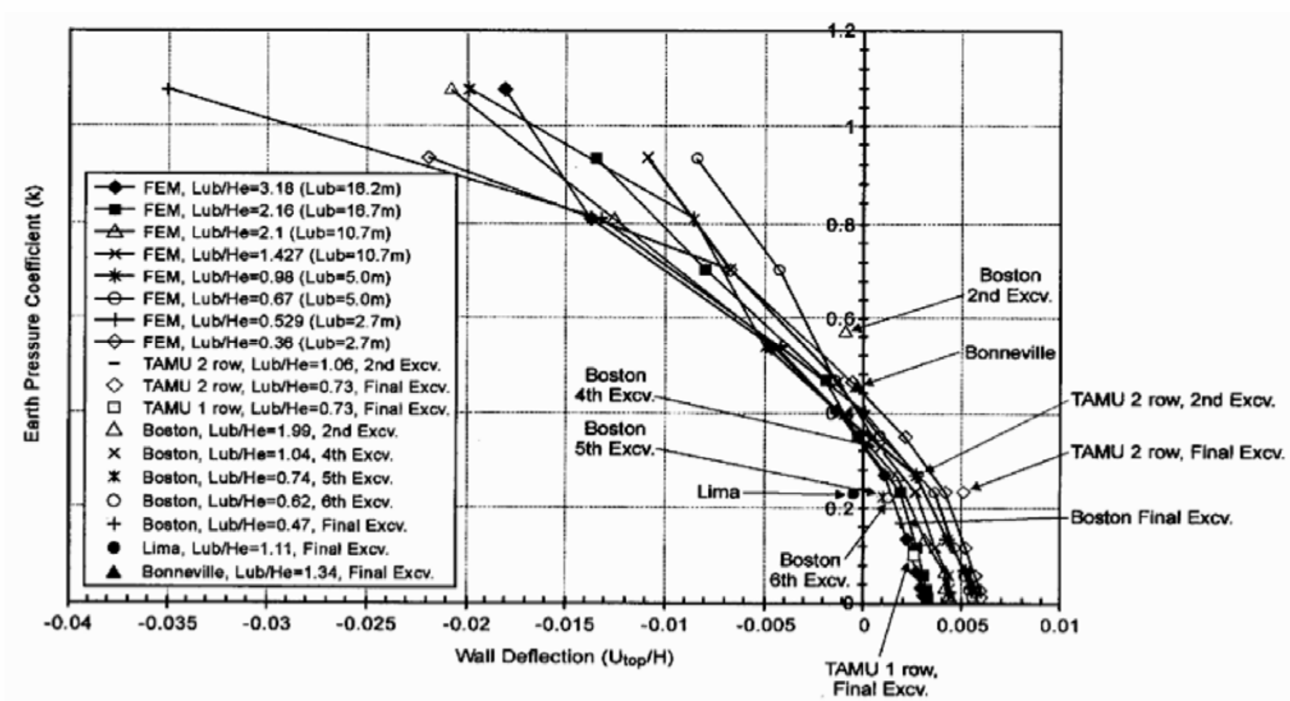



*Figure 4. Earth Pressure Coefficients Vs. Top Deflection (after Briaud and Lim, 1999).*

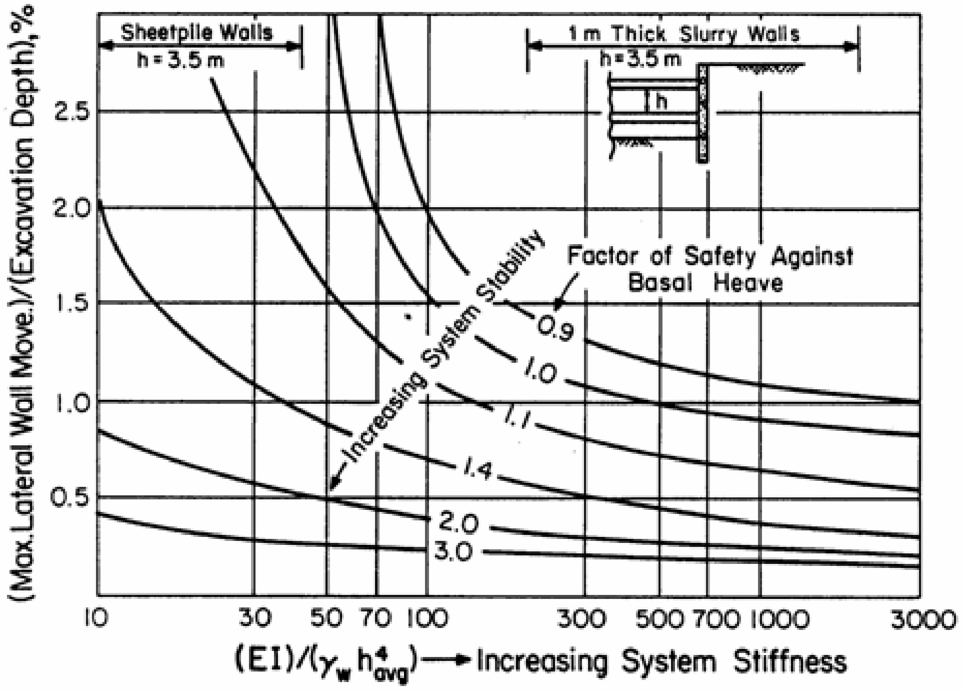



*Figure 5. Design Curves to Obtain Maximum Lateral Wall Movement (or Soil Settlement) for Soft to Medium Clays (after Clough and O'Rourke, 1990).*

**The Beam-Column Method for Design of DSM Walls**

Computer-aided analysis is growing in use and represents the future trend in excavation support analysis and design. One advantage of computer-aided analysis over simplified methods is the availability of profiles of bending moments and deflection estimates. However, these methods require detailed knowledge of soil and wall properties and construction procedures.

The beam-column method (Fig. 6) for tieback walls deals with the analysis of the wall as a structural element interacting with the soil and the anchors; it leads to sizing the wall and the anchors (Briaud and Kim, 1998). The beam-column method models the wall as a set of elements of length $\Delta z$, with bending stiffness (EI) and axial stiffness (AE). The soil is represented by a series of vertical and horizontal springs placed at the nodes between the elements.

The springs are characterized by load-deflection or p-y curves. The equilibrium of a wall element under soil and support loads leads to the governing differential equations, which are solved by the finite difference technique. This method originates from the work of Winkler (1867) and Hetenyi (1946), but Matlock was one of the first to apply the general computer solution for the beam-column analysis to this reference (Matlock et al., 1981), while Halliburton (1968) first applied it to the flexible retaining wall problem. The beam-column method allows the estimation of a deflection profile, a shear force profile, a bending moment profile and an axial-load profile for the wall.

In the analysis of DSM walls with the beam-column method, only the stiffness, EI, of the H-beam reinforcement is modeled, while the flexural stiffness added by the soil-cement is ignored. Based on laboratory testing, the bending stiffness (EI) of soil-cement columns is approximately 2 to 4% of the EI of the reinforcing beams (Wong et al., 1993). Rutherford et al. (2005) illustrated that this assumption does not greatly affect the resulting bending moments and deflections. Wong et al. (1993) used the beam column method to predict the performance of a deep soil mixed wall in Texas.

**The Finite Element Method for Design of DSM Walls**

The finite element method, FEM allows great flexibility, but requires a number of input parameters including the material parameters for the wall (usually elastic), the anchors (elastic–plastic), and the soil (hyperbolic non-linear elastic or other), detailed boundary conditions and boundary loads (surcharge, buildings, etc.).

Determination of boundary conditions for finite element models is one of the first steps in the numerical simulation (Fig. 7). In order to minimize the effect of the boundaries on the predicted ground movements, the mesh boundaries can be determined using Briaud and Lim (1997) recommendations. The width of the mesh is

$$B_e + W_e \tag{2}$$

where $B_e = 3(H_e + D)$, $W_e = 3D$, $H_e$ is the height of the excavation and D is the depth from the bottom of the excavation to the hard layer.

The simulated wall section or repeatable wall section must also be determined. Because the soil-cement is weak in tension its contribution to the stiffness of the wall is neglected; therefore the stiffness of the section is based solely on the stiffness of the wide flange beam.

A drawback of using the finite element method is the number of soil parameters required for meaningful finite element models. Additionally, some finite element programs are difficult and time consuming to use. The use of the FEM is becoming simpler and simpler, but quality control of the results is critical.

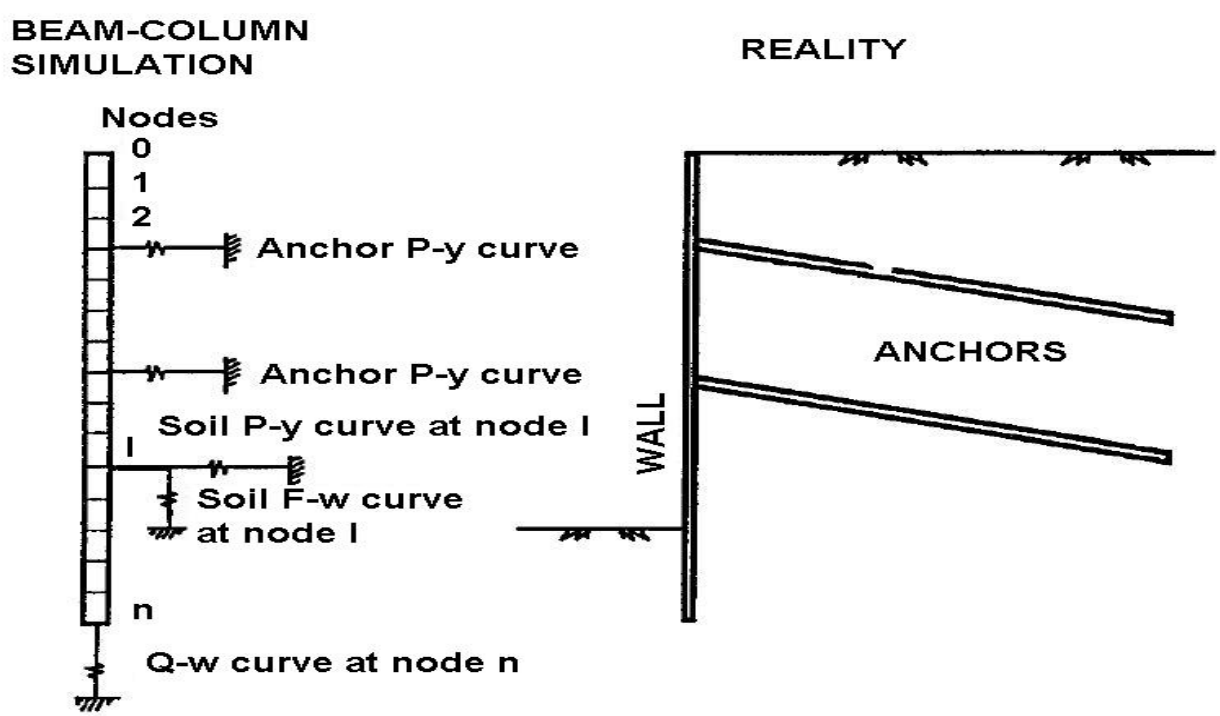



*Figure 6. Beam-Column Method Schematic (after Briaud and Kim, 1998).*

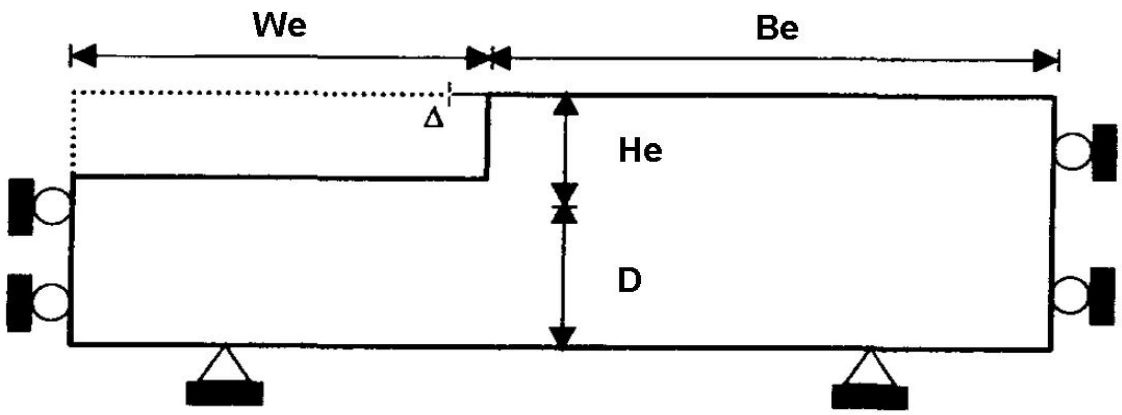



*Figure 7. Finite Element Method Schematic (after Briaud and Lim, 1997).*

**STRUCTURAL DESIGN**

The structural design (Step 8) of the components of DSM walls is the aspect in which DSM wall design most differs from traditional excavation support design. The approach for soil-cement walls most resembles that for soldier pile and lagging walls or secant walls. The design methodology includes steps to determine the reinforcement members to resist bending moment and shear stresses. The tensile strength of soil-cement is low; therefore designing soil-cement columns to resist bending stresses is not economical. The soil-cement between the reinforcement members is considered as lagging and is designed to resist and redistribute the horizontal stresses to the adjacent reinforcement (Taki and Yang, 1991). The stress analysis of the soil-cement between the reinforcement beams includes the evaluation of shear and bending moments in the horizontal plane and stresses inside the soil-cement mass as illustrated in Fig. 8.

The steel reinforcement is designed to resist bending stresses in the vertical plane due to the lateral earth pressures transmitted by the soil-cement to the steel beams. The wide-flanges exhibit bending in the vertical plane while the soil-cement, acting as lagging, bends in the horizontal plane. The most critical shear location is at the junction of the soil-cement and steel beam and requires calculations to check for bending failure and shear failure.

Based on finite element studies by Taki and Yang (1991), an empirical design criterion was developed to avoid bending failure in the soil-cement. Bending failure is unlikely if

$$L_2 \leq D + h - 2e \tag{3}$$

where $L_2$ is the distance between the end of the wide flange beams, D is the diameter of the soil-cement column, h is the height of the wide flange beam, and e is the eccentricity defined as the distance between the center of the wide flange beam and the center of the soil-cement column after trimming.

Shear failure is checked following recommendations by Pearlman and Himick (1993). The nominal shear strength of the soil-cement can be estimated similarly as that of concrete. Using the American Concrete Institute (ACI) equation, the shear resistance of the soil-cement block, $V_c$ (lbf) is defined as:

$$V_c(lb_f) = \lambda 2\sqrt{f_c'(psi)}\,b_w(in)\,d(in) \tag{4}$$

where $\lambda$ is estimated as 0.75 for lightweight concrete, $f_c$' is the soil-cement shear strength in psi, $b_w$ is the width of the block in inches and d is the height of the block in inches. The shear resistance of the block, $V_c$, must be greater than the shear force applied to the wall.

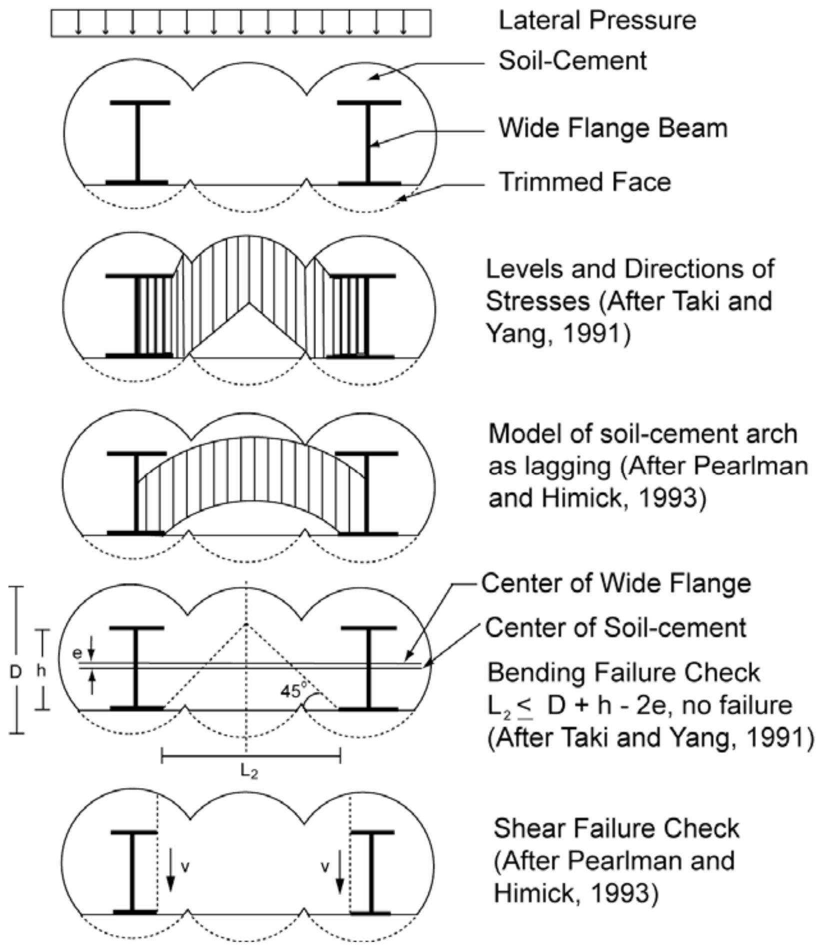



*Figure 8. Structural Resistance of Wall Elements.*

## CASE HISTORY

The Islais Creek Transport/Storage Project in San Francisco, California included the construction of approximately 500 m of box sewer along Army, Indiana and Tulare Streets. A portion of the box sewer also serves as a controlled overflow structure in the event that the storage capacity of the system is exceeded. Construction of the sewer and overflow structure required approximately 12-m deep excavations in relatively soft soil with a high groundwater table. Key issues for the excavation were stability and deformations particularly along the overflow structure where weak subsurface soils extended to depths of up to 33 m (Dames & Moore, 1997).

A segment of the Army Street DSM wall will be used to illustrate the three wall component design methods. For the shoring system, deep soil mixing was selected to construct a diaphragm wall with wide flange beams spaced at 1.3 m. The soil cement columns were 0.91 m in diameter and wide flange beams (W30 x 108, EI = 304,565 kN·m2/m) were inserted 1.3 m center-to-center. Three levels of internal struts were used to brace the excavation walls at 0.9 m, 4.9 m and 9.1 m. The excavation was 11 m wide and 11.7 m deep. The subsurface soil consisted of layers of fill to a depth of 5.5 m, a layer of dark gray, soft to medium stiff, plastic clay, known locally as Bay Mud, to a depth of 14.0 m, Marine Sand to a depth of 15.8 m and Colluvium to a depth of 18.3 m. The soil layers are underlain by bedrock of the Franciscan Formation. The soil parameters used in the analysis are in Figure 9. Based on limited knowledge of the soil properties, conservative parameters were selected and only a drained analysis was conducted. An external stability analysis was performed using Bishop's method. Satisfactory external stability was determined. The seepage analysis in Plaxis showed limited flow and no piping or unstable conditions likely to develop.

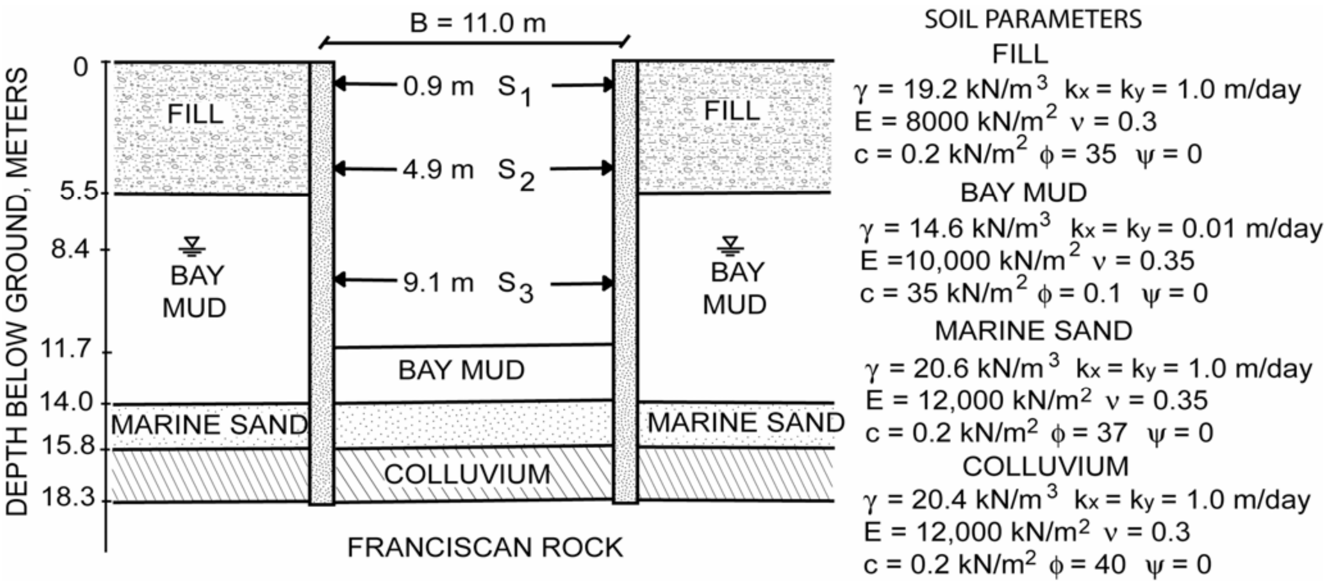



*Figure 9. Islais Creek Transport/Storage Project Assumed Section..*

**Simplified Methods for Design of Case History**

Selecting a pressure diagram for the design is very difficult in the case of a multiple layer stratigraphy. Simplified profiles must be used to facilitate the calculations required in the simplified method. Since the least competent soil controls the lateral earth pressure, the Bay Mud was combined with other layers into one design soil unit.

Two different earth pressure diagrams were constructed (Fig. 10). First, a trapezoid-shaped diagram with a maximum pressure of 51.25 kN/m$^2$ was calculated using an apparent earth pressure of (Weatherby, 1998):

$$p_{AEP} = 0.3\gamma H \tag{5}$$

where $p_{AEP}$ is the apparent earth pressure, $\gamma$ is the total unit weight of the soil and H is the height of the excavation. Using the tributary area method, the strut loads were determined as $S_1$=95.8 kN, $S_2$=288.1 kN and $S_3$=196.4 kN.

The second diagram follows suggestions by Weatherby (1998) for multiple strut walls. A maximum pressure of 45.97 kN/m$^2$ was calculated using the "25H" trapezoid diagram. Using the tributary area method, the strut loads were determined as $S_1$=79.29 kN, $S_2$=189.14 kN and $S_3$=147.46 kN.

Once the pressure diagram and the anchor loads are known, the shear and bending moment diagrams can be drawn. The maximum moment for each diagram was calculated, as is typical in practice, at the depth of the first strut (3.14 kN·m and 2.82 kN·m, respectively). Both of these moments can be safely resisted by the W30x108 wide flange beam, which was selected as reinforcement for the soil-cement columns. A major limitation of this design method is the lack of wall deflection predictions and the assumption that the maximum bending moment occurs at the first strut. The validity of this assumption will be discussed later.

The lateral displacement of the wall can be predicted using the deformation control method as described by Clough and O'Rourke (1990) and Briaud and Lim (1999). Based on the Briaud and Lim method (Fig. 3 and 4), the estimated mean deflection could range from 11.7 mm to 47.6 mm, and the estimated deflection at the top of the wall could range from 29 mm to 46.8 mm using k equal to 0.3 and H equal to 11.7 m.

Based on Clough and O'Rourke (1990), the shoring system stiffness (S) from Equation 1 was 128 given wall stiffness (EI) of 304,565 kN·m$^2$/m (neglecting the soil-cement) and average vertical spacing between the strut supports (h) of 4.1 m. The factor of safety against basal heave was estimated using the software program, XSTABLE, as 1.5. The estimated maximum horizontal movement of the system was 0.5% of the depth of the excavation, or 58 mm from Fig 5.

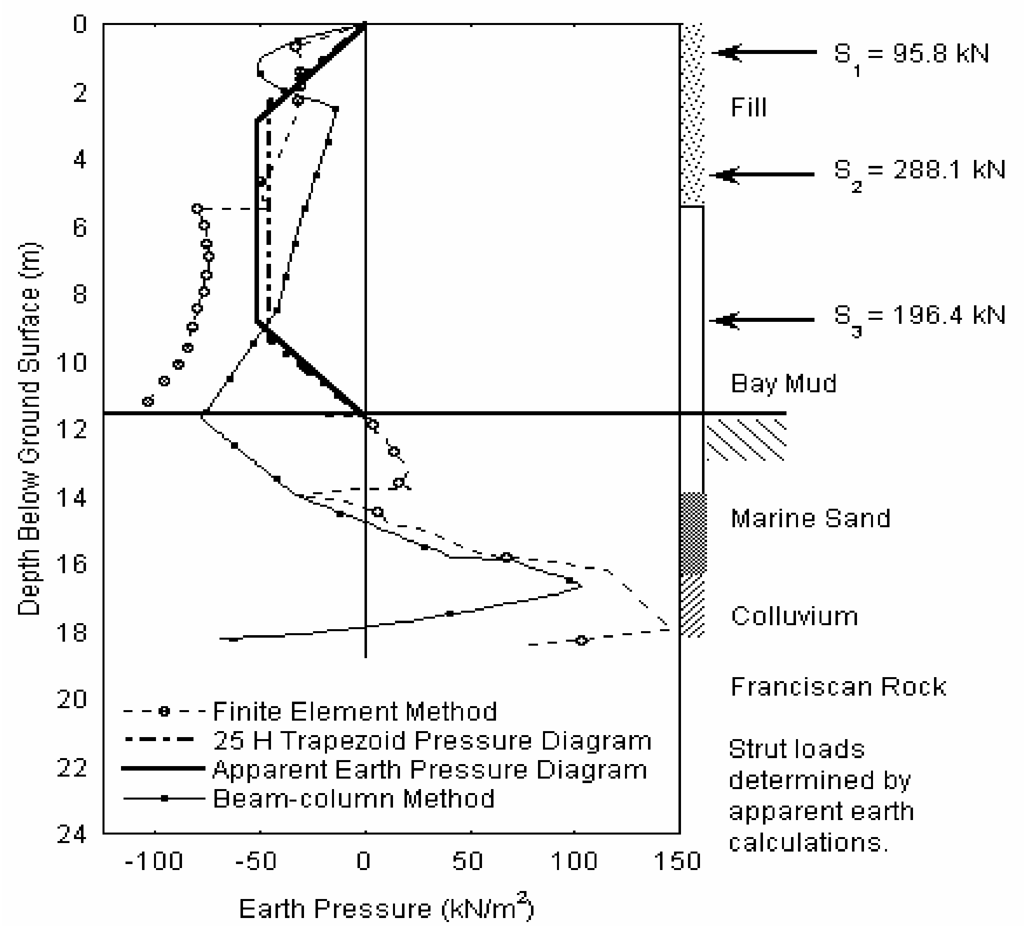



*Figure 10. FEM and Beam-column Earth Pressures on Wall.*

**The Beam-Column Method for Design of Case History**

The beam-column method requires the development of load-deflection curves, or p-y curves, to represent the soil layers and the supports. BMCOLM76 (Matlock et al. 1981) was used to implement the beam-column approach for the analysis of the DSM wall. The width (b) of the repeatable wall section was selected as the horizontal spacing between the vertical reinforcement (steel beams) or 1.3 m. The wall stiffness (EI) of the section was taken as 304,465 kN·m$^2$/m or the stiffness of the reinforcing beam only, ignoring the contribution of the soil-cement. The strut loads were determined using the tributary area method from the apparent (0.3 $\gamma$ H) earth pressure diagram as $S_1$=95.8 kN, $S_2$=288.1 kN and $S_3$=196.4 kN. P-y curves were determined for multiple points in each layer using the soil reactions and the deflections of the wall in passive and active resistance following recommendations of Briaud and Kim (1999) (Fig. 11). Table 1 provides the earth pressure coefficients and deflections used for each soil layer. In the zone where the wall had both sides in contact with soil (below excavation level), the p-y curves were constructed according to Briaud (1992). Fig. 10 compares the pressure diagram from the beam-column method with the diagrams used in the simplified method. Soil reactions in the vertical direction were neglected for this study. Additionally, the construction sequence was not simulated.

*Table 1. Earth Pressure Coefficients and Displacements*

| Soil Type (1) | Range of Depth (m) (2) | $K_a$ (3) | $K_p$ (4) | $y_a$ (mm) (5) | $y_p$ (mm) (6) |
|---|---|---|---|---|---|
| Fill | 0 - 5.5 | 0.27 | 3.70 | 1.27 | 12.7 |
| Bay Mud | 5.5 – 14.0 | 0.28 | 3.54 | 25 | 5.1 |
| Marine Sand | 14.0 – 15.8 | 0.25 | 4.00 | 1.27 | 12.7 |
| Colluvium | 15.8 – 18.3 | 0.22 | 4.60 | 1.27 | 12.7 |

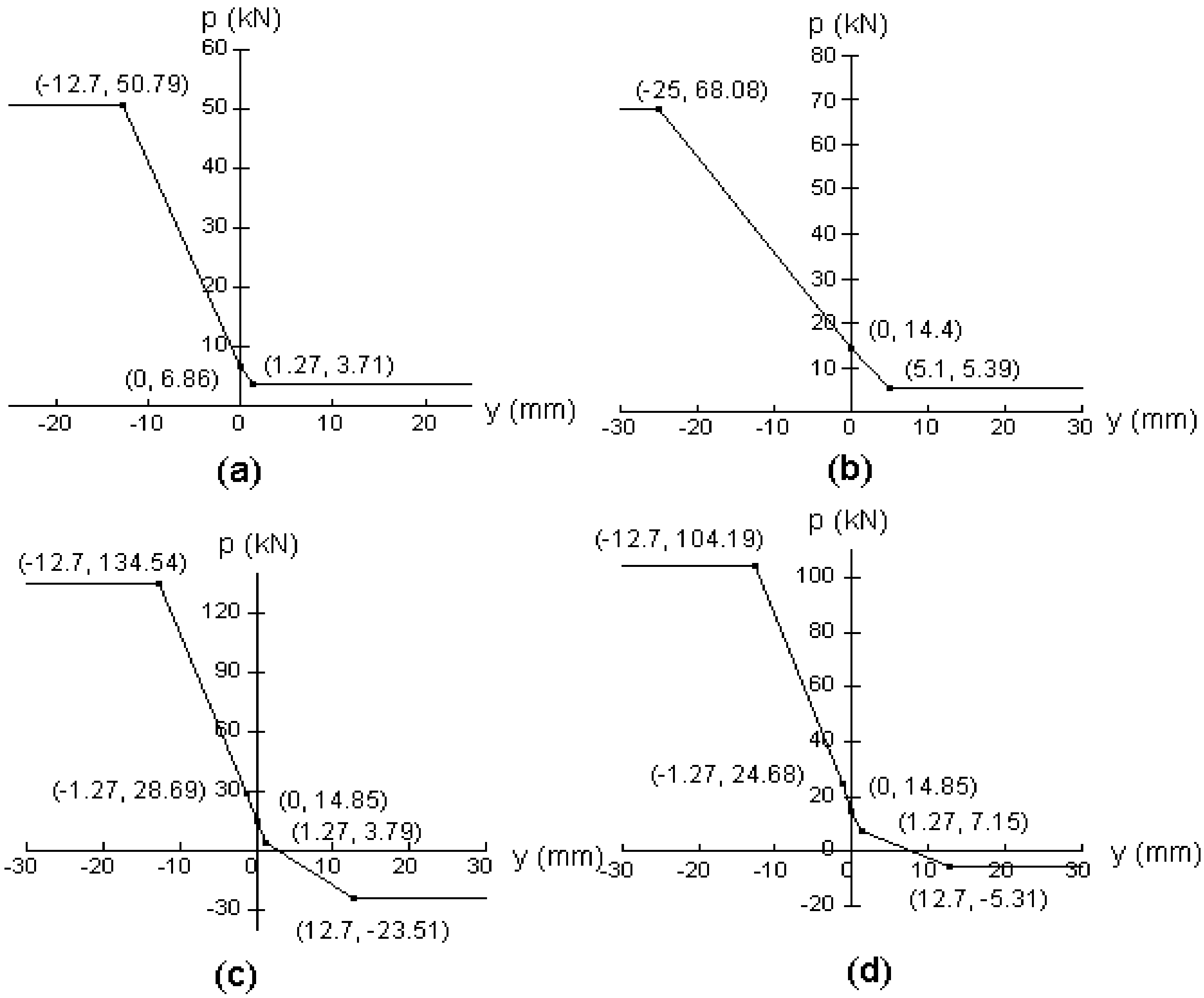



*Figure 11. P-y curves (a) located at 5.5 m depth in Fill (b) located at 8.4 m in Bay Mud (c) located at 15.8 m in Marine Sand (d) located at 18.3 m in Colluvium.*

**The Finite Element Method for Design of Case History**

A number of finite element method programs are available. Plaxis (Brinkgreve and Vermeer, 1998) was used to model the DSM wall case history. The Mohr-Coulomb elastic-plastic model was used to simulate the response of the soils. The choice was mandated by the lack of information on the soil properties beyond strength. Additional soil parameter assumptions were based on reported properties at nearby locations. Due to symmetry, only half of the excavation was analyzed in the simulation. The width of the mesh was 95.5 m and the height was 22.0 m based on recommendations by Briaud and Lim (1997). The finite elements mesh consisted of plane strain, 15-node elements and was generated at a very fine global coarseness, additionally refined around the wall. Soil parameters are given in Fig. 9.

The construction sequence was simulated using several phases. The initial conditions were assigned based on a phreatic ground water level at 8.4 m. The pore water pressures were generated assuming hydrostatic conditions and then the initial stresses were calculated using the Plaxis $K_0$-procedure. The strut loads were estimated as previously discussed in the beam-column method section. First, the wall was installed to the desired depth (18.3 m), the soil was excavated to 1.83 m, the first level of struts was installed at 0.91 m, and pre-stressed to 95.8 kN or 73.67 kN/m (Phase 1). Phase 2 consisted of additional excavation to 6.71 m and installation of the second level of struts at 4.88 m, pre-stressed to 288.1 kN or 221.6 kN/m. Phase 3 required a third excavation to a depth of 11.7 m and installation of the final level of struts at 9.1 m pre-stressed to 196.4 kN or 151.1 kN/m. Since this phase consists of excavating to a depth below the ground water, de-watering of the excavation was undertaken. To reflect this condition, the phreatic water level in the FEM was drawn down. The finite element simulations provide bending moment estimates and predictions of wall deflection and soil deformation (Fig. 12 and 13). More detail of the simulations is provided by Rutherford (2004).

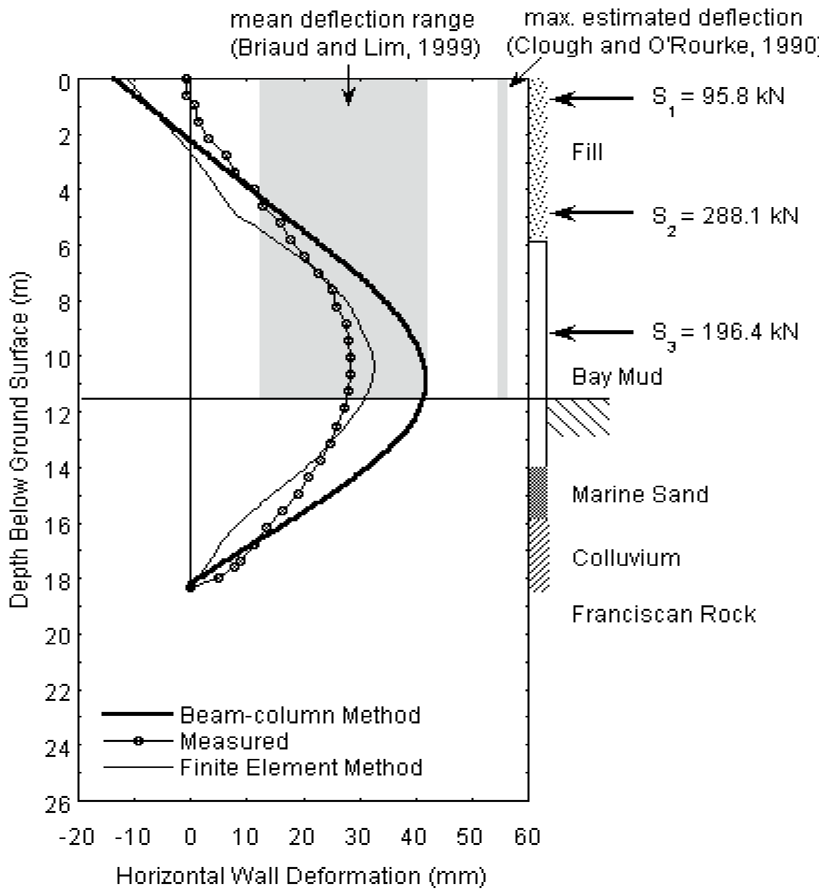



*Figure 12. Horizontal Wall Deformation Comparison.*

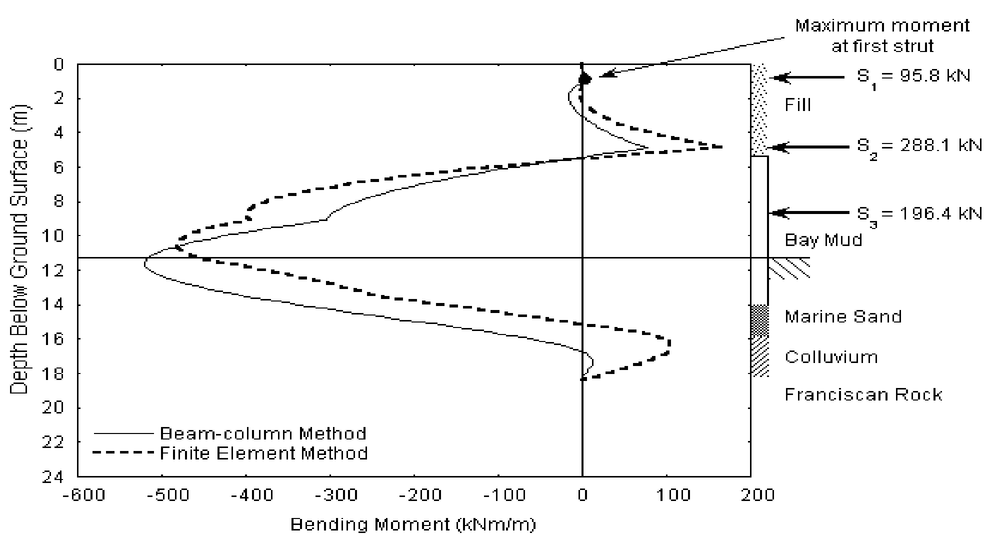
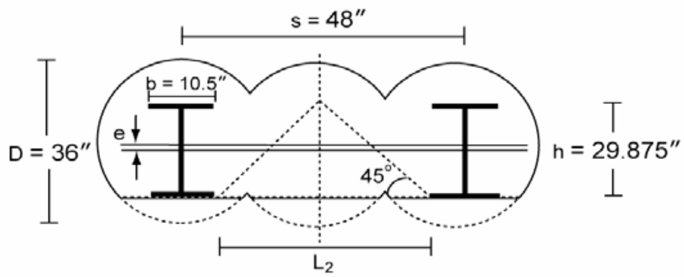



*Figure 13. Bending Moment Comparison.*

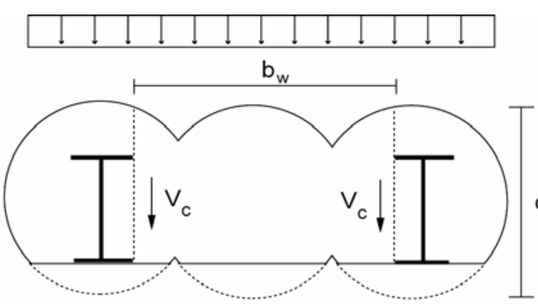
**Structural Design of Case History**

The structural design of the DSM wall is performed following methods recommended by Taki and Yang (1990) and Pearlman and Himick (1993), as described in previous section. Fig. 14 illustrates the calculations required for bending failure check and Fig. 15 illustrates the calculations required to check for shear failure. Because the distance between the reinforcement was less than the sum of the diameter of the soil-cement and the height of the reinforcement, the soil-cement will not fail in bending. Shear failure was checked by ensuring that the shear resistance of the soil-cement is greater than the shear transmitted to the wall by earth pressures. Therefore, the H-section is not overstressed.




If $L_2 < D + h - 2e$ then no bending failure
W30x108 steel reinforcement
Assuming e = 0
Beam spacing, s = 48″
Diameter, D = 36″
Height of beam, h = 29.875″
Width of beam, b = 10.5″
$L_2 = s - b = 48″ - 10.5″ = 37.5″$
$D + h - 2e = 36″ + 29.875″ - 0 = 65.875″$
$L_2 < D + h - 2e$
37.5″ < 65.875″ therefore no bending failure

*Figure 14. Bending Failure Calculations.*




$$V_c = \lambda 2\sqrt{f_c'}\, b_w d$$

where λ for lightweight concrete is estimated as 0.75
$f_c$ for soil-cement is assumed to be 2,000 kPa = 290 psi
$b_w$ is the width of the block = 39″
d is the height of the block = 36″

Therefore,

$$V_c = 0.75 \cdot 2\sqrt{290 psi}(39")(36")$$

$$V_c = 35,863.9\ lb = 159.5\ kN$$

Ensure that $V_{max} < V_c$

*Figure 15. Shear Failure Calculations.*

## DISCUSSION

Fig. 12 shows the measured wall deflections for the case history compared with estimated deflections from the simplified methods, beam column method, and finite element method. The deformation based methods predicted a maximum lateral deformation of 58 mm (Clough and O'Rourke, 1990) and a range of 11.7 mm to 47.6 mm (Briaud and Lim, 1999). All three methods compare favorably to the measured deformations that ranged between 38 and 56 mm for the wall segment. The finite element and beam-column predictions tend to overestimate the maximum deflection; this may be due to the conservative assumptions for the soil parameters based on the limited knowledge of the soil properties. The negative deflection at the top of the wall predicted by the finite element and beam-column methods are likely due to the fact that no hysteresis is considered in the simulation, but occurs due to soil plasticity in reality. The actual strut loads for the site were unavailable; therefore, the loads were taken as those calculated using the tributary area method.

The differences between the methods are additionally illustrated by comparing the bending moment profiles (Fig. 13). Using the simplified method, the design maximum bending moment is calculated at the top strut. Although the bending moment magnitude at this location calculated by the simplified method is relatively close to those predicted by the finite element and beam-column method, the bending moment prediction by the simplified method is much smaller than the maximum bending moment which occurs much farther along the wall at a location closer to the excavation depth. The finite element and beam-column methods provide bending moment profiles along the length of the wall. Both methods provide similar shaped bending moment curves and maximum values, although the finite element method estimates a larger moment at each strut.

The FEM method has the ability to simulate multiple phases of construction easily whereas to do so using the beam-column method is quite time consuming. Briaud and Kim (1998) show how to simulate the construction sequence with the beam-column method and indicate that it makes little difference in predicted bending moment profiles compared to a one step excavation; however, it does provide deflections which are closer to the measured deflections. The actual bending moments for the site were unavailable.

Pressure diagrams are very simple but use restricting and often unrealistic assumptions; additionally, they do not lead to deflection predictions. This limits the effectiveness of these methods for DSM wall design. The beam-column method is of intermediate complexity, theoretically sound, and includes deflection and bending moment profile predictions. However, it cannot predict settlement behind the wall. The finite element method is more complicated and requires extensive knowledge of the soil as well as the construction technique; however, it provides the most realistic deformation, deflection and bending moment predictions.

The design of the overall system for DSM walls is similar to traditional excavation support systems; however, the design of the wall components is unique to DSM walls. Ensuring safety against failure in bending and shear in the soil-cement is an important addition to the design of the structural resistance of the wall system. This additional step is carried out through a simplified method since most 2-D numerical tools cannot handle composite cross sections.

## CONCLUSION

DSM walls are used more and more because of the advantages they provide over traditional excavation support. This paper presented a design process providing more standardized procedures including alternatives suitable for simplified calculations and computer-aided analysis. The design flowchart was illustrated through a case history and the results of various methods were compared.

The simplified method is easier to use, but can only be applied to much simplified soil stratigraphies. However, the limitations associated with deflection and bending moment estimates suggest that this method should only be used as a preliminary tool for the design of DSM walls. The beam-column method provides both wall deflection predictions and bending moment estimates; however, one must keep in mind the limitations associated with soil movement predictions. Finite element simulations allow for more realistic ground deformation and wall deflection predictions for DSM supported excavations. FEM also allows simulation of different phases of the construction sequence permitting direct evaluation of the displacement at each stage of construction. These are advantageous for DSM walls, which are often installed in soft soils where reliable ground deformation predictions are needed.

Future developments should include more detailed analysis of soil-cement acting as lagging and its contribution to resist bending. Currently, DSM design does not account for any contribution of the soil-cement to the bending stiffness of the wall.

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