

PRECAST CONCRETE SECONDARY CONTAINMENTS FOR ABOVEGROUND STORAGE TANKS CONTAINING PETROLEUM PRODUCTS; A SIMPLIFIED DESIGN APPROACH

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Abstract

Secondary containments are installed surrounding aboveground storage tanks containing petroleum products to prevent soil, groundwater and surface water contamination in the event of a leak or to prevent spread of fire to adjacent properties in the event of a fire. Requirements for secondary containments are strictly regulated through industry standards and guidelines developed by environmental regulatory authorities of federal and provincial/state governments across North America.

Earthen dykes containing impermeable layer/liner are often installed surrounding large storage tanks to provide secondary containment. Desired containment volume is achieved by increasing the area of containment or the height of dyke or both. Earthen dykes can be expensive to maintain and clean in the event of a leak. Earthen dykes require considerably more real estate than dykes built using concrete walls supported by a system of lateral bracings. Concrete dykes can be a cost effective alternate solution when compared with the lifecycle cost of earthen dykes.

The economic design can be achieved by optimizing dyke wall thickness and/or lateral support spacing. Slenderness and crack control are critical design parameters for an optimized design. Hence the design of optimized concrete dykes is governed by displacement criteria. The mid-span lateral displacement due to hydrostatic pressure can be controlled by embedding a weaker steel member at the top of the concrete wall panel and a stronger steel member at the bottom of the concrete wall panel that is designed to allow uniform mid-span lateral displacement across the wall. Strength of the wall panel is calculated by enforcing displacement compatibility of steel and concrete.

This paper will outline a simplified design method for concrete dykes and discuss design parameters, methodologies, tools, and limitations. Finite element analysis will be used to validate the proposed design methodology.

Keywords: Concrete Dyke, Secondary Containment, Slenderness, Crack Control Parameter, Finite Element Analysis.

1. Introduction

Secondary containment provides an essential line of defence in the event of a leak or a failure of an oil container (primary containment), such as a bulk storage tank, a mobile or portable container, pipes, or other oil-filled operational equipment. The secondary containment provides a temporary containment for spilled oil until appropriate actions are taken to abate the source of the discharge and remove the spilled oil before it reaches navigable waters and adjoining shorelines. The entire containment system, including walls and floor, must be constructed so that any discharge from a primary containment system will not escape the containment system before clean up occurs.

A secondary containment shall be provided for a single or multiple aboveground storage tanks using permanent, engineered barriers, such as dykes, berms, raised earth embankments or concrete containment walls sufficiently impervious to contain oil.

The regulations of Environment Canada - Part 3 CCME PN 1326 (2003) and Environmental Protection Agency of USA, Chapter 4: Secondary Containment and Impracticability (2005) specify the requirements of secondary containments for all areas with a potential for a discharge of harmful substances from primary containments. In general, provisions for secondary containment require that the chosen containment type be sized to contain the entire volume of the largest oil tank plus “sufficient freeboard” to contain precipitation.

An engineered secondary containment system such as a dyke, berm or concrete wall system should include (CFR Directive 40 Chapter 1 (2003)): the required capacity to contain oil and sufficient freeboard to contain precipitation in accordance with good engineering practice and the requirements of the rules applicable to the jurisdiction; no cracks in containment material (e.g. concrete, liners, coatings, earthen materials) or discoloration and the materials meet the permeability requirements; no spilled or leak material (standing liquid); no corrosion or erosion of the system; Operational provisions to drain and drainage controls; Preventive measures for burrowing animals creating holes or penetration to the containment system.

2. Design Requirement for Secondary Containment

As per Environment Canada - Part 3 CCME PN 1326, section 3.9, a secondary containment system for an aboveground storage tank shall be sized to: “1). for a storage tank system that consists of a single storage tank, have a volumetric capacity of not less than 110% of the capacity of the tank; or 2). for a storage tank system that consists of more than one storage tank, have a volumetric capacity of not less than the sum of: the capacity of the largest storage tank located in the contained space; and 10% of the greater of: the capacity of the largest storage tank located in the contained space; or the aggregate capacity of all other storage tanks located in the contained space.”

The area surrounding a single-walled aboveground storage tank must have a secondary containment system designed to contain leakage. Secondary containment systems must consist of an impervious

liner and a dike. The area within the secondary containment system must be graded to a sump or low-lying area (within the diked area) to allow for the collection of rainwater, snow-melt water, and any possible leakage from the tanks. No uncontrolled discharge of collected fluids or discharge of untested fluids is permitted.

A dike must 1). be constructed of soil, steel, concrete, solid masonry, or synthetic material and designed to contain liquids within the diked area, to be able to withstand the hydrostatic head associated with it being full of liquid, and so that it will not deteriorate or develop leaks during the projected life of the structure; 2). be sized to have a required volumetric capacity; 3). have no openings in it; and 4). be maintained in good condition. The area encompassed by the dike must be kept free from weeds, debris, and extraneous combustible material.

A liner must; 1). be of a material that is inert to or compatible (chemically resistant) with the material being stored in the tank; 2). be impervious; 3). be durable and appropriate for the operating in various ambient conditions; and 4). cover the area within the dike, including the area beneath the tanks, and be keyed into the dike walls.

3. Earthen Dykes

Earthen dykes are the most commonly used secondary containment system across North America. Earthen dykes (Figure 1) are built using well graded clay materials with a minimum of 1 m wide levelled top, 2.5: 1 to 3:1 side slopes with a minimum of 0.6 m thick impermeable Compacted Clay Liner (CCL) inside the dike and on the base of the entire containment. The design height of the dike depends on the required containment volume. For the purpose of secondary containment, the clayey soil must be compacted to achieve a hydraulic conductivity of 1×10^{-6} cm/s or less determined in situ or 1×10^{-7} cm/s or less determined in a laboratory from a representative disturbed sample (material must meet hydraulic conductivity requirements under full hydrostatic head) as per ERCB directive 55 (2001).

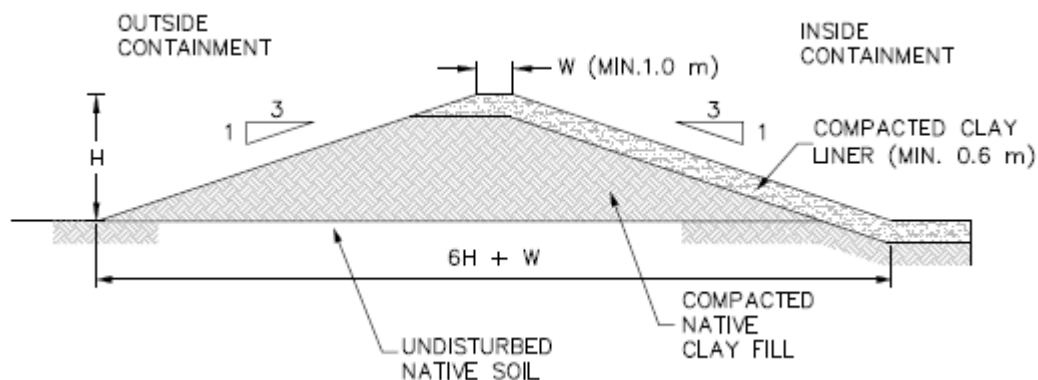


Figure 1: Typical cross-section of an earthen dike

A hydraulic conductivity of $\leq 1 \times 10^{-7}$ cm/s is achievable if suitable starting material (clayey soil) is excavated, reworked, or homogenized and laid down and compacted in lifts following appropriate construction protocols on a properly prepared sub-base. Well graded clay material meeting ERCB specifications should be compacted to a minimum of 95% Standard Proctor Maximum Dry Density at 2% to 3 % of optimum moisture in lifts not exceeding 150 to 200 mm. The construction of earthen dykes with CCL requires an application by qualified personnel overseen by an experienced professional geotechnical engineer.

4. Concrete Dykes

Usage of precast concrete dykes (Figure 2) is becoming increasingly popular as a secondary containment for large aboveground storage tanks, due to the fact that: the earthen dykes require considerably more effort, time and quality control to build; are often subjected to vegetation, erosion and abuse by burrowing animals; are difficult and expensive to maintain over the service life; are significantly more expensive to clean up in an event of a leak (or decommission upon reaching its service life); and require considerably more real estate.

Precast concrete dykes require considerably less real estate; are relatively easy to install; are easy to maintain (and decommission); can be reused; and are aesthetically appealing.

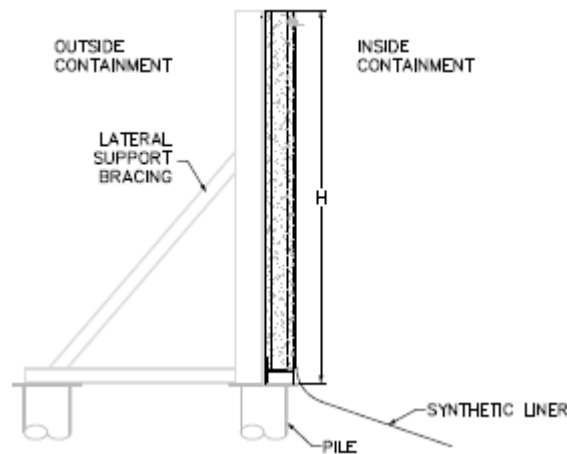


Figure 2: Typical cross-section of a concrete dyke

The precast concrete panels designed to support full hydrostatic pressure are often supported by steel lateral support braces on piles foundations. The lateral braces for relatively short walls can be supported on concrete sleepers in place of piles.

Existing earthen dykes are often replaced by concrete dykes when increased secondary containment volume is required due to adding new tanks or increasing volume of the primary containments, especially when additional real estate for containment is not available.

5. Example: Earthen Vs Precast Concrete Wall Panel Dykes

This section compares secondary containment sizing requirements for an 80,000 m³ (503,185 bbl) aboveground storage tank using an earthen dyke and a concrete dyke.

The required size of the secondary containment is 88,000 m³, which is 110% of primary containment of the tank volume.

The required height of a concrete dyke to contain 88,000 m³ on a 200 m x 200 m footprint is 2.58 m. The concrete wall panels are supported by 1.5 m wide lateral support bracings at 6 m spacing. The volume of concrete required is 406 m³ for 200 mm thick wall panels.

To maintain a 200 m x 200 m footprint, the required height of an earthen dyke with a 3:1 side slopes and a 1 m wide flat top is 3.42 m. The required volume of fill clay material is 27,435 m³. The height of the dyke is increased by 33% over the concrete wall height, and hence the width of the toe becomes 21.5m wide (Figure 3).

To maintain a 2.58m earthen dyke height, the required size of the footprint increases to 214 m x 214 m (15 % more real estate). The required volume of fill clay material is reduced to 17,788 m³, which is 54 % less clay material, provided the additional 15% real estate is available.

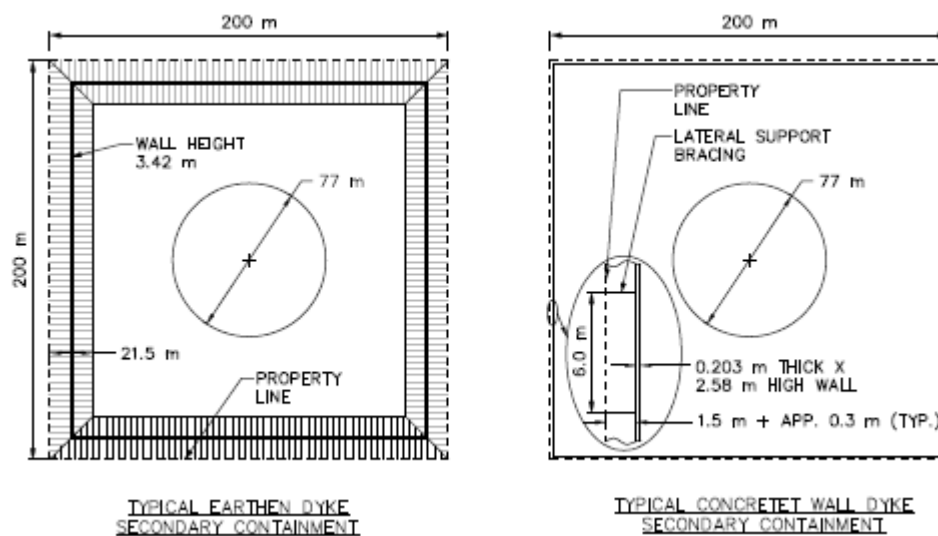


Figure 3: Typical layout of earthen and concrete secondary containments

The selection of the wall type depends on various factors, such as availability of real estate, clay materials, construction equipment, and skilled labour. Cost considerations include construction, operation, maintenance and decommissioning.

The advantages and disadvantages of earthen dykes in comparison to concrete dykes will not be addressed within the scope of this paper.

6. Design of Precast Concrete Dyke; Simplified Approach

Governing design load for dyke walls is hydrostatic pressure that imposes a triangular load distribution on the wall. Resultant force intersects at the 1/3rd distance from the bottom of the wall and hence the bottom section of the wall is subjected to a large bending moment and lateral displacement. It is not economical to design a thicker wall to meet crack control parameters (limit wall slenderness) at maximum displacement. This paper proposes embedding a strong steel member (I-Section) at the bottom of the wall and a weak steel member (C-Section) at the top of the wall such that the combined wall experiences similar top and bottom lateral displacements. The intent is to design a wall that would produce uniform displacement across the wall height and hence the uniform bending stress.

This section summarises design details for a 2.58 m high precast concrete wall panel dyke (Figure 4) required to contain 88,000 m³ outlined in the **Section 5**. It is assumed that the concrete wall panels are supported at every 6 m intervals. Design is based on the CSA Standard A23.4 (2004).

Step 1: Select a trial wall thickness, reinforcements, support spacing to meet crack control parameter;

Wall height;	$b := 2.58\text{m}$	
Wall thickness;	$h := 203\text{mm}$	
Space between lateral supports;	$L_w := 6000\text{mm}$	
Main reinforcement – top (size and quantity);	$\phi_t := 16\text{mm}$	$n_t := 1\epsilon$
Main reinforcement – bottom (size and quantity);	$\phi_b := 16\text{mm}$	$n_b := 1\epsilon$
Transverse reinforcement – (size and quantity);	$\phi_{tbar} := 11.3\text{mm}$	$n_{tbar} := 2C$
Reinforcement cover;	$c_{bar} := 50\text{mm}$	
Strength of concrete and reinforcement steel;	$f_c := 35\text{MPa}$	$f_y := 400\text{MPa}$
Steel and concrete resistance factors;	$\phi_s := 0.8\epsilon$	$\phi_c := 0.6\epsilon$
Elastic modulus of steel	$E_s := 200000\text{MPa}$	
Live load factor;	$\phi_{LD} := 1.5$	

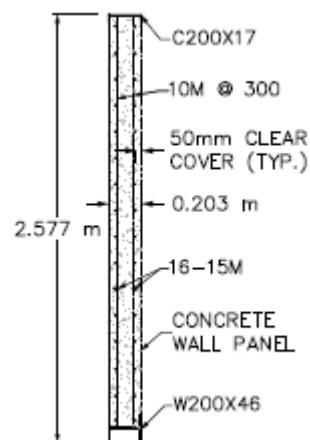


Figure 4 Cross-section of concrete wall panel.

Unit weight of water and concrete;	$\gamma_w := 9.81 \frac{\text{kN}}{\text{m}^3}$	$\gamma_c := 24 \frac{\text{kN}}{\text{m}^3}$
Lateral force due to hydrostatic pressure;	$H_h := \frac{\gamma_w \cdot b^2}{2} \cdot L_w$	$H_h = 195.4 \text{ kN}$
Bending moment due to hydrostatic pressure;	$M_h := \frac{1}{8} H_h \cdot L_w$	$M_h = 146.6 \text{ kNm}$
Factored moment at Ultimate Limit State (ULS);	$M_f := \phi_{LD} M_h$	$M_f = 219.9 \text{ kNm}$
Modulus of rupture (clause 8.6.4);	$f_r := 0.6 \lambda \cdot \sqrt{f_c}$	$f_r = 3.55 \text{ MPa}$
Cracking Moment (clause 9.8.2.3);	$M_{cr} := \frac{f_r \cdot I_g}{y_t}$	$M_{cr} = 62.8 \text{ kNm}$
Where;	$y_t := \frac{h}{2}$	$I_g := \frac{b \cdot h^3}{12}$
Elastic modulus of concrete (clause 8.6.2.2);	$E_c := (3300 \sqrt{f_c} + 6900) \left(\frac{\gamma_c}{2300 \text{ g}} \right)^{1.5}$	$E_c = 29002 \text{ MPa}$
Moment capacity of concrete based on equivalent rectangular stress distribution for doubly reinforced concrete (clause 10.1.7);	$M_r = 159.4 \text{ kNm}$	
Distance to tension reinforcement;	$d := h - c_{bar} - \frac{\phi_b}{2}$	$d = 145 \text{ mm}$
	$d_c := h - d$	$d_c = 58 \text{ mm}$
	$A_c := \frac{2 \cdot d_c \cdot b}{n_b}$	$A_c = 18683 \text{ mm}^2$
Crack control parameter (cl 10.6.1);	$z := 0.6 f_y \cdot (d_c \cdot A_c)^{\frac{1}{3}}$	
$< = 25,000 \text{ N/mm}$ for exterior exposure;	$z = 24651 \frac{\text{N}}{\text{mm}}$	

Step 2: Assume a trial applied moment for concrete section, calculate effective moment of inertia and mid-span deflection of concrete at Serviceability Limit State (SLS);

Trial bending moment of concrete at SLS;	$M := 95.4 \text{ kNm}$	
Effective moment of inertia at SLS (clause 9.8.2.3);	$I_e := I_{cr} + (I_g - I_{cr}) \cdot \left(\frac{M_{cr}}{M} \right)^3$	
	$I_e = 7.6 \times 10^8 \cdot \text{mm}^4$	
Deflection of concrete at SLS (clause 9.8.2.2);	$\Delta := \left(\frac{5}{48} \right) \cdot \frac{M \cdot L_w^2}{E_c \cdot I_e}$	$\Delta = 16.3 \text{ mm}$

Check for slenderness ratio;

Slenderness ratio (clause 9.8.5.3) > 360;

$$\frac{L_w}{\Delta} = 368$$

Step 3: Select a C-Section for top steel and an I-Section for bottom steel with depths of sections equal to the trial wall thickness. Compute bending moments of top and bottom steel due to computed deflection in **Step 2**;

Select C200x17 for top steel and W200x46 for bottom steel;

Moment of inertia and moment capacities of selected sections are;

$$I_t := 13.5 \times 10^6 \text{ mm}^4 \quad M_t := 35.7 \text{ kN}\cdot\text{m}$$

$$I_b := 45.4 \times 10^6 \text{ mm}^4 \quad M_b := 139 \text{ kN}\cdot\text{m}$$

Bending moments of top and bottom steel due to computed displacement (imposing displacement compatibility) at SLS are given by;

$$M_{\text{top}} := \frac{48\Delta \cdot E_s \cdot I_t}{5 \cdot L_w^2} \quad M_{\text{top}} = 11.8 \text{ kN}\cdot\text{m}$$

$$M_{\text{bot}} := \frac{48\Delta \cdot E_s \cdot I_b}{5 \cdot L_w^2} \quad M_{\text{bot}} = 39.5 \text{ kN}\cdot\text{m}$$

Combined bending moment of the wall at SLS;

$$M_{\text{wall}} := M_{\text{top}} + M_{\text{bot}} + M$$

$$M_{\text{wall}} = 146.7 \text{ kN}\cdot\text{m}$$

The combined bending moment of the wall should exceed the bending moment due to hydrostatic pressure at SLS, which;

$$M_h = 146.6 \text{ kN}\cdot\text{m}$$

Repeat **Step 2** and **Step 3** until the combined bending moment exceeds the applied bending moment at SLS, while satisfying the slenderness requirement.

Note: If computed slenderness ratio is less than required slenderness, select higher strength for top and bottom steel or change thickness of wall and repeat the above steps.

As seen above, the assumed bending moment of concrete in **Step 2** satisfies both the slenderness requirement and required moment capacity at SLS.

Step 4: Increase the trial bending moment of concrete selected at **Step 2** gradually. Repeat **Step 2** and **Step 3** until the combined bending moment of concrete and steel exceeds the factored applied moment. This is the design strength check at ultimate limit state;

Trial bending moment of concrete at ULS; $M := 124.3 \text{ kN}\cdot\text{m}$

Effective moment of inertia at ULS (clause 9.8.2.3); $I_e = 5.3 \times 10^8 \cdot \text{mm}^4$

Deflection of concrete at ULS (clause 9.8.2.2); $\Delta = 30.5 \text{ mm}$

Bending moments of top and bottom steel due to computed displacement (imposing displacement compatibility) at ULS are given by;

$$M_{\text{top}} = 21.9 \text{ kN}\cdot\text{m}$$

$$M_{\text{bot}} = 73.8 \text{ kN}\cdot\text{m}$$

Combined bending moment of the wall at ULS; $M_{\text{wall}} := M_{\text{bot}} + M_{\text{top}} + M$

$$M_{\text{wall}} = 220 \text{ kN}\cdot\text{m}$$

The combined bending moment of the wall should exceed the applied factored bending moment due to hydrostatic pressure at ULS, which;

$$M_f = 219.9 \text{ kN}\cdot\text{m}$$

Repeat **Step 2** to **Step 4** until the combined bending moment exceeds the applied factored bending moment at ULS.

As seen above, the combined bending moment is exceeded the applied factored moment at ULS.

Step 5: Check ratios of moment capacities of concrete and steel, and combined wall at ULS;

Concrete bending strength ratio (≤ 1); $\frac{M}{M_r} = 0.78$

Top steel bending strength ratio (≤ 1); $\frac{M_{\text{top}}}{M_t} = 0.611$

Bottom steel bending strength ratio (≤ 1); $\frac{M_{\text{bot}}}{M_b} = 0.531$

Combined wall bending strength ratio (≤ 1); $\text{Ratio} := \frac{M_f}{M_{\text{wall}}} \quad \text{Ratio} = 0.999$

Step 6: Check the location of resultant shear force due to the maximum moment;

Compute by taking the first moment of force exerted by steel and concrete, about the bottom of the wall. The distance to the resultant shear force is $0.38 \times$ wall height. The theoretical location of the resultant force due to hydrostatic pressure is at $1/3$ of wall height.

Step 7: Repeat **Step 1** to **Step 6** until concrete or steel strength ratio in **Step 5** close to 1.0 for optimum design, while bringing the location of resultant force exerted by combined wall close to $1/3$ of the wall height that ensures the uniform deformation and stress distribution across the wall height.

7. Design of Precast Concrete Dyke; Verification by Finite Element Analysis

The concrete wall panel design described in the **Section 7** was modelled using finite element mesh containing 1560 bi-linear shell elements. The embedded steel at top and bottom were modelled using the second order beam finite elements. Simply supported boundary conditions were considered at both ends of the wall.

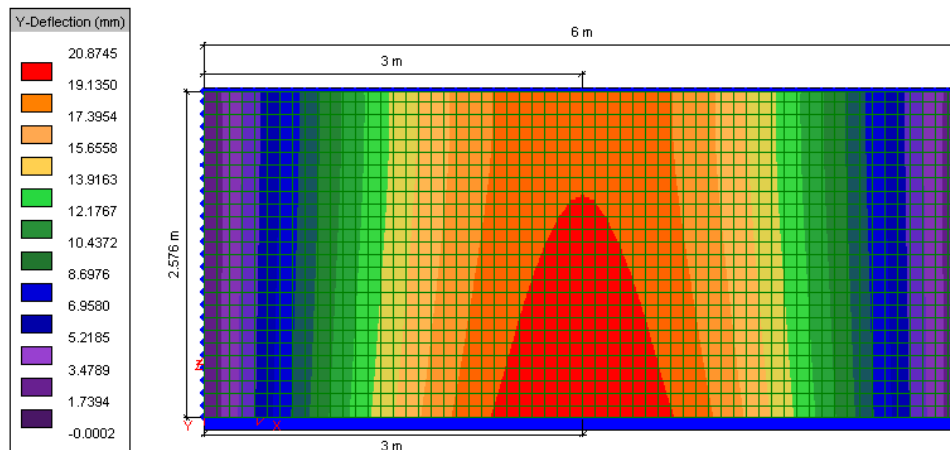


Figure 4: Lateral displacement contours at SLS

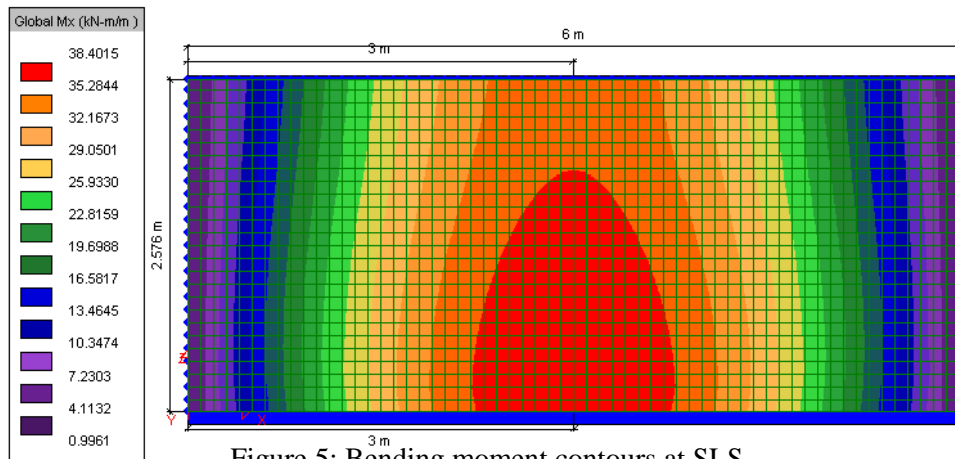


Figure 5: Bending moment contours at SLS

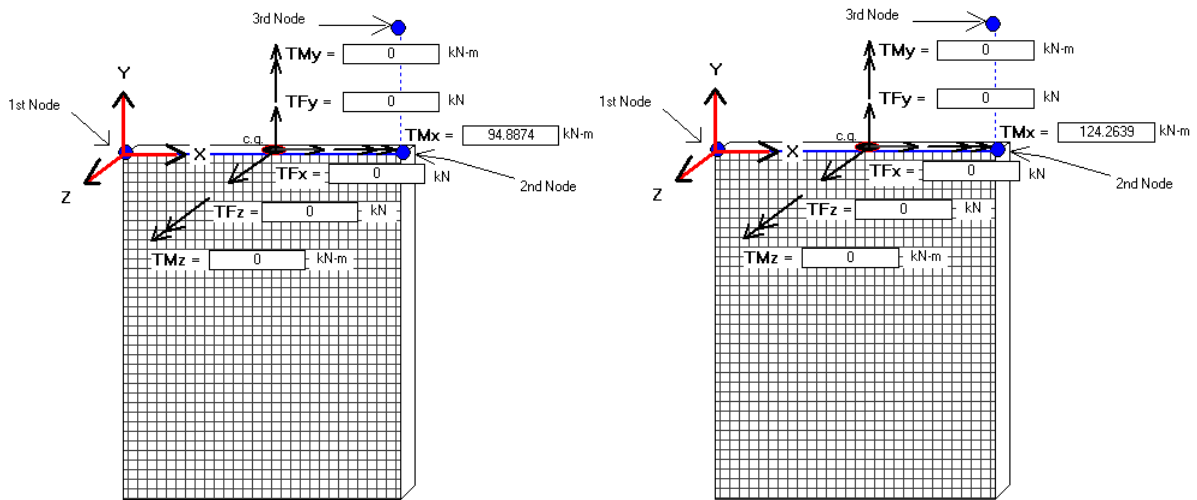


Figure 6: Bending moment in concrete at mid span at SLS and ULS

The lateral displacement and bending moment contours and bending moment in concrete at mid span are shown in Figures 5 and 6.

The results of the proposed simplified method and finite element analysis are summarized in Table 1.

Table 1: Summary of results

	Simplified Approach		Finite Element Analysis		Difference	
	SLS	ULS	SLS	ULS	SLS	ULS
<u>Mid Span Lateral Deflection (mm)</u>						
Top Steel	16.3	30.5	15.3	28.9	1.00	1.60
Concrete	16.3	30.5	15.8	29.4	0.50	1.10
Bottom Steel	16.3	30.5	17.0	31.3	-0.74	-0.80
Average	16.3	30.5	16.0	29.9	0.3	0.6
<u>Bending Moment (kN m)</u>						
Top Steel	11.8	21.9	11.1	20.9	0.73	1.00
Concrete	95.4	124.3	94.9	124.3	0.50	0.00
Bottom Steel	39.4	73.8	40.6	74.7	-1.20	-0.90
Combined Wall	146.7	220.0	146.6	219.9	0.13	0.10

As shown in the table, differences between average displacements and combined wall strengths are 0.3 mm and 0.13 kN m for SLS and 0.6 mm and 0.10 kN m for ULS.

8. Conclusion

The differences in the results obtained from the proposed simplified method are negligible in comparison to the results obtained using finite element analysis. Therefore the simplified proposed method for design of concrete wall panel embedded in steel to deform uniformly across the wall height for hydrostatic loading is justified. Analysis of this problem is complex and this paper proposed a simplified method that involves several steps.

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